US Army Corps of Engineers ®
Kansas City District

Upper Turkey Creek Flood Risk Management Project
Merriam, Kansas

P2 Number: 505229

50% Design Documentation Report

April 2026

This page was intentionally left blank for duplex printing.

Table of Contents

1.0. Introduction 1-1

1.1 General 1-1

1.2 Purpose 1-1

1.3 Authority 1-1

1.4 Location 1-1

1.5 Local Cooperation 1-3

1.6 Historical Performance and Flood History 1-3

1.7 Existing System Description 1-4

1.9 Previous Reports and/or Studies 1-4

1.10 Design Charette 1-5

1.11 Semi-Qualitative Risk Assessment (SQRA) 1-6

1.12 Document Authors 1-6

2.0. Project and Associated Features 2-1

2.1. Existing Project Features 2-1

2.2. Proposed Project Features 2-1

2.3. Major Changes from Feasibility Study 2-1

3.0. Site Development AND Survey 3-1

3.1. Site Access 3-1

3.2. Staging Area 3-1

3.3. Datum 3-2

3.4. Survey Methods 3-2

5.0 Interior Drainage 5-1

5.1 Design Methodology 5-2

5.2 Interior Drainage Hydraulic Modeling 5-5

5.3 Coincident Frequency Analysis 5-21

5.4 Rate of Rise Analysis 5-22

5.5 Analysis of Existing and Proposed Conditions with Theoretical Storm Events 5-24

5.6 Induced Impacts Mapping 5-30

5.7 Interior Drainage References 5-31

6.0 Subsurface Conditions 6-1

6.1 Regional Geology and Geomorphology 6-1

6.2 Historical Subsurface Investigations 6-2
6.3 2023 USACE Subsurface Investigation 6-3
6.4 2025 USACE Subsurface Investigation 6-5
6.5 Future Subsurface Investigation 6-6
6.6 Historical Laboratory Testing 6-6
6.7 Recent Laboratory Testing 6-7
6.8 Subsurface Conditions 6-11
6.9 Engineering Soil Property Development for Design 6-11
7.0. Geotechincal 7-1
7.1. Underseepage 7-1
7.2. Slope Stability 7-7
7.3. I-Wall Analysis 7-15
7.4. Bearing Capacity for Floodwalls 7-17
7.5. Settlement 7-17
7.6. Riverside Slope Protection 7-18
7.7. Levee System Overtopping Resilience 7-23
7.8. Levee Construction Materials 7-23
8.0. Borrow Areas 8-1
8.1. Estimated Borrow Quantities 8-1
9.0 STRUCTURES 9-1
9.1 General Design 9-1
9.2 Design Loads 9-2
9.3 Design Criteria 9-6
9.4 Floodwalls 9-9
9.5 Drainage System Structures 9-11
9.6 Bridges 9-12
10.0. Civil 10-1
10.1. Utilities 10-1
10.2. Bridges 10-2
10.3. Existing Roads 10-3
10.4. Existing Access 10-4
11.0. Environmental 11-1
11. Threatened and Endangered Species 11-1

11.2. Cultural Resources 11-1
11.3. Hazardous, Toxic, and Radioactive Waste 11-1
12.0. Real Estate 12-1
12.1. General 12-1
12.2. Survey 12-1
12.3. Site Specific Information 12-1
12.4. Utility Relocations 12-1
12.5. Site Access 12-1
13.0. Future COnsiderations 13-1
13.1. PM Considerations 13-1
13.2. Design Considerations 13-2
13.3. Communication 13-8
13.4. Environmental Considerations 13-8
14.0. Cost Estimate 14-1
14.1. General 14-1
14.2. Contract Acquisition 14-1
14.3. Estimated Project Duration 14-1
14.4. Basis of Estimate 14-1
14.5. Estimate Preparation 14-1
14.6. Project Owner Markups 14-2
14.7. Cost Drivers and Risks 14-2
14.8. Market Conditions and Competition 14-2
14.9. Scope Uncertainty 14-3
15.0 Project Feature Changes Due to Engineering During Construction 15-1
16.0. References 16-1
16.1. USACE Criteria 16-1
16.2. Federal and State Criteria 16-3
16.3. Associations and Institutes 16-3
16.4. Computer Programs 16-4
17.0. List of Acronyms and Abbreviations 17-1

List of Figures

Figure 1.1. Upper Turkey Creek in Merriam, Kansas 1-2
Figure 3.1. Proposed Staging Areas Figure 3-2
Figure 5.1. Project Features and Stationing 5-1
Figure 5.2. Cumulative Precipitation for Theoretical and Real Precipitation Events – TU03 Gage. 5-5
Figure 5.3 Upper Turkey Creek Watershed 5-7
Figure 5.4. Turkey Creek Interior Drainage Project Area 5-8
Figure 5.5. Stormwater Gage Locations for Calibration. 5-9
Figure 5.6. Turkey Creek Existing Conditions Storm Pipe Network. 5-13
Figure 5.7. Upper Turkey Creek Proposed Conditions Storm Pipe Network. 5-15
Figure 5.8. Detention Basin Drainage Outlet Structure. 5-19
Figure 5.9. Proposed Detention Basin Location. 5-20
Figure 5.10. Detention Basin Implementation in the Hydraulic Model. 5-21
Figure 5.11. 0.1 Annual Exceedance Probability, 24-hour rainfall event: Change in Water Surface Elevation from Existing Conditions to Proposed Conditions (Map Value=Existing Conditions-Proposed). 5-28
Figure 5.12. 0.01 Annual Exceedance Probability, 24-hour Rainfall Event: change in Water Surface Elevation from Existing Conditions to Proposed Conditions (Map value=Existing Conditions-Proposed).. 5-29
Figure 5.13. Peak Induced Impact Mapping. 5-31
Figure 6.1. Boring Locations. 6-4
Figure 6.2. Boring Locations within 2nd Potential Borrow Area 6-6
Figure 6.3. PI – Phi Correlation 1. 6-13
Figure 6.4. PI-Phi Correlation 2 6-13
Figure 6.5. PI-Phi Correlation 3 6-14
Figure 6.6. Lab Results vs No Cohesion Plot. 6-15
Figure 6.7. Drained Strength Function 6-16
Figure 6.8. Curvilinear Strength Function. 6-16
Figure 6.9. OCR vs Depth Trendline. 6-19
Figure 7.1 Levee Design Reaches. 7-4
Figure 7.2 Vertical Tension Crack. 7-10
Figure 7.3. General Levee Minimum Setback. 7-11
Figure 7.4. Minimum Setback for Levees Less than 5 feet Tall. 7-11
Figure 7.5. T-wall Minimum Setback. 7-12
Figure 7.6. I-wall Minimum Setback. 7-12
Figure 7.7. Table 2-5 excerpted from EM 1110-2-1601. 7-19
Figure 7.8. Table 3-1 excerpted from EM 1110-2-1601. 7-20
Figure 7.7.9. Bedding Gradation Criteria. 7-22
Figure 9.1. Cross Section of the Merriam Marketplace Bridge. 14
Figure 9.2. Cross Section of the Shawnee Mission Parkway Pedestrian Bridge. 14
Figure 10.1. Merriam Drive Bridge Typical Section. 10-2
Figure 13.1. Current left bank alignment of floodwall at Merriam Christian Church. 13-7
Figure 13.2: Left Bank Potential Staging Area Expansion. 13-8

List of Tables

Table 1.1. Document Authors 1-6
Table 2-1. Closure Type Requirements[1] 2-2
Table 3.1. Existing Merriam Drainage District Operation and Maintenance Access 3-1
Table 3.2. Proposed Staging Areas Table 3-1
Table 3.3. Ground Penetrating Radar Investigation Summary 3-4
Table 5.1. 12-hour Duration Rainfall Depths for Johnson County, Kansas. 5-4
Table 5.2. 24-hour Duration Rainfall Depths for Johnson County, Kansas 5-4
Table 5.3. Historic Storm Event Information 5-4
Table 5.4. Calibrated Manning's n-value by Land Cover. 5-10
Table 5.5. August 2017 Calibration Results. 5-10
Table 5.6. May 2020 Calibration Results. 5-11
Table 5.7. July 2025 Validation Results. 5-11
Table 5.8. Drainage Structure Pipe Design. 5-16
Table 5.9. Historic Storm Event Maximum Hourly Rate of Rise (ft/hr). 5-23
Table 5.10. Historic Storm Event Warning Time (hrs). 5-23
Table 5.11. Subbasin Properties. 5-24
Table 5.12. 10% AEP Theoretical Event Results 5-26
Table 5.13. 1% AEP Theoretical Event Results 5-27
Table 5.14. Induced Impact Areas. 5-30
Table 6.1. Soil Unit Weight. 6-8
Table 6.2. Consolidated Undrained Test Results. 6-8
Table 6.3. Consolidated Undrained Test Results – Borrow Site (Remolded). 6-8
Table 6.4. Unconsolidated-Undrained Test Results. 6-9
Table 6.5. Consolidation Test Results. 6-10
Table 6.6. Rock Core Compressive Strength. 6-10
Table 6.7. Underseepage Design Soil Parameters. 6-11
Table 6.8. Steady State/Drained Material Properties. 6-17
Table 6.9. End of Construction/Undrained Material Properties. 6-17
Table 6.10. Rapid Drawdown Design Parameters – Nearby Projects. 6-18
Table 6.11. Rapid Drawdown Material Properties. 6-18
Table 6.12. Consolidation Parameters 6-19
Table 6.13. Design OCR vs. Depth. 6-20
Table 7.1. Underseepage Criteria. 7-2
Table 7.2 Levee Design Reaches. 7-4
Table 7.3. Underseepage Analysis. 7-6
Table 7.4. Slope Stability Criteria. 7-8
Table 7.5. Steady State Stability Results 7-13
Table 7.6. End of Construction Stability Results. 7-14
Table 7.7. Rapid Drawdown Stability Results. 7-14
Table 7.8. Required I-Wall Sheet Pile Penetration Depth for Varying Subsurface Conditions (ft). 7-16
Table 7.9. Recommended Riprap Weights. 7-21
Table 7.10. Recommended Maximum sizing for left bank. 7-21
Table 7.11. Recommended Maximum sizing for right bank. 7-21

Table 7.12. Recommended Bedding Gradation. 7-23
Table 9-1. Design Material Unit Weights..... 1
Table 9-2. Typical Soil Properties..... 2
Table 9-3. Seismic Parameters. 6
Table 9-4. Inland Flood Wall Stability Load Cases Adapted from EM 1110-2-2100 Table B-4. 7
Table 9-5. Stability Factors of Safety for Critical Structures. 7
Table 9-6. Reinforced Concrete Design Parameters. 8
Table 10.1. Roadway Impacts. 10-3
Table 10.2. Existing and Proposed Operations and maintenance Access. 10-4
Table 10.3. Existing and Proposed Pedestrian Trail Access. 10-5

APPENDICES

Appendix A – Boring & Lab Data.....
Appendix B – Geotechnical.....
Appendix C – Structural .....
Appendix D – Civil .....
Appendix E – H&H .....
Appendix F – HTRW .....
Appendix G – Preliminary DDR .....
Appendix H – Survey Report.....
Appendix I – SQRA Report .....
Appendix J – Design Charette 2023 & 2024.....
Appendix K – Specification Responsibility Assignment Matrix (SRAM).....
Appendix L – Engineering Considerations and Instructions for Field Personnel (ECIFP) .....
Appendix M – Interior Drainage .....
Appendix N – Real Estate Maps.....

1.0. INTRODUCTION

1.1 General

The Upper Turkey Creek Flood Risk Management (FRM) project is a new project located in the City of Merriam, in Johnson County, Kansas. Efforts as part of this project include placing new levees or floodwalls, utility relocations, and replacing the Merriam and Johnson Drive bridges to increase the current line of protection. The project was designed based on updated Atlas 14 flows and hydrology provided by Johnson County, Kansas.

In accordance with the approved review plan, a facilitated Semi-Quantitative Risk Assessment (SQRA) has been conducted prior to 65-percent submittal. See Appendix I for the draft SQRA Report. The SQRA report will be updated throughout construction.

1.2 Purpose

The Design Documentation Report (DDR) provides the technical basis for the plans and specifications and serves as a design summary. The DDR covers the preconstruction engineering and design phase and the construction phase of the project. The DDR is not considered complete until after the plans, specifications, and construction are complete.

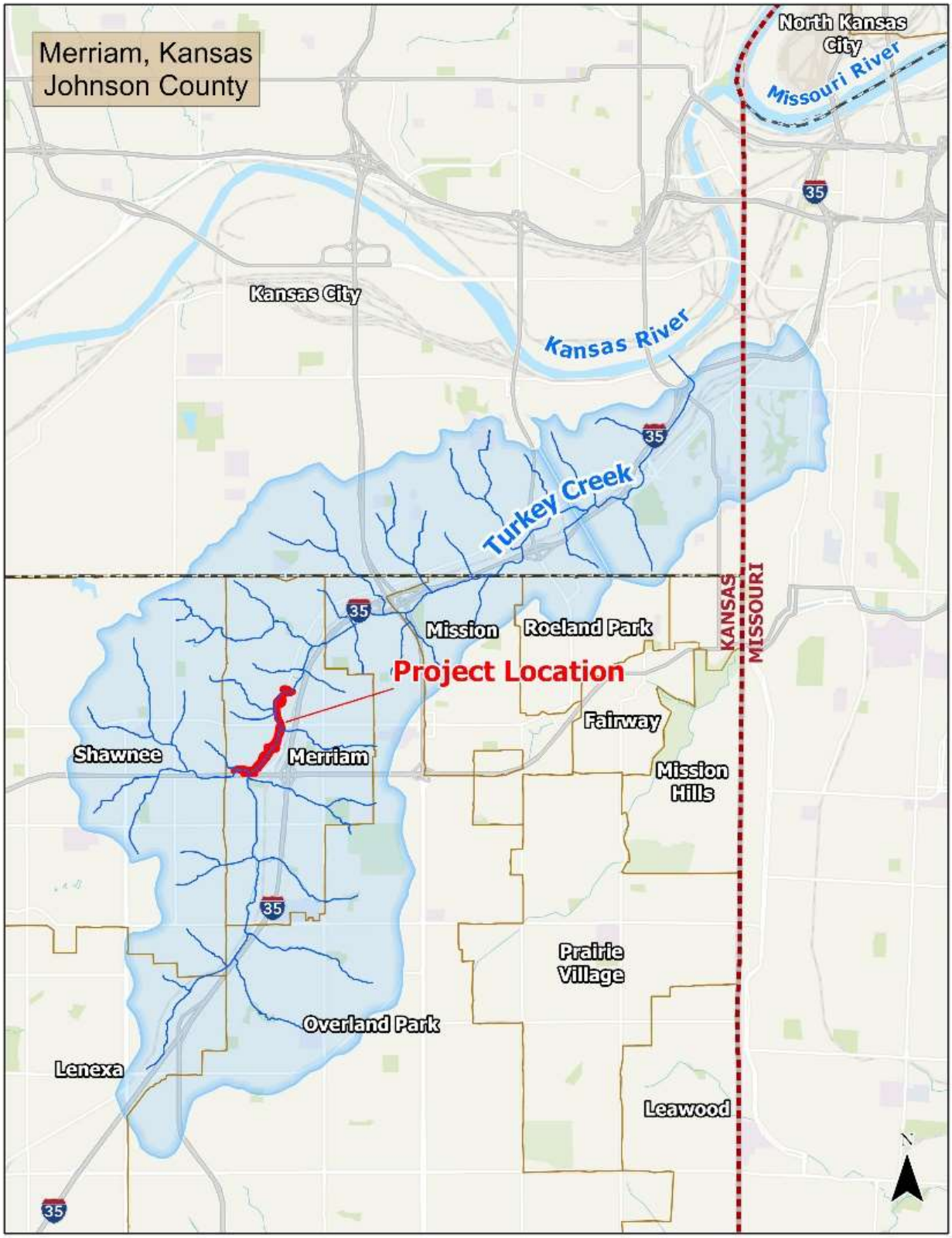
1.3 Authority

The project was originally authorized for a feasibility study by the Resolution of the Committee on Transportation and Infrastructure, U.S. House of Representatives, Docket 2616, adopted 16 February 2000. Authority for the design and construction of this project was provided in the Water Infrastructure Improvements for the Nation Act (WIIN Act) of 2016, Public Law (PL) 114-322, as recommended in the report of the Chief of Engineers dated 22 December 2015, based on the 15 March 2015 Feasibility Report with Integrated Environmental Assessment. Funding was provided in the amount of $500,000 under the fiscal year (FY) 2022 Omnibus funding-Community Project Funding/Congressionally Directed spending and in the amount of $23.9 million under the Infrastructure Investment and Jobs Act, PL 117-58.

1.4 Location

The Upper Turkey Creek FRM project is located in the City of Merriam, in Johnson County, Kansas. Turkey Creek is a right-bank tributary of the Lower Kansas River located in Johnson and Wyandotte counties, Kansas, between approximate river miles (RMs) 2.7 to 3.7. Figure 1.1 shows the location of the Upper Turkey Creek FRM project.

Figure 1.1. Upper Turkey Creek in Merriam, Kansas



1.5 Local Cooperation

The non-federal sponsor for this project is the City of Merriam, Kansas. Johnson County, Kansas, contributes to the city's cost share and is involved in the design phase. Part of the Turkey Creek channel in the study area is operated and maintained by the Merriam Drainage District (MDD) under a state charter that provides MDD authority independent of the City of Merriam or Johnson County. MDD may be responsible for operation and maintenance of this project in the future.

1.6 Historical Performance and Flood History

The Turkey Creek watershed has a history of U.S. Army Corps of Engineers (USACE) involvement. Major flood events occurred in 1951, 1958, 1961, 1968, 1977, 1983, 1986, 1993, 1995, 1996, 1998, and 2017. Although the floods of 1993 and 1998 were both extreme events, the 1998 flood appears to be the flood of record for Turkey Creek, based on high water marks.

Although the 1977 flood is locally known as the Plaza Flood on Brush Creek, significant damage occurred in the Kansas City, Kansas, neighborhood of Rosedale and elsewhere in the Turkey Creek Basin. Flood damage estimates from the September 1977 flood exceeded $8.1 million dollars and prompted the Mid-America Regional Council (MARC), a metropolitan planning organization, to request a USACE study of the Turkey Creek flood problems as a part of an ongoing Urban Study. The Urban Study determined it was not feasible to develop and implement a program for managing flood risks along Turkey Creek that would contain the 1-percent annual exceedance probability (AEP, or 100-year event) and greater flood events.

Following the 1983 flood, the cities of Kansas City, Missouri, and Kansas City, Kansas, requested additional studies on Turkey Creek flooding. In 1987, USACE produced the Reconnaissance Study of Turkey Creek. This commenced a series of studies culminating in an authorized construction project for Lower Turkey Creek in the Water Resources Development Act (WRDA) of 1999.

Flooding in and around Johnson County, Kansas, led to presidential disaster declarations in 1993 and 1998. The estimated cost of these disasters exceeded $50 million. Heavy rains on 4 October 1998, produced flooding that caused several million dollars in damage to businesses and public property along Turkey Creek in Johnson and Wyandotte counties, Kansas. Reports indicate that parts of the Kansas City metropolitan area, in both Kansas and Missouri, received almost eight inches of rain that day with some locations receiving three to five inches of rain within a three-hour period. More than five inches fell in Lenexa, Kansas, in a little more than an hour. Nearly 100 calls for water rescue were received, and numerous roads throughout the area, including individual sections of Interstate Highway 35 (I-35) and Interstate Highway 435 (I-435), were impassible.

Following the 1993 and 1998 flood losses and as Lower Turkey Creek investigations were ongoing, the U.S. House of Representatives Committee on Transportation and Infrastructure passed the 1999 resolution and adopted the Upper Turkey Creek Basin Study authority in 2000. The 1999 resolution favored a watershed perspective to future water resource development proposals. In response to the study authority, the reconnaissance phase of the study was initiated on 16 February 2000 (USACE 2001). This phase of the study found there was a federal

interest in continuing the study into the feasibility phase. The Kansas City District, along with local sponsor, the City of Merriam, Kansas, initiated the feasibility phase of the Upper Turkey Creek watershed in June 2002 to examine measures for FRM.

As of 2012, Lower Turkey Creek has an active USACE construction project. One component is the repair of an aging tunnel that diverts all flow from the Turkey Creek watershed through a bluff to the Kansas River. The channel has also been widened.

Three significant rainfall events occurred in the Turkey Creek watershed during the summer of 2017 from 26 through 27 July, 5 through 6 August, and 21 through 22 August. Storm total precipitation was highest between 21 and 22 August 2017, but the intensity was greater for 26 to 27 July 2017 and 5 to 6 August 2017. Rainfall from the events produced some of the highest stages along Turkey Creek since the flood of record on 4 October 1998. Of the events in 2017, 26 to 27 July had the highest stage and flow rate at the Johnson Drive gage, while 5 to 6 August produced the highest stage and flow rate at the Southwest Boulevard gage. The flood of 6 August 2017 likely had a peak discharge comparable to the 1977, 1983, 1986, and 1993 floods and would have caused significant damage if the constructed flood damage reduction projects had not been in place in the lower reach of Turkey Creek.

1.7 Existing System Description

The Turkey Creek watershed contains some of the most intensely developed urban locations in Johnson and Wyandotte counties, Kansas. Approximately 75 percent of the watershed consists of residential, commercial, and industrial land use. Most of Turkey Creek and its tributaries have been channelized. The need for the project lies in the potential for flooding that could result in life loss (LL) and/or property damage in the Upper Turkey Creek Basin.

1.8 Protected Area Description

The proposed protected area for this project is heavily urbanized, including downtown Merriam, Kansas, with most of the properties consisting of commercial or industrial land use. The majority of the Turkey Creek channel that passes through the City of Merriam, Kansas, can contain a 50-percent AEP (1/2-year event), although two areas flood at the 20-percent AEP (1/5-year event).

1.9 Previous Reports and/or Studies

- **Flood Protection Project, Turkey Creek, Merriam, Kansas, USACE, September 1962.** This Design Memorandum reevaluated the economic justification of a flood protection plan developed in 1946. The project was designed to alleviate flooding along Turkey Creek in Merriam, Kansas. The plan of improvement called for constructing levees along the banks, a short section of floodwall, a new channel through the entire length of the improvement area, and the modification of several bridges and roads. The report revised the recommendations for conditions, as of 1962, and provided a map of improvements and typical channel sections. This project was not constructed.
- **Letter Report for Proposed Inclusion of Turkey Creek Improvements, Modification of Local Protective Works, Kansas River, Kansas City, Kansas (Flood Control Act of 1962), USACE, Kansas City District, May 1968.** The report described the Turkey Creek flood problem and proposed a solution for addressing flood hazards. The report

Creek flood problem and proposed a solution for addressing flood hazards. The report recommended a new 28-foot-diameter tunnel to be driven adjacent to the existing tunnel, and an enlarged and deepened channel extending 11,400 feet upstream from the existing tunnel entrance. The proposed plan was never authorized or constructed.

- **Flood Plain Information Report for Turkey Creek in Metropolitan Kansas City, USACE, January 1974.** The report was authorized by Section 206 of the 1960 Flood Control Act (Public Law 86-645) as amended. The report included an evaluation of the flooding hazards associated with Turkey Creek in the metropolitan Kansas City area. The report was prepared at the request of the cities of Kansas City, Merriam, and Overland Park, Kansas, the Kansas Water Resources Board, and Kansas City, Missouri.
- **Upper Turkey Creek Basin, Johnson and Wyandotte Counties, Kansas: Section 905(b) Analysis, USACE, July 2001.** This report evaluated the potential federal interest in solutions to recurring flood damages, environmental degradation, and related water and land resource needs and opportunities in the Upper Turkey Creek Basin. A positive determination of federal interest and recommended approval by the USACE District Engineer led to development of the 2002 Project Management Plan (PMP).
- **Stormwater Management Ordinance, Chapter 7, Article 1 of City code of ordinances, City of Merriam, June 24, 2002.** This article establishes the stormwater run-off management criteria and standards for the city's secondary or major drainage system and its components (Ordinance No. 1400, §1, 3-26-01). The secondary or major system includes the primary/minor system, its overflow elements, and all Turkey Creek tributaries and drainage structures both public and private that are not identified as part of the Turkey Creek regulatory floodplain located within the City of Merriam, Kansas.
- **Northeast Johnson County Watershed Study, Johnson County, Kansas, 2005.** Johnson County Public Works conducted a watershed study of the major creeks and streams in the Northeast corner of the County. The Northeast watershed is approximately 38.5 square miles and contains 5 watersheds (Brush Creek, Dykes Branch, Lake Quivira, Rock Creek, and Turkey Creek). The project includes data collection—channels, culverts, bridges, and low opening elevations on houses and business structures in or near flood-prone areas—public meetings, and development of a plan to address future stormwater issues. The study generated new floodplain maps, identify flood prone areas and other problems (stream erosion), alternative solutions will be considered.

1.10 Design Charette

During the 35 percent design stage, USACE, the City of Merriam, Johnson County, and MDD held a design charette focused within the Merriam downtown reach of the project to explore additional alternatives besides the recommended plan from feasibility. Ultimately, it was determined the alternatives would require a significant amount of real estate acquisition and not result in significantly lower levees to deviate from the recommended plan. Appendix J contains the findings and conclusion of the design charette.

1.11 Semi-Qualitative Risk Assessment (SQRA)

A SQRA was completed after the 35 percent design stage per the approved review plan. See Appendix I – SQRA Report for more details.

1.12 Document Authors

Table 1.1. Document Authors

Lead Author	Role	Registration
Kailey Jones	Project Manager	
Courtney Forsythe	Lead Engineer	PE (MO)
Conner Szarwinski	H&H Engineer	
Lucas Gannon	H&H Engineer	EIT
Austin Noll	Geologist	
Derek Jenson	Geotechnical Engineer	PE (KS, MO, NJ)
Paige Blasberg	Geotechnical Engineer	
Maeve VanLandingham	Civil Engineer	PE (MO)
Forrest Pritt	Civil Engineer	
John Westbrook	Structural Engineer	PE (KS, MO)
Eddie Fernandez	Structural Engineer	EIT (MO)
Olivia Jackson	Environmental	
Gina Powell	Cultural Resources	
Paul Speckin	HTRW	
Rachel Edkin	Real Estate	
Andy Yoakum	Cost Engineer	

2.0. PROJECT AND ASSOCIATED FEATURES

2.1. Existing Project Features

The existing Turkey Creek channel in the project area has approximately 1V:1H to 1V:2H side slopes and is mostly lined with stacked limestone blocks. The channel is generally less than 8 feet in depth. The limestone block retaining wall was built in sections from the 1960s to the 1980s and sections of it have been repaired since. The wall is maintained by the MDD. MDD inspects this area of the creek twice a year. They estimate that ten percent of the original wall has been replaced.

The limestone block retaining wall is a gravity block wall with no reinforcements tying the blocks together. The individual blocks are approximately 6 feet long, 2 feet deep, and 14 to 18 inches tall. The face of each block in the stack is offset approximately 2 inches back from the face of the block below it. There is no drain behind the wall. The wall was backfilled with local soil which appears to be mainly clay. In some recent replacement sections of wall, filter fabric and aggregate backfill were placed behind the wall.

The wall experiences regular breaches with short sections (roughly 100 feet long or less) completely collapsing into the creek (see Section 4.0 of Appendix B-5 of the Feasibility Study). The cause of the breaches is unknown, but the wall was described in the Feasibility Study as, “not an engineered structure.” When breaches occur, MDD rebuilds the section with the original limestone blocks. During site visits for the Feasibility Study, soil was observed migrating through the limestone blocks. The walls were judged to not be capable of providing support to the proposed levee or floodwall.

2.2. Proposed Project Features

The primary components of the project are earthen levee, floodwalls, bridge replacements, detention basin, and drainage structures. The overall project is approximately 1.5 miles long. A majority of the existing limestone block wall will remain, but the floodwall/levee will be set back to limit the impacts of failure of the existing wall (see Section 7.2.3). The maximum levee height is approximately 10 feet, and the floodwall height ranges from 6 to 12 feet. The levee embankment consists of compacted impervious material.

The project is broken out into two Contracts. One for Merriam and Johnson Drive bridge replacement, and a second for the levees and floodwalls construction. The bridge contract will be completed prior to the levees and floodwall contract.

2.3. Major Changes from Feasibility Study

Several items have been updated since the Feasibility Study was completed. Appendix G discusses Feasibility Study design and analyses in detail in the Preliminary DDR. This DDR discusses current criteria, design, and analysis. The following subsections discuss the major changes that have increased the scope of work for this project.

2.3.1. Bridges

There are two bridges within the project limits that must be removed and replaced that previously had either no modifications or minor modifications. Due to increased water elevations and further analysis, the existing bridges at Johnson Drive and Merriam Drive do not meet the required uplift or overturning criteria to remain. The new bridges for Merriam Drive and Johnson Drive will be Kansas Department of Transportation (KDOT) slab bridges. See Part 09– Structures for more discussion.

2.3.2. Hydraulics

Due to updates in hydraulic modeling from the Feasibility Study, and implementation of surveyed channel bathymetry, the existing conditions' hydraulic profile has changed, and an update to the design profile has been performed. Coordination between USACE and project sponsors will be required to select the adopted design profile based on the validation effort and current hydraulic analysis. Channel armoring and maintenance efforts have been an ongoing process with MDD since the Feasibility Study was completed.

Unsteady two-dimensional (2D) hydraulic modeling is being used to update the design profile in consideration of the increased level of detail and hydraulic modeling capabilities, as well as changes to the stream channel that have occurred since the completion of the Feasibility Study. Unsteady flow hydrographs computed during the Feasibility Study are being adapted and used for the current modeling effort to compare the steady flow profiles from the Feasibility Study with the unsteady flow profiles generated during this modeling update and validation effort. Appendix E presents further detail regarding the modeling validation and development.

2.3.3. Storm Drainage Structures

EM 1110-2-2902, *Conduits Pipes and Culverts Associated with Dams and Levee Systems* was updated in December 2020, after the completion of the Feasibility Study. Closure type requirements based on pipe diameter and flood rise are now prescribed in the updated EM 1110-2-2902. The closure type requirements are for new construction but are also highly recommended for retrofitting existing pipes. Table 2.1 summarizes the active closure requirements.

Table 2-1. Closure Type Requirements[1]

Flood Rise[2]	Pipe Diameter (inches)	Closure Configuration[3]
Fast	<36	One Passive
Fast	≥36	One Passive AND One Active
Slow	<36	One Passive OR One Active
Slow	≥36	One Passive AND One Active OR Two Active

[1]Table 9-1 of EM 1110-2-2902

[2]Fast flood rise refers to floods capable of rising to a flood stage with less than 12 hours prediction time. Slow flood rise refers to floods capable of rising to a flood stage with greater than 12 hours prediction time.

[3]Passive closure are typically flap gates. Active closures are typically sluice gates.

During the SQRA, the project delivery team (PDT) recommended a deviation from Table 2.1 to use one passive closure for pipes greater than or equal to 36 inches. This recommendation is based on concerns that it is not feasible for the sponsor to manually operate more than 10 sluice gates throughout the project with less than 12 hours prediction time. Note that EM 1110-2-2902, paragraph 9.4.1.3.1, states, "Active gates are used when the rate of rise of water is slow enough to allow ample time for complete operation of the gates (minimum 12-hour flood prediction time according to EM 1110-2-1913)." This contradicts Table 2.1 (EM Table 9-1PD) but agrees with the PDT's recommended deviation. See Appendix I for the Draft SQRA Report and Draft Secondary Closure Variance request.

3.0. SITE DEVELOPMENT AND SURVEY

3.1. Site Access

There are limited site access options as the proposed project is within a fully developed urban area of Merriam, Kansas, parallel to I-35 and adjacent to railroad tracks. Site access for construction and routine maintenance and operation of the project will require coordination with various stakeholders. Stakeholders include the City of Merriam, MDD, KDOT, commercial properties, and private properties. Sheet G-004 of the plan set shows contractor access during construction.

Table 3.1. Existing Merriam Drainage District Operation and Maintenance Access

Bank	Approximate Upper Turkey Creek Stationing	Details
Right	27+00	Access to creek off of West 61st Street
Right	61+74	Access to creek immediately south of Merriam Drive Bridge

See Section 10.4.1 for proposed O&M access.

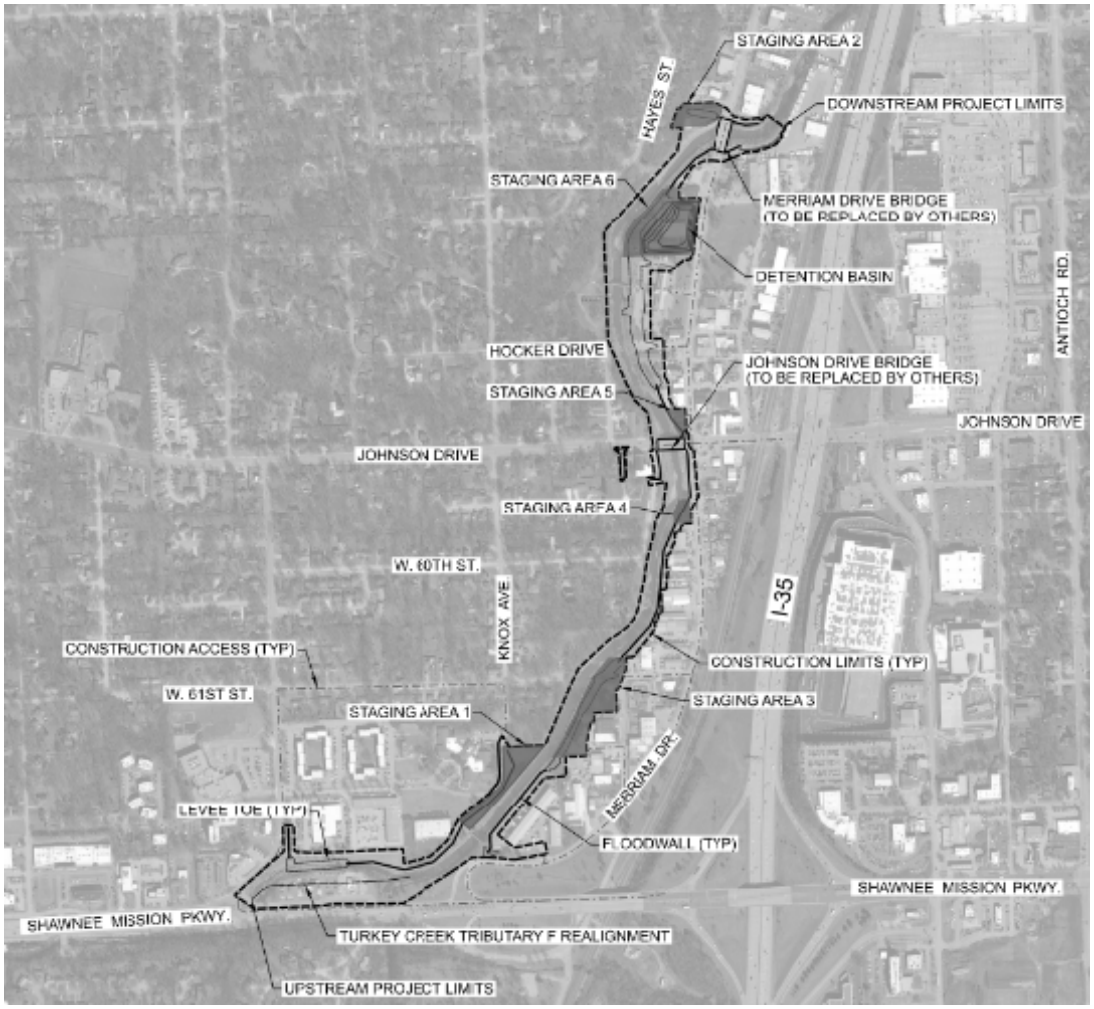
3.2. Staging Area

Table 3.2 lists proposed staging area locations. Figure 3.1 shows the staging areas of the project.

Table 3.2. Proposed Staging Areas Table

Staging Area	Bank	Details
Staging Area 1	Left Bank	South of W. 61st Street. Access via Knox Avenue
Staging Area 2	Left Bank	North of Merriam Drive Bridge. Access via Merriam Drive. Also use for Merriam Drive Bridge replacement.
Staging Area 3	Right Bank	North of West 61st Street. Access via Merriam Lane.
Staging Area 4	Right Bank	South of Johnson Drive. Access via Merriam Drive.
Staging Area 5	Right Bank	North of Johnson Drive. Access via Merriam Drive Also use for Johnson Drive Bridge replacement.
Staging Area 6	Right Bank	Detention Basin Area. Access via Merriam Drive.

Figure 3.1. Proposed Staging Areas Figure



3.3. Datum

The horizontal datum for this project is Kansas State Plane North (1501), North American Datum of 1983 (NAD 83) in U.S. survey feet. The vertical datum for this project is North American Vertical Datum of 1988 (NAVD 88), in U.S. survey feet. Ellipsoid heights acquired by Global Positioning System (GPS) were converted to NAVD 88 using GEOID18. All survey control was connected to the National Geodetic Survey (NGS).

3.3.1. Datum Conversion

For the purposes of this analysis, the approximate conversion between the National Geodetic Vertical Datum of 1929 (NGVD 29) and NAVD88 is +0.41 feet. Therefore, an NGVD 29 elevation of one foot would be assumed to be identical to a NAVD 88 elevation of 1.41 feet. Datum conversion was determined using the U.S. NGS Coordinate Conversion and Transformation Tool. All elevations referenced in this report are NAVD 88 unless stated otherwise.

3.4. Survey Methods

A topographic survey was completed by Stockwell Engineers Inc. from February 2023 to April 2023. NGS point R328 was used to establish the elevation of CP-01 toward the south end of the

site near 9800 W. 62nd Terrace (known as Skate City). Two separate Online Positioning User Service (OPUS) solutions were performed on the point to obtain an accurate position. Checks were then completed on NGS points R328 and Shawnee2.

All survey field notes were kept in a field book and given to USACE electronically. Manhole Information Sheets were also kept to document the interior of manholes with depths to inverts, size and material of pipes, and present use.

Underground utility locates were requested through Kansas State One Call System multiple times by Stockwell Engineers, Inc. during their ground survey efforts but limited locates were marked in the field. USIC, LLC was the utility location contractor through Kansas State One Call. Stockwell Engineers Inc. contacted all utility companies that were part of the Kansas State One Call System and spoke with representatives of the companies to obtain their locating contractor to mark the lines in the field.

A ground penetrating radar (GPR) investigation was also conducted between 20 through 24 February 2023 at six locations within the project area. According to the Environmental Protection Agency (EPA) these locations have existing or abandoned underground storage tanks (UTSs).

GPR identified eight underground void shapes within the project area. Table 3.3 summarizes these GPR locations.

Table 3.3. Ground Penetrating Radar Investigation Summary

GPR Number	Approximate Depth (feet)	Approximate Surface Area (square feet)	Approximate Location along UTC Alignment	Bank (Left or Right)
1	Undetermined	103.6	26+75	R
2	4.5	235.8	28+00	R
3	3.3	89.1	29+75	R
4	Undetermined	319.3	53+30	R
5	5.2	77.7	53+50	R
6	1.9	13.8	64+00	R
7	Undetermined	133.7	62+50	L
8	3.3	55.7	63+00	L

Stockwell completed a boundary survey in September 2023. The boundary survey deliverable included written legal descriptions for all parcels within the project area, closed boundary lines in MicroStation, and title reports for 160 parcels.

Project extents near Turkey Creek Tributary F and the proposed detention basin have expanded since the 2023 topographic survey request. LiDAR data from Johnson County Automated Information Mapping System (AIMS) was used to supplement the topographic survey in areas outside the survey extents where there are proposed project features. The LiDAR was acquired in November 2020 by Airborne Imaging. LiDAR date is in horizontal datum Kansas State Plane North and vertical datum NAVD 88.

4.0 HYDROLOGY AND HYDRAULICS

4.1 Hydrology

Hydrology used for design of the levee profile was previously developed as part of a 2005 Johnson County hydrologic study using Hydrologic Engineering Center 1 (HEC-1), which was a precursor to Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS). HEC-1 is a software that simulates infiltration and runoff in a watershed or network of watersheds due to an input storm event. Runoff hydrographs can be generated at desired locations to serve as frequency inflow hydrographs for hydraulic modeling in Hydrologic Engineering Center River Analysis System (HEC-RAS).

Rainfall frequencies used for the HEC-1 model to compute runoff were originally developed using the 24-hour precipitation maps included in the National Oceanic and Atmospheric Administration's (NOAA's) Technical Paper 40 (TP-40). In 2013, updated rainfall estimates for midwestern states were published by NOAA in a report titled, *NOAA Atlas 14–Precipitation Frequency Atlas of the United States–Volume 8: Midwestern States* (Atlas 14). This occurred during the USACE Upper Turkey Creek Feasibility Study (FS), which used the 2005 Johnson County study hydrology. It was suggested to use these updated precipitation frequency depths from Atlas 14 and generate updated frequency flows in the HEC-1 model. The annual maximum series was used for precipitation depths from a single point location for all frequencies. The single point location was determined to be suitable as all locations within the upper portion of the watershed used the same Atlas 14 precipitation depths. Table 4.1 provides a comparison of the precipitation frequency depths for Turkey Creek watershed with a 24-hour storm duration. Note the Atlas 14 precipitation depths were larger for all frequencies when compared to the TP-40 values.

Table 4.1. Annual Exceedance Probability and Precipitation Totals for 24-hour Storm Events at Turkey Creek.

AEP	Return Interval (years)	Precipitation Depth, TP-40 (inches)	Precipitation Depth, Atlas 14 (inches)
50%	2	3.6	3.7
20%	5	4.5	4.7
10%	10	5.3	5.5
5%	25	6.2	6.8
2%	50	6.9	7.8
1%	100	7.8	8.8
0.5%	200	8.4	9.9
0.2%	500	9.3	11.4

Table 4.2 summarizes the hydrologic parameters used in the HEC-1 model. Atlas 14 precipitation depths replaced the original TP-40 precipitation depths to generate the updated rainfall runoff flow hydrographs. The flow hydrographs were computed at various locations along Upper Turkey Creek to serve as inflow hydrographs in the HEC-RAS hydraulic models for the feasibility study. These same flow hydrographs and locations are being used with the current 1D and 2D hydraulic models for this study. Appendix E.1 summarizes inflow boundaries and locations while Appendix E.2 contains inflow hydrographs.

Table 4.2. Feasibility Report HEC-1 Modeling Parameters.

Process	Parameter	Data Source
Precipitation	Hypothetical Storm	Atlas 14 Precipitation Depths
Infiltration	SCS Curve Number Method	2005 Johnson County Hydrologic Study
Transform/Runoff	SCS Dimensionless Hydrograph	2005 Johnson County Hydrologic Study
Flow Routing	Muskingum-Cunge	2005 Johnson County Hydrologic Study
	Modified Puls	2005 Johnson County Hydrologic Study

4.2 Hydraulics

A fully 2D model was developed by Stantec on behalf of Johnson County, Kansas who is acting as a Cooperating Technical Partner (CTP) with the Federal Emergency Management Agency (FEMA). The purpose of the model, developed in HEC-RAS version 6.5, is to develop base flood elevations and floodplain boundaries for the streams within the Turkey Creek watershed. Stantec has provided this model for USACE use in the concurrent Upper Turkey Creek design project. The 2D rain-on-grid model was developed to perform both hydrologic and hydraulic computations and provide rainfall runoff flood mapping as well as the hydraulic response of Turkey Creek from local inflows. This model was adopted moving forward for 50% design.

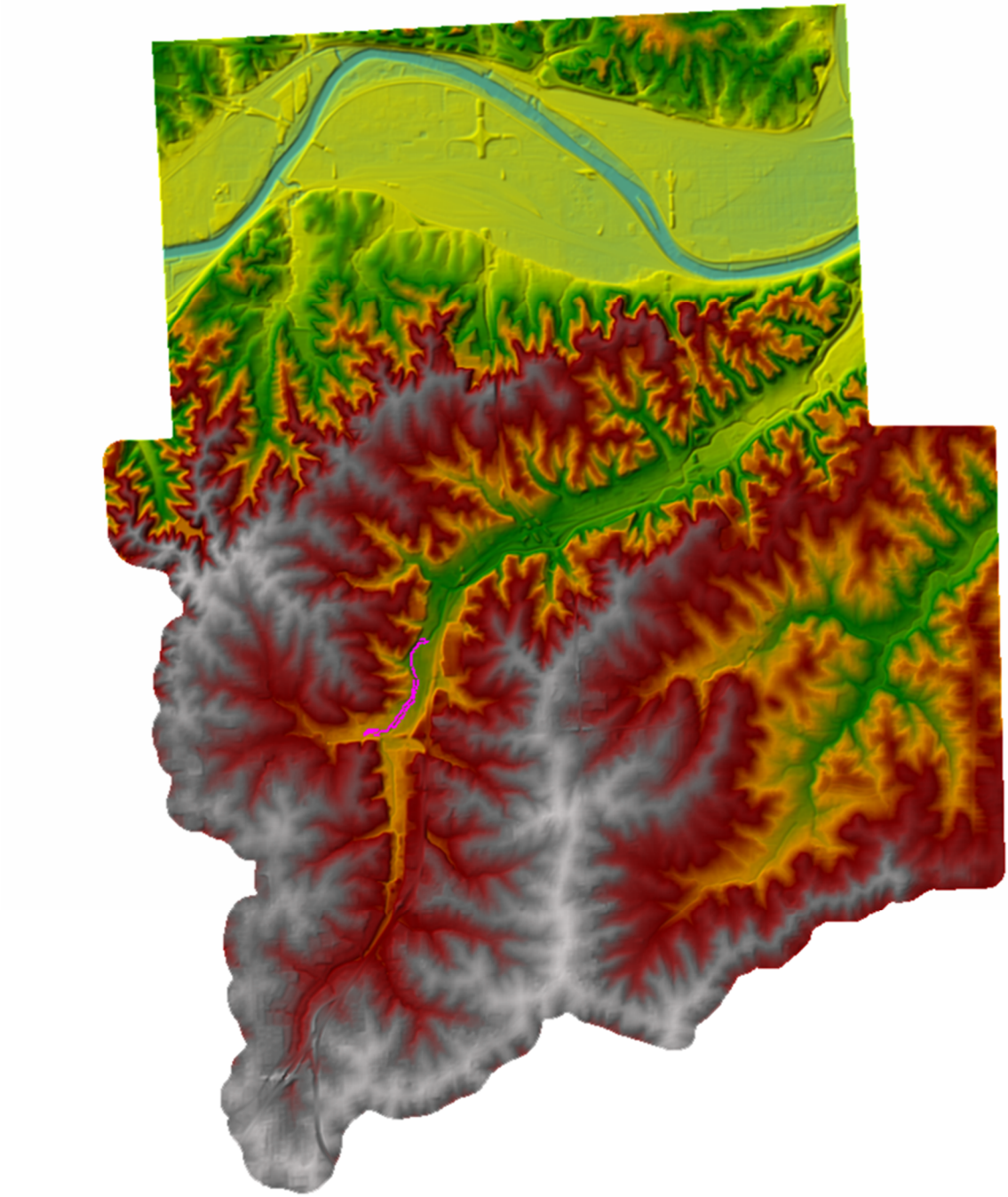
The original development of this model is documented in Appendix E.7, which contains Stantec's Turkey Creek 2D Watershed H&H Report. Appendix E.1 contains information on the modifications of the Stantec 2D model to suit the purposes of this design project.

Terrain data was provided by Johnson County through the Automated Information Mapping System (AIMS) for Turkey Creek and Brush Creek (2-ft resolution). The LiDAR was flown in December of 2020. For the portion of Turkey Creek that is outside of Johnson County, located in Wyandotte County, supplemental LiDAR was downloaded from Kansas' Data Access and Support Center (DASC) at 1-meter resolution. The LiDAR acquired from DASC was collected in 2018. The DASC LiDAR was reprocessed to match the resolution of the Johnson County LiDAR and were mosaiced together.

Cross-sectional channel survey was collected along Turkey and Brush Creeks. This channel bathymetry was burned into the terrain where differences in survey and LiDAR indicated that the LiDAR was not capturing the true channel bottom. The channel survey was interpolated and merged with the LiDAR to account for the additional conveyance not captured in the LiDAR. For Turkey Creek, surveyed cross-section locations showed little to no difference between the surveyed elevations and the LiDAR suggesting that the LiDAR was flown during a low-flow condition in the channel and adequately captures the channel conveyance. The survey was incorporated into the final 2D model terrain, interpolating between survey locations, and retaining only survey locations where the hydro-flatted LiDAR was higher than the survey.

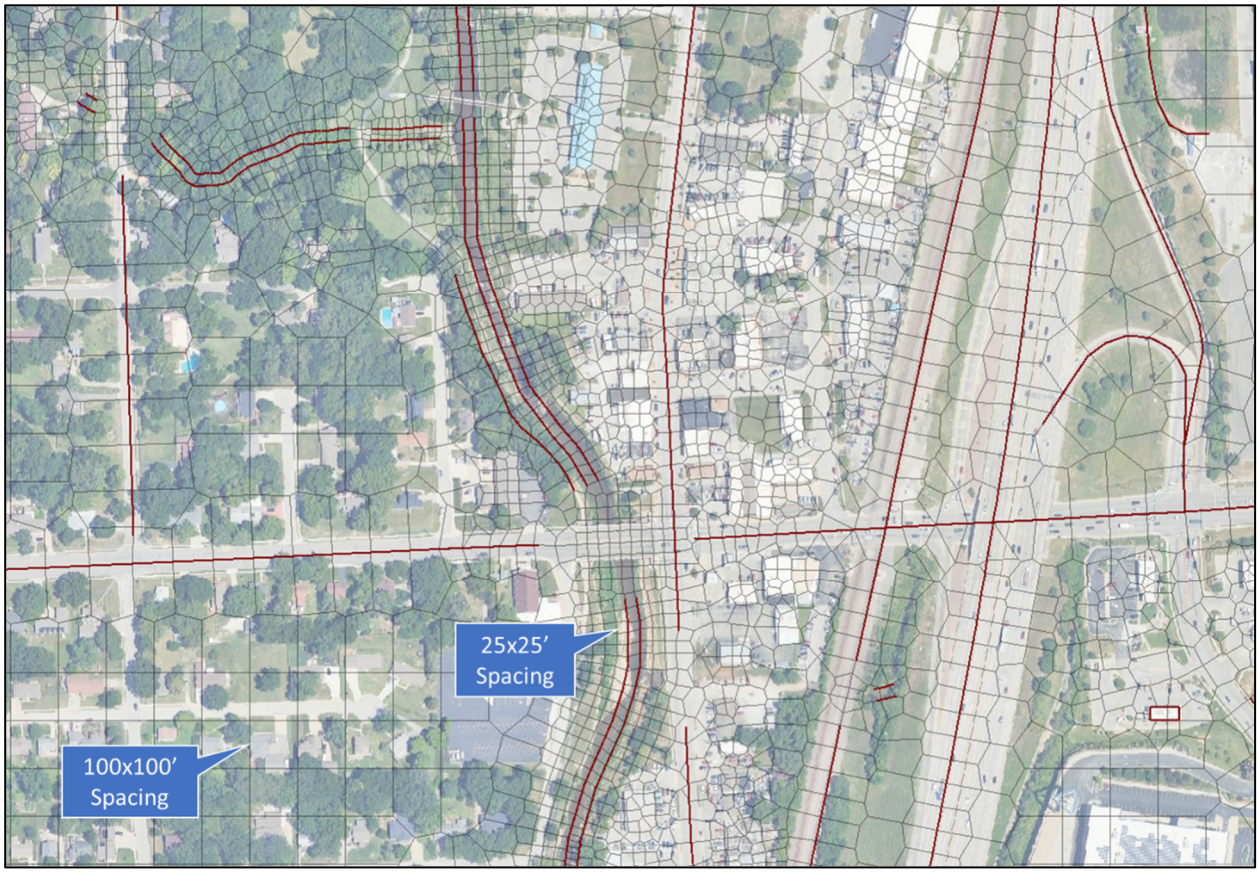
Figure 4.1 shows the terrain and proposed flood protection alignment in pink. The terrain encompasses almost the entirety of the Turkey watershed.

Figure 4.1. Terrain used for Hydraulic Modeling.



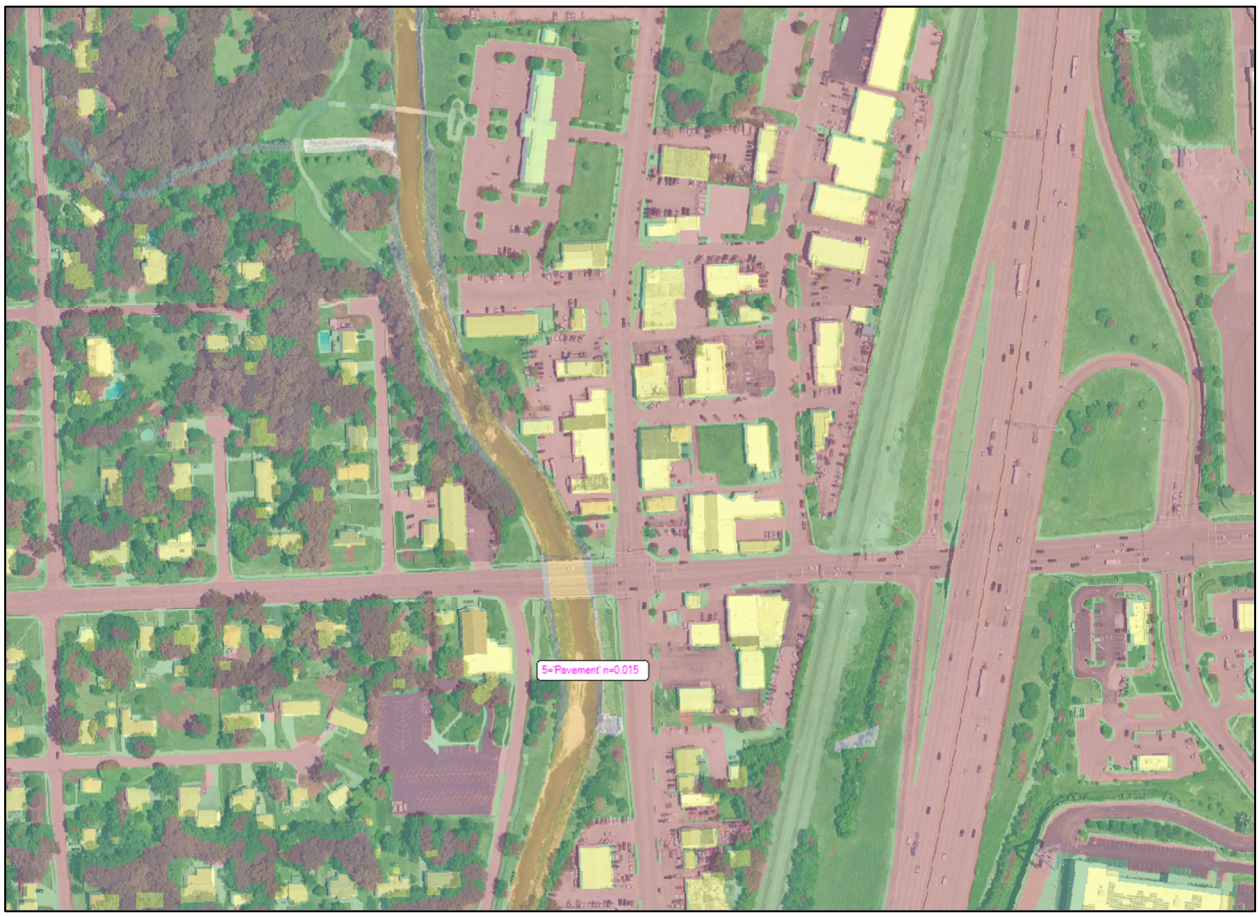
The HEC-RAS 2D mesh generally used cell sizes of 25x25 feet for the channel and 100x100 foot cell sizes for the floodplain. Breaklines were utilized to enforce high ground and better align cells to flow directions. Figure 4.2 provides an example of the 2D mesh.

Figure 4.2. Example of 2D Mesh in Project Area.



A custom land cover layer from the original Stantec model development was utilized for the design model. The layer acknowledges pavement and building footprints as well as channels. This is ideal given the smaller and heavily urbanized nature of the project area. The National Land Cover Database (NLCD) land cover layer was too coarse for the project area and was not used. Figure 4.3 provides an example of the land cover layer.

Figure 4.3. Example of Land Cover Layer.



4.2.1 Calibration/Validation

Stantec conducted calibration and validation of the 2D hydraulic model during original development, which was documented in their modeling report, attached in Appendix E.7. The model was calibrated with August 22, 2017, and May 28, 2020, events and the July 2017 event was used for validation. It should be noted that calibration and validation of the FEMA model was completed using gridded precipitation of these historic events. There is no observed gage flow to serve as model inflows nor as calibration points. USACE found general uncertainty in the efficacy of channel and floodplain calibration of this model due to the use of rain-on-grid in such an urban area and a lack of stormwater modeling which significantly impacts routing times and volume accumulation. For this reason and to account for uncertainty, Manning's n-values for the channel were increased from 0.025 to 0.035 to account for any potential for underestimation of stages during levee profile design. The channel has seen sediment deposition over the last decade, so these changes were deemed appropriate.

Additionally, Manning's n-values meant to represent buildings were initially set to 1.0 in the model. This is typically only appropriate if cells with a 1.0 roughness value are exactly encompassing the building footprint. Otherwise, the large 1.0 value significantly skews the roughness where flow should be able to actively convey. In these situations, the HEC-RAS

user's manual recommends setting the roughness values of buildings in the 2D mesh to 0.5. Since cells were not specifically configured to represent only the building footprints, the building roughness value was changed to 0.5. Cell alignments within the channel and around critical areas were improved.

The model cannot be calibrated in a non-rain-on-grid configuration due to the previously mentioned lack of observed flow data. This represents a major uncertainty in the hydraulics. HEC-RAS best practices were considered, and geometries were developed in such a way to lean towards conservatism given the lack of calibration data. As of the 50% design submittal, there are no remaining active flow gages downstream of the project area. While recent high water marks were collected for the July 2025 flood event, no gages along Turkey Creek recorded a peak flow estimate to correspond to the surveyed stages. Additionally, given recent findings (Appendix E.1.2) that Turkey Creek flooding seems to be primarily driven by the most intense 1-2 hours of any given storm, the 24-hour inflow hydrographs used for design are not applicable for comparison to observed storm data.

4.3 Levee Profile Design

The goal of the flood protection system is to protect against the 1% AEP design event with an assurance level of 85% and to meet the criteria set forth by FEMA to accredit the levee for the National Flood Insurance Program (NFIP). Criteria for accreditation can be found in FEMA 44 CFR 65.10.

Development of the levee profile is a multi-stage effort. Levee height is determined through risk-based analysis outlined in EM 1110-2-1619. Hydrologic Engineering Center Flood Damage Reduction Analysis (HEC-FDA) software is used to determine levee height with a required assurance level based on HEC-RAS model outputs. This includes the following sets of data for both the FWOP and with project conditions at each index location:

- Unsteady HDF plan files
- Frequency Functions (Flow Frequency Curve)
- Stage Transform Functions (Rating Curve)
- Lateral Structures (Levee Alignment Shapefile)

Each item is discussed in further detail in Appendix E.1 while Appendix E.3 contains the HEC-FDA input data. The following sections detail the adopted levee profile for 50% design and considerations for future levee profile design.

4.3.1 Adopted Levee Profile

After H&H inputs were created, an estimated levee height was entered at each index location. HEC-FDA performed a monte-carlo analysis, randomly sampling points within the uncertainty bounds of the flow frequency curve and stage-discharge rating curve. HEC-FDA then computed an assurance level based on the resulting expected 1% AEP water surface elevation and the freeboard from the estimated levee height. Levee height iteration was necessary to reach the 85% assurance level threshold at each index location.

The 85% assurance levee heights for each index location were pulled from the HEC-FDA outputs and assembled in tabular and graphical format to create the updated levee profile. It should be noted that the levee profile increased or decreased in some locations compared to the previous post-35% levee profile. To minimize redesign, the post-35% levee profile was kept

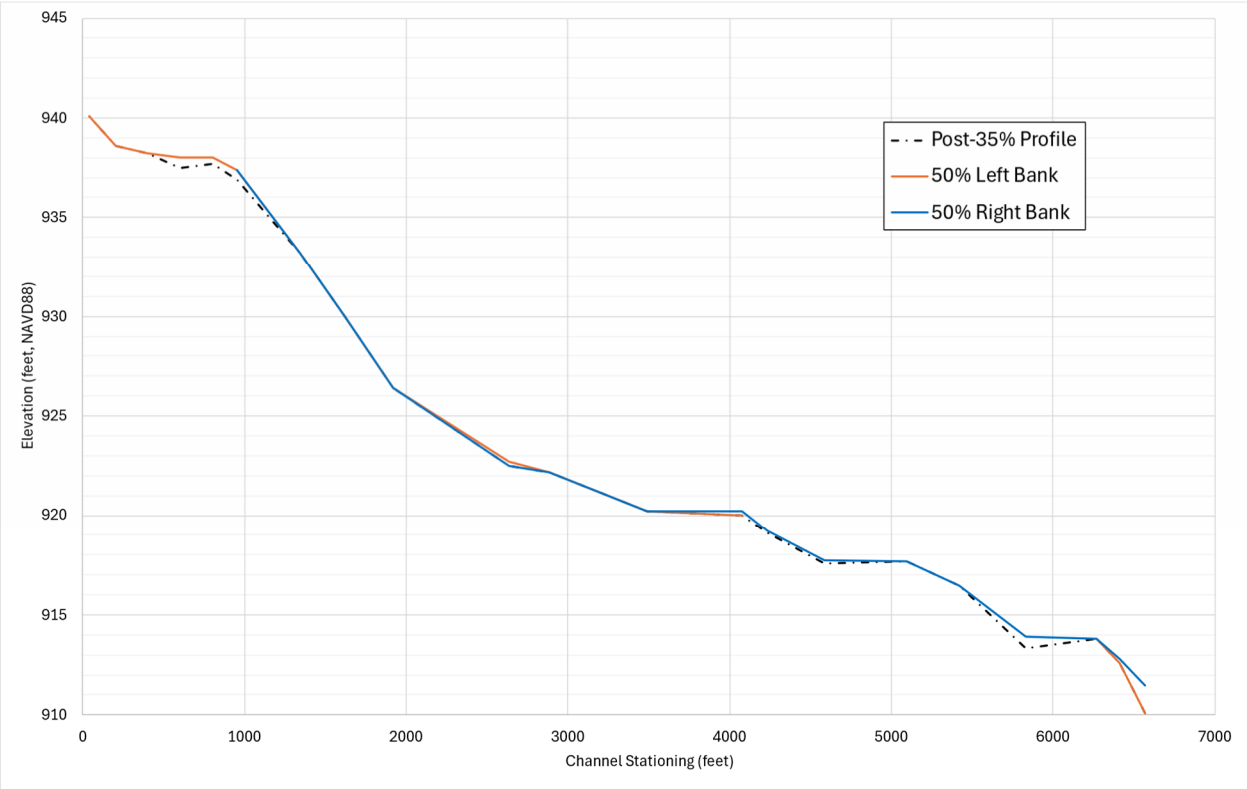
(also documented in Appendix E.3), only modifying it where the 50% HEC-FDA run showed increases in height. The table below provides elevations for the left and right bank systems. Generally, any notable increase in levee height from post-35% design was due to incorporation of superelevation. The post-35% design originally assumed the same levee elevations on both the left and right bank, disregarding superelevation impacts. A comparison of profiles is shown below in Figure 4.4.

Table 4.3. Adopted Levee Profile for 50% Design.

Left Bank*		Right Bank	
STA (feet)	**Elevation (feet)**	**STA (feet)**	**Elevation (feet)**
65+66 W	910.1	65+66 E	911.5
64+14 W	912.6	64+14 E	912.8
62+70 W	913.8	62+70 E	913.8
58+28 W		58+28 E	913.9
54+18 W		54+18 E	916.5
50+96 W		50+96 E	917.7
45+86 W		45+86 E	917.7
42+30 W		42+30 E	919.2
40+76 W	920.0	40+76 E	920.2
34+88 W	920.2	34+88 E	920.2
28+82 W		28+82 E	922.2
26+38 W		26+38 E	922.5
19+20 W	926.4	19+20 E	926.4
16+16 W	930.0	16+16 E	930.0
13+44 W	933.2	13+44 E	933.2
9+54 W	937.4	9+54	937.4
8+00	938.0		
6+00	938.0		
4+00	938.2		

**Note: Left bank is not continuous, and grey boxes represent high ground with no levee/floodwall.*

Figure 4.4. Adopted 50% Levee Profile Comparison with Post-35% Profile.



4.3.2 Future Levee Profile Design

Given that the project is being paused at 50% design, future considerations for changes in levee heights should be documented. The post-35% design left bank included continuous flood protection from project channel station 0+00 to 40+76 (Johnson Drive bridge). The left bank design was then revised prior to 50% submittal to provide protection only to the businesses from channel stations 3+00 to 20+00 and around the church adjacent to Johnson Drive. This change necessitated re-running the assurance computations in HEC-FDA at locations where water surface elevations increased in order to maintain an assurance level of 85%. Index locations where levee height increases are necessary in future design iterations are shown in red in Table 4.4. Figure 4.5 *shows the changes in flood protection alignment from post-35% (blue centerline) to 50% design (red centerline). It should be noted that design changes to the left bank generally resulted in decreases in water surface profile for the 1% AEP event, with the average change in water surface profile along the project extents being -0.17 feet. A comparison of the 50% design levee profile with the potential future profile is shown in*

.

<u>Recommendation</u>

While a levee profile was computed to inform post-50% design, it is recommended that a full re-computation of the levee profile be completed in HEC-FDA should design resume in the future. Project area conditions or design guidance may have changed since the 50% design submittal. Additionally, consideration should be given to refinements of index locations. Additional index locations and/or adjustments to spacing may be necessary to best represent superelevation throughout the system.

Figure 4.5. Changes in Flood Protection Alignment from Post-35% (red) to 50% Design (blue).

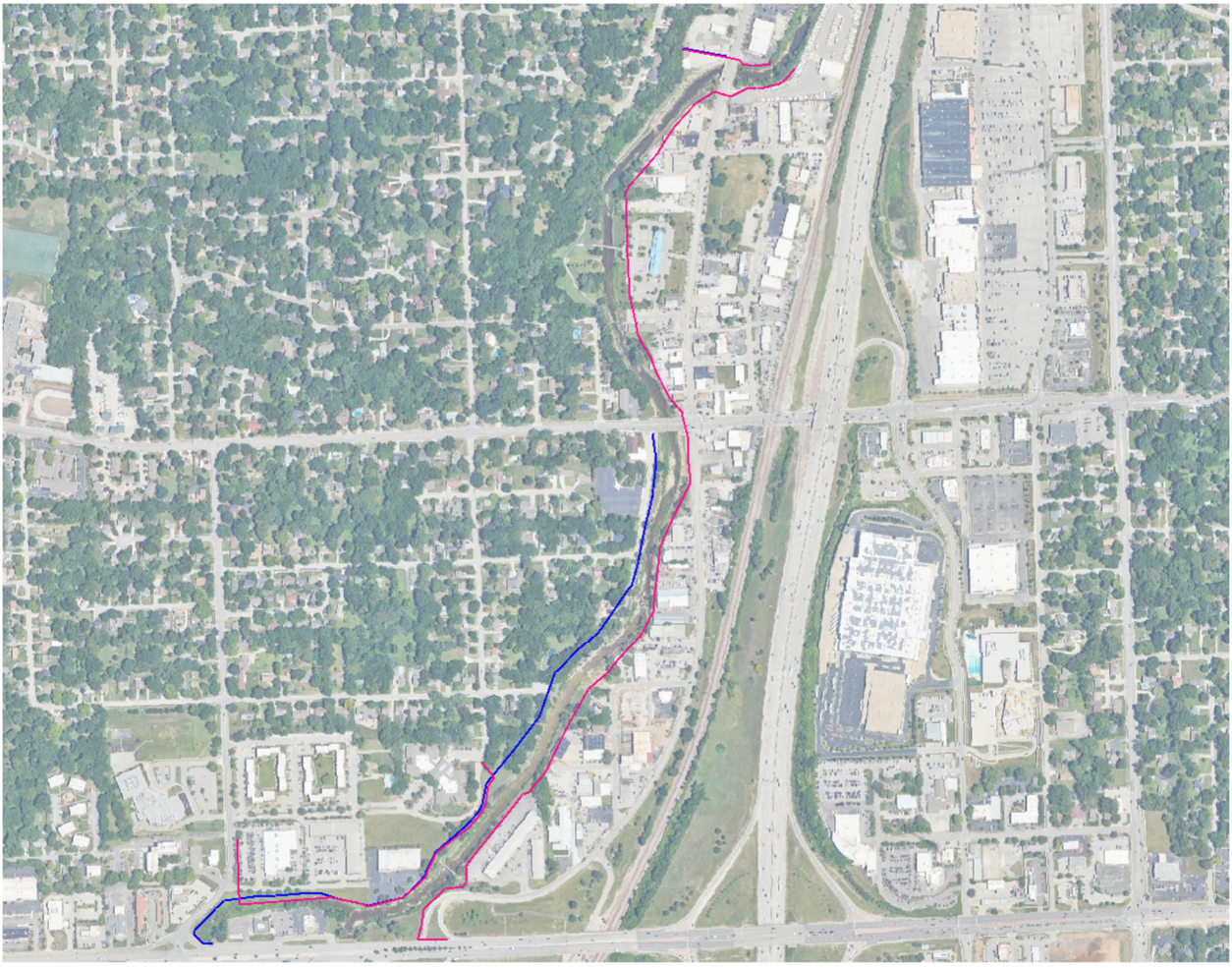


Table 4.4. Future Levee Profile – Increases shown in red, potential decreases not yet evaluated.

Left Bank		Right Bank	
STA (feet)	**Elevation (feet)**	**STA (feet)**	**Elevation (feet)**
65+66 W	910.1	65+66 E	911.5
64+14 W	912.6	64+14 E	912.8
62+70 W	913.8	62+70 E	913.8
58+28 W		58+28 E	913.9
54+18 W		54+18 E	916.5
50+96 W		50+96 E	917.7
45+86 W		45+86 E	917.7
42+30 W		42+30 E	919.2
40+76 W	920.0	40+76 E	920.2

Left Bank		Right Bank	
STA (feet)	**Elevation (feet)**	**STA (feet)**	**Elevation (feet)**
34+88 W	920.2	34+88 E	920.2
28+82 W		28+82 E	922.2
26+38 W		26+38 E	922.5
19+20 W	926.4	19+20 E	926.4
16+16 W	930.0	16+16 E	930.1
13+44 W	933.7	13+44 E	933.2
9+54 W	937.8	9+54	937.4
8+00	938.0		
6+00	938.0		
4+00	938.2		

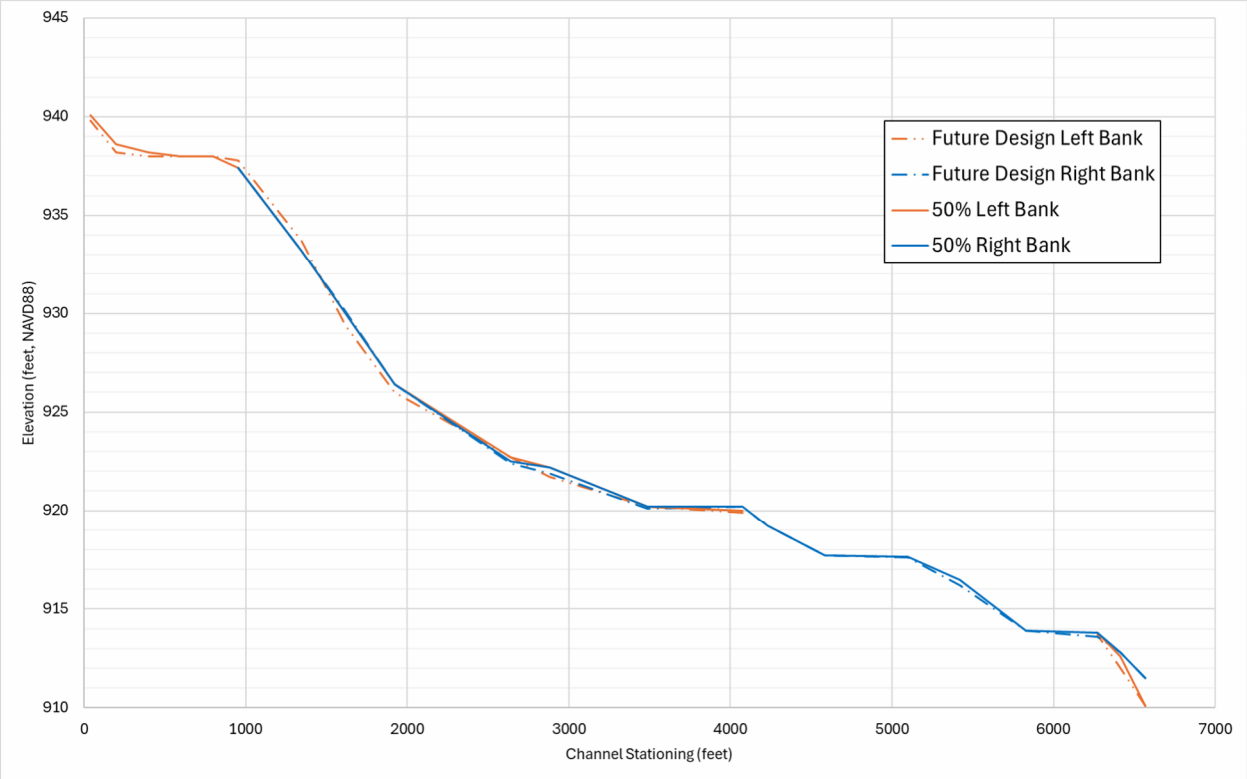


Figure 4.6. 50% vs Future Design Comparison of Levee Profiles.

4.3.2.1 Managed Overtopping

A designated overtopping location was initially considered to follow criteria set out by Engineering and Construction Bulletin (ECB) 2019-8. The guidance generally recommends implementing levee superiority, which means to select a reach of the levee to overtop first where risks to life safety or property are least prevalent. But given the highly urbanized nature of the proposed flood protection system, all locations behind the flood protection system would likely result in immediate flood inundation of private or commercial property. Consideration was given to the location of the detention pond discussed later in Section 5. But any event large enough to result in overtopping of the levee would also result in completely filling the detention pond under gravity denied conditions and would result in overtopping flow spreading out around downtown Merriam.

With this information in mind, it was decided to leave the levee profile as is with the 85% assurance profile which would result in balanced overtopping. Future consideration should be given to implementing a designated overtopping location if the project is resumed and the City of Merriam has a master plan for the redevelopment of the downtown area that might allow a safe and low consequence overtopping location.

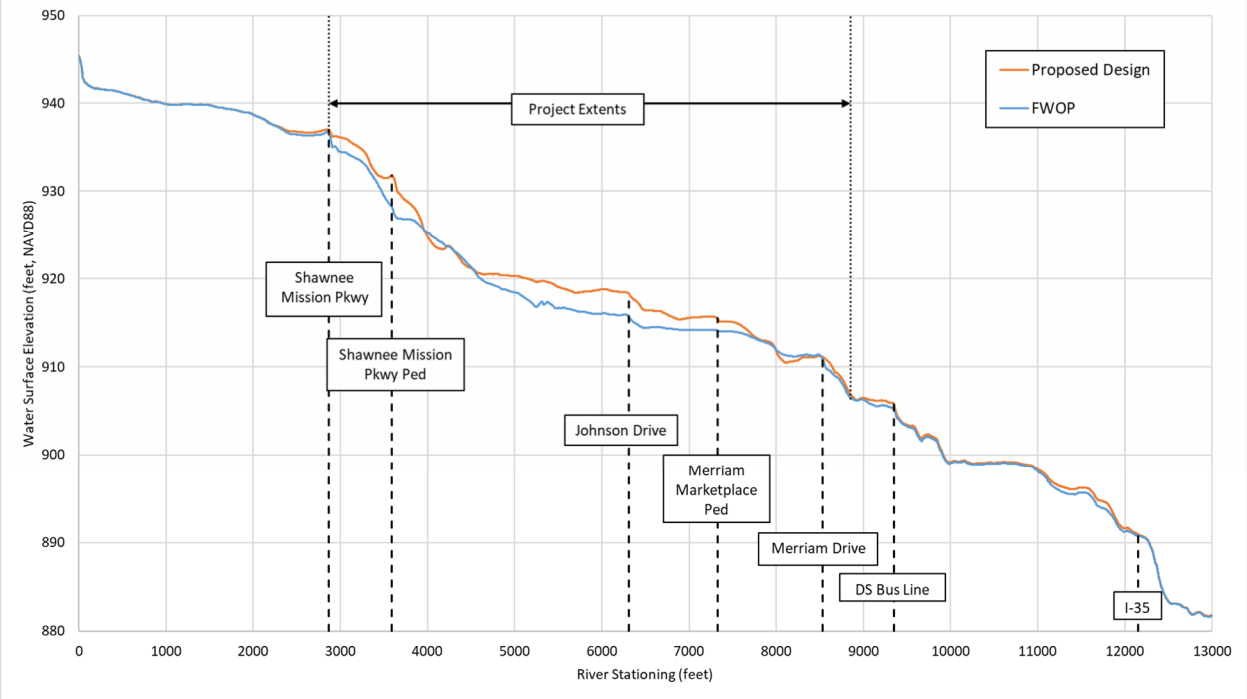
4.4 Comparison of Levee Design to FWOP

The following sections provide a comparison of hydraulic conditions between the 50% design results and the FWOP results. This includes water surface profile for the 1% AEP design event, channel velocities, and water surface elevations. Flow and stage hydrographs are provided at each bridge location in Appendix E.6.

4.4.1 Channel Profile

The hydraulic model was used to evaluate the water surface profile changes from Future Without Project (FWOP) conditions along the project extents at various project features discussed in Section 2. Figure 4.7 compares water surface profiles between FWOP conditions and proposed design. As expected, the water surface profile generally increased within the project area, between Shawnee Mission Parkway and Merriam Drive. The average increase in water surface elevation for this stretch is approximately 1.24 feet with a maximum increase of 3.7 feet on the upstream face of Shawnee Mission Parkway Pedestrian Bridge.

Figure 4.7. Water Surface Profile Comparison between FWOP and Proposed Design.



4.4.2 Channel Velocities

Channel velocities were evaluated within the project extents for the FWOP and 50% design conditions for the 1% AEP design event. Maximum channel velocities for both scenarios generally range from 7-19 ft/s along the project length. Velocities were most extreme near the pedestrian bridge just downstream of Shawnee Mission Parkway, peaking at 18.5ft/s (Channel STA 14+30) for FWOP. Channel velocity at bridges for the 50% design decreased when compared to FWOP as seen in Table 4.5.

Table 4.5. Peak Channel Velocities at Bridges.

Bridge	Location	FWOP Velocity (feet/sec)	50% Design Velocity (feet/sec)
Shawnee Mission Parkway Bridge	Upstream	7.9	7.2
	Downstream	9.4	7.1
Shawnee Mission Parkway Pedestrian Bridge	Upstream	18.4	16.3
	Downstream	18.5	15.9
Johnson Drive Bridge	Upstream	9.9	8.8
	Downstream	9.7	9.2
Merriam Marketplace Bridge	Upstream	8.8	8.3
	Downstream	10.1	9.3
Merriam Drive Bridge	Upstream	11.1	10.9

	Downstream	12.2	10.0

The 50% design velocities increased relative to FWOP most notably in narrow straightaways where flow is fully constrained to the channel and no longer allowed to spread into the floodplain. Figure 4.8 provides a velocity profile plot. Spikes above FWOP are located at channel stations 18+70, 46+80, 58+90, and 66+80. Figure 4.9 and Figure 4.10 provide velocity grids for the FWOP and 50% design, respectively.

Figure 4.8. Channel Velocity Profile Plot for 1% AEP Event – FWOP vs. 50% Design.

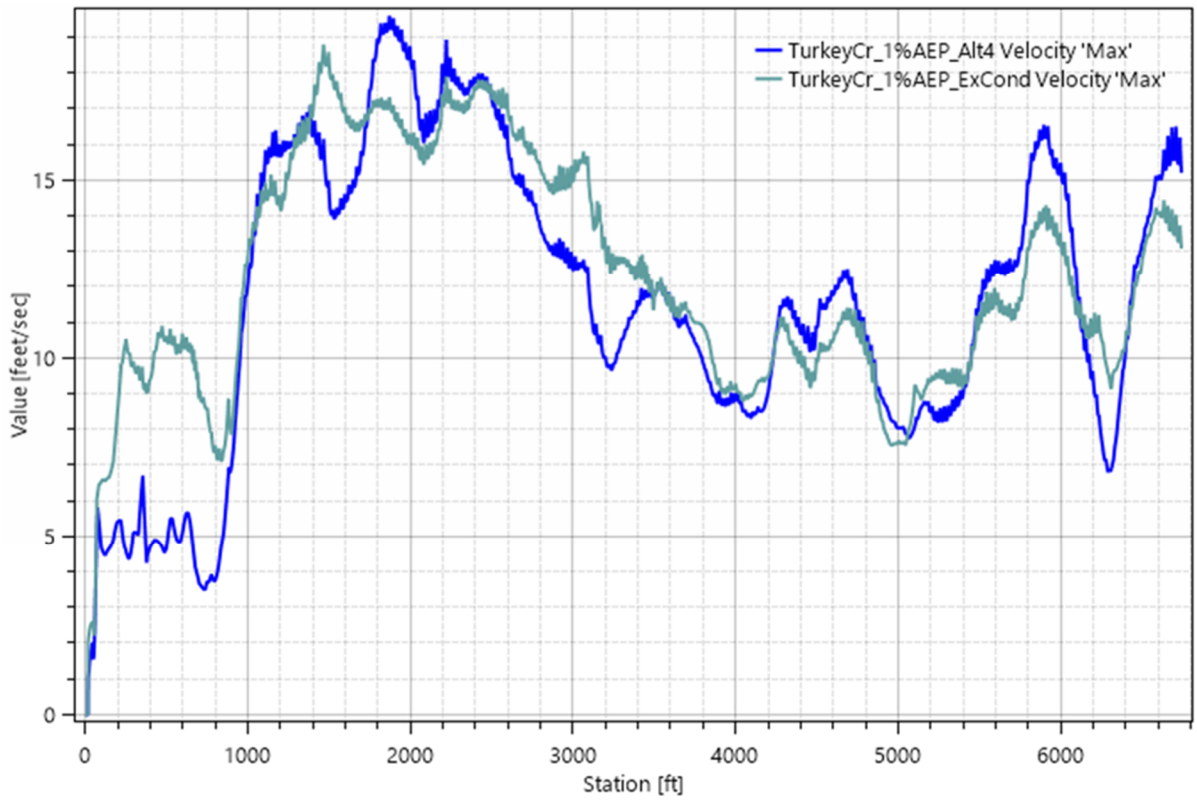


Figure 4.9. FWOP Peak Velocity for 1% AEP Event.

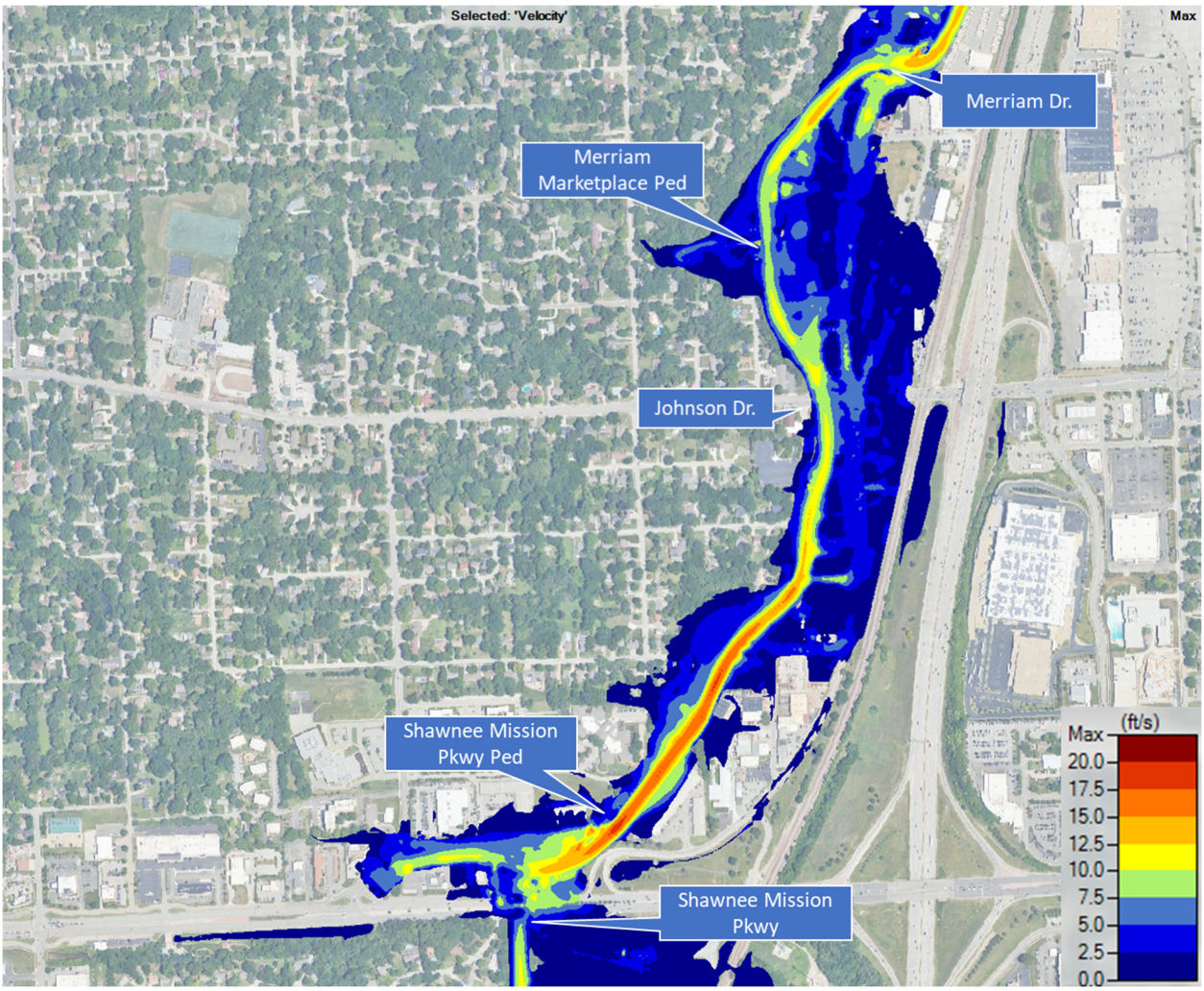


Figure 4.10. 50% Design Peak Velocity for 1% AEP Event.



4.4.3 Bridge Water Surface Elevations

Water surface elevations for the 1% AEP event were taken at the upstream and downstream side of each bridge to better inform the suitability of the existing bridges and inform potential bridge raises and/or replacements. Elevations were also taken from the 2D model for comparison in .

Johnson Drive and Merriam Drive bridges both have proposed replacements which allow increased flow conveyance. The increase in flow conveyance for the Merriam Drive bridge replacement design results in a decrease in water surface elevation on the upstream face. The other bridges still see an increase in water surface elevation from FWOP to the 50% design because of the flood protection system being in place and preventing floodplain conveyance. The upstream face of Shawnee Mission Parkway bridge has a much smaller increase because the bridge culvert is inlet controlled. Any increase in the water surface is because the 1% AEP event is already overtopping Shawnee Mission Parkway in the FWOP condition.

Table 4.6. Water Surface Elevations at Bridges.

Bridge	Location	2D Existing Conditions (feet)	2D 50% Design (feet)
Shawnee Mission Parkway Bridge	Upstream	936.51	936.89
	Downstream	935.14	936.24
Shawnee Mission Parkway Pedestrian Bridge	Upstream	927.96	931.62
	Downstream	926.86	929.92
Johnson Drive Bridge	Upstream	915.85	918.43
	Downstream	915.02	917.62
Merriam Marketplace Bridge	Upstream	914.16	915.64
	Downstream	913.99	915.1
Merriam Drive Bridge	Upstream	911.29	910.95
	Downstream	909.66	910.64
DS Bus Line Bridge	Upstream	905.2	905.73
	Downstream	903.71	903.84

4.4.4 Duration Analysis

A duration analysis was conducted for both the 1% AEP event and an incipient overtopping event to determine the duration of river stage above certain thresholds. These thresholds include channel capacity, levee toe, 1 foot of freeboard, and top of levee. It should be noted that the overtopping event provided an overtopping depth of 0.1 feet to determine the location of incipient overtopping. The location of incipient overtopping is on the upstream side of the Merriam Marketplace pedestrian bridge. The Merriam Marketplace is a open air pavilion with a parking lot surrounding it. This immediate area present a low risk to life safety and property, though any overtopping flow is likely to rapidly spread out and inundate downtown Merriam. Figure 4.11 provides a visual of the two event hydrographs and stage thresholds at the pedestrian bridge.

Table 4.7 summarizes the durations above the previously mentioned thresholds for both the 1% AEP and overtopping events. The results illustrate the flashiness of Turkey Creek, even for events as severe as an overtopping scenario for the proposed levee profile. In that scenario, the levee is loaded for less than 7 hours in total and within one foot of the top of levee elevation for approximately 3 hours.

Figure 4.11. Upstream Side of Merriam Marketplace Pedestrian Bridge 1% Annual Exceedance Probability and Overtopping Hydrographs.

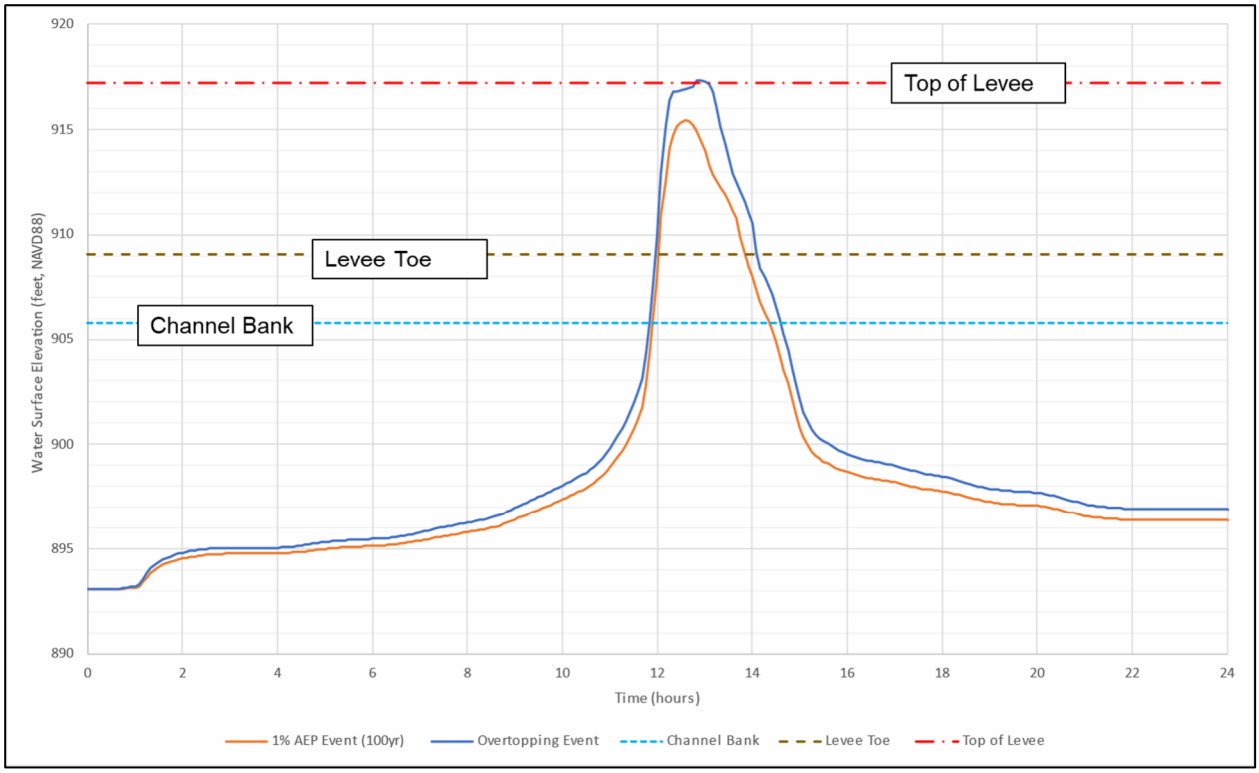


Table 4.7. Duration Analysis at Upstream Side of Merriam Marketplace Pedestrian Bridge.

Stage Threshold	Elevation (feet)	Duration Exceeded (hours) for 1% AEP Event	Duration Exceeded (hours) for Overtopping Event
Channel Bank	905.75	2.5	2.75
Levee Toe	909.05	1.75	2
1 foot of freeboard	916.22	0	1.25
Top of Levee	917.22	0	0.25

4.4.5 Stage Exceedance Analysis

An annual stage exceedance analysis was conducted using Statistical Software Package (HEC-SSP) to find the percent of time Turkey Creek exceeds a given stage (WSE) at the Turkey Creek at Johnson Drive Stormwatch river gage. Stage data is available back to 2001 at this gage location but data prior to 2014 is erratic with season-long shifts in normal flow stage and numerous erroneous gage readings. Therefore, stage data from 2014 onwards, plotted in Figure 4.12 and summarized in , was used for the analysis. The approximate levee toe stage at the right bank upstream side of Johnson Drive (location of Stormwatch gage) is shown, along with

the top of levee stage. According to the analysis, the levee toe would be loaded less than 0.1 percent annually at this location.

Figure 4.12. Annual Stage Exceedance Plot at Turkey Creek at Johnson Drive (Stormwatch TU04).

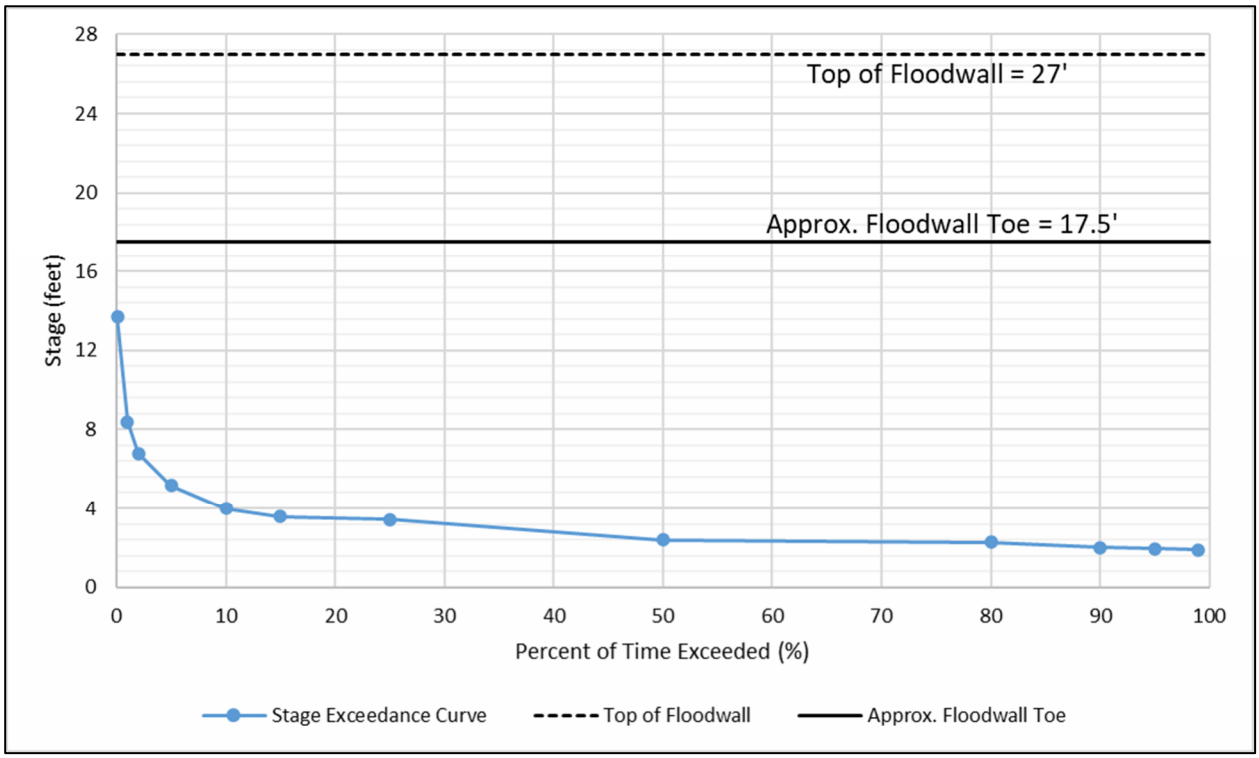


Table 4.8. Annual Stage Exceedance Summary at Turkey Creek at Johnson Drive (StormWatch TU04)

Percent of Time Exceeded	Stage (feet)	Water Surface Elevation (feet)
99.0	1.89	894.89
95.0	1.93	894.93
90.0	2.01	895.01
80.0	2.28	895.28
50.0	2.38	895.38
25.0	3.41	896.41
15.0	3.58	896.58
10.0	3.98	896.98
5.0	5.17	898.17
2.0	6.8	899.8

1.0	8.38	901.38
0.1	13.7	906.7

4.4.6 FEMA Flood Profile Comparison

This analysis is to be completed in a future design submittal as it is dependent on furthering of bridge design and finalization of levee configuration and alignment.

4.5 Riprap Sizing

Riprap sizing was determined by using the HEC-RAS hydraulic design tool which utilizes guidance outlined in EM 1110-2-1601. Maximum riprap diameter (D_{100}) is largely dependent on flow velocities and radius of curvature, as well as other factors. Required riprap sizes were determined for both the left and right banks to act as levee slope protection and toe protection for floodwalls. Riprap was sized at any location where notable changes in velocity occurred.

4.5.1 Parameters

The riprap sizing tool requires several input parameters based on the hydraulic results of the 1% AEP design event and the general geometry/configuration of the terrain and flood protection system. Table 4.9 below summarizes the parameters used for selecting riprap gradations and sizes. Radius of curvature and side slope angles were computed at each design cross-section location. EM 1110-2-1601 states the minimum factor of safety for riprap design is 1.1. But given that this is a life-safety project, a factor of safety of 1.5 was deemed more appropriate. Based on internal discussions and referencing other projects within the metro area, the typical stone unit weight can be sourced locally is 155 lb/ft^3. An angle of repose of 40 degrees was used which is stated as the typical value in EM 1110-2-1601.

Average channel velocity, hydraulic depth, and channel top width were all computed for both the upstream reference cross-section and each design cross-section. It should be noted that to incorporate higher confidence in maximum riprap diameter, the left or right overbank (depending on which is being evaluated) velocity was used instead of the average channel velocities. Given the higher fidelity of 2D modeling, spatially varied velocities across a cross section can be obtained as shown in Figure 4.13. Using this example, the velocity used in the riprap tool would be 12 ft/s for the right bank levee.

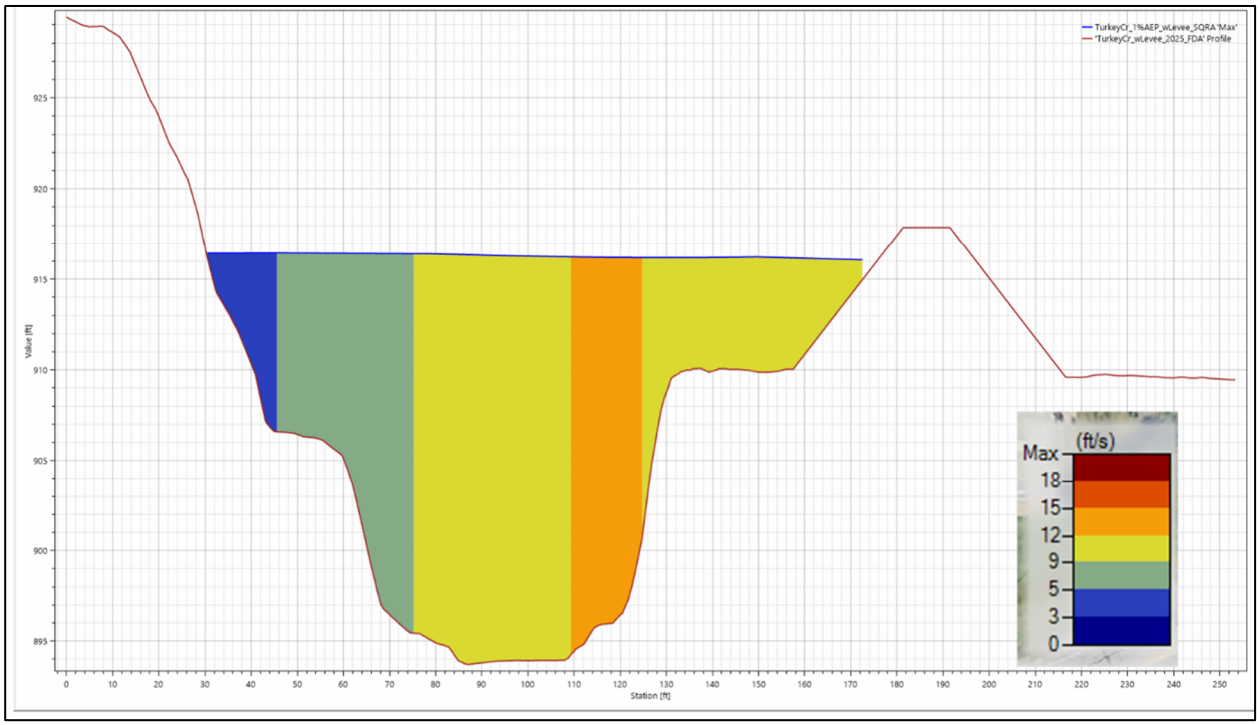
Riprap sizes can be computed using two methodologies in the HEC-RAS tool: The USACE method and the Isbash method. Default parameters were chosen for both, aside from channel alignment which was dependent on design cross-section location. Specific parameters for each design cross section are detailed in Appendix E.4.

Table 4.9. HEC-RAS Riprap Design Tool Parameters.

Input Data	
Parameter	**Value**
Radius of Curvature (ft)	*Varied*
Side Slope Angle (deg)	*Varied*
Safety Factor	1.5
Unit Weight (lb/ft^3)	155
Angle of Repose (deg)	40

Upstream Reference Cross Section	
Velocity Average (ft/s)	9.7
Hydraulic Depth (ft)	10.3
Width (ft)	80
Design Cross Section	
Velocity Average (ft/s)	*Varied*
Hydraulic Depth (ft)	*Varied*
Width (ft)	*Varied*
USACE Parameters	
Channel Type	Natural Channel
Rock Type	Angular Rock
Site Alignment	*Varied*
K1 Method	Graphical
Isbash Parameters	
Channel Alignment	*Varied*
Turbulence Value	1.2

Figure 4.13. Spatially Varied Velocities in Design Cross-Section.



4.5.2 Maximum Riprap Diameters

A total of six unique maximum riprap diameters were determined from the HEC-RAS tool and then condensed down to three to reduce design complexity. Table 4.10 and Table 4.11 provide the maximum riprap sizes for the left and right banks, respectively. The largest riprap required has a D_{100} of 42 inches. This is located downstream of the Merriam Drive bridge on the right bank which is a sharp outside bend. A 36-inch diameter was selected for this area due to the lack of local availability of 42-inch riprap. A D_{100} of 36 inches was deemed suitable as it retained a factor of safety of 1.3 which is still above the minimum factor of safety (1.1) specified in EM 1110-2-1601. Additionally, this section of the flood protection system is lower risk given the riprap is located at the downstream end of the project. Any floodwall failure at this location would result in backwater flooding which would act as a bathtub until river stages drop and water drains out of the interior through the breach. The bathtub effect occurs when backwater enters the downstream portion of a protection system and slowly fills rather than conveying through the leveed area. This presents less life safety risk than breaches that occur upstream and have hydraulic head to flow through the leveed area.

Numerous locations only required a D_{100} of 15 inches, but EM 1110-2-1601 specifies a minimum of ~18 inches (W_{100} of 80 lbs.) in urban areas to prevent theft and vandalism. So, all instances of 15-inch riprap were increased to 21 inches. 24- and 30-inch riprap was changed to 33-inch riprap to simplify design as previously mentioned.

Table 4.10. Maximum Riprap Sizing for Left Bank.

Approx. UTC STA		Approx. Left Bank STA		Length (ft)	Initial D100 (in)	Recommended D100 (in)
Start	End	Start	End			
0+00	9+00	13+00	22+38	938	21	21
9+00	14+00	22+38	26+50	412	33	33
15+25	19+00	28+20	31+60	340	15	21
21+00	25+00	34+00	37+60	360	15	21
33+00	36+00	45+40	48+45	305	15	21
63+00	65+75	78+40	80+76	236	24	33

Table 4.11. Maximum Riprap Sizing for Right Bank.

Approx. UTC STA		Approx. Right Bank STA		Length (ft)	Initial D100 (in)	Recommended D100 (in)
Start	End	Start	End			
15+25	19+00	6+80	10+47	367	15	21
19+00	26+00	10+47	17+60	713	15	21
30+00	33+00	21+40	24+75	335	30	33
33+00	48+00	24+75	40+00	1525	21	21

53+00	57+00	44+80	48+12	412	21	21
57+00	60+00	48+12	51+30	318	33	33
60+00	64+00	51+30	54+60	320	21	21
64+00	65+75	54+60	57+78	318	42	36

4.5.3 Bridge Scour Analysis

A bridge scour analysis was conducted on the two pedestrian bridges within the study area. The Shawnee Mission Parkway pedestrian bridge and the Merriam Marketplace pedestrian bridge. Johnson Drive and Merriam Drive bridge designs are being contracted out and bridge scour analysis will be performed by the contractor.

The bridge scour analysis is to be conducted in accordance with the Federal Highway Administration (FHA) Hydrologic Engineering Circular No. 18. This document provides guidelines for evaluating scour at bridges. Cross-sectional channel info and hydraulic data was pulled from the HEC-RAS model for computing scour along bridge abutments only, given that neither pedestrian bridge had piers. Appendix E.5 contains the hydraulic input data used for the analysis. Section 9.6 of the DDR will contain the full bridge scour analysis in a future design submittal. Should riprap be required to mitigate any expected bridge abutment scour, the sizing will be documented in this section.

4.6 Channel Modifications

Modifications were made to the channel and overbanks along the tributary at the upstream portion of the project. After the decision was made to remove the flood protection from around the two existing businesses and instead buy them out The channel alignment would be shifted to the south and the left (north) bank would be built out to place a floodwall. This was done to avoid the need for utility relocations that run along W. 62nd Terrace. This provides minor additional storage and comes with the added benefit of creating a local borrow source. Figure 4.14 provides a visual of the benching, and Figure 4.15 shows a cross-section of new channel alignment and minor benching.

The realigned channel is to have a 55-foot bottom width which is generally slightly wider than the existing channel bottom width. The channel banks will have 3H:1V bank slopes for stability. The channel profile will be generally maintained to facilitate continued drainage from the upstream culverts and to still terminate at the downstream concrete encase utility crossing at the confluence with the Turkey Creek channel.

Figure 4.14. Tributary Benching

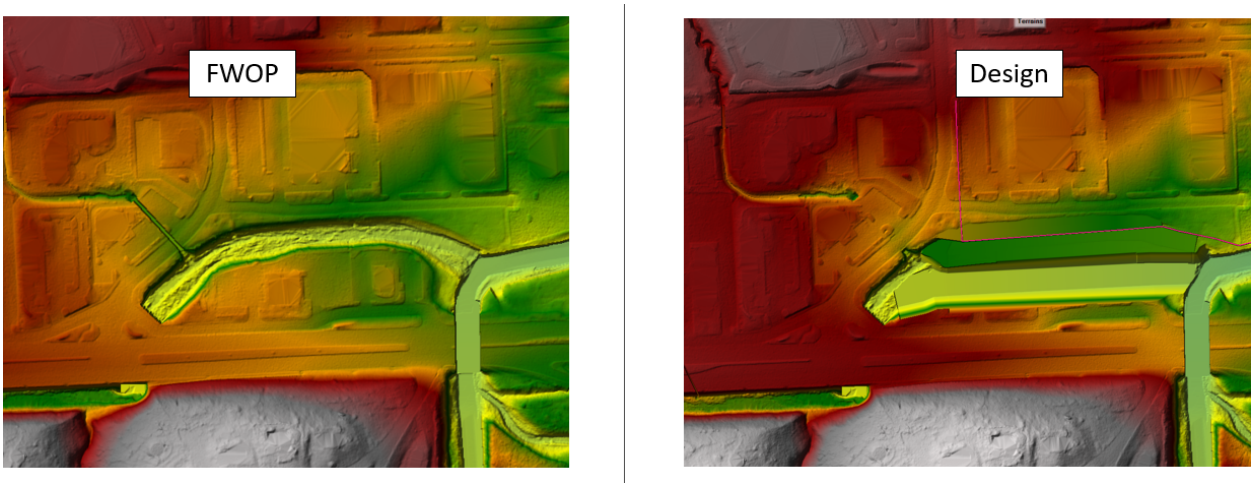
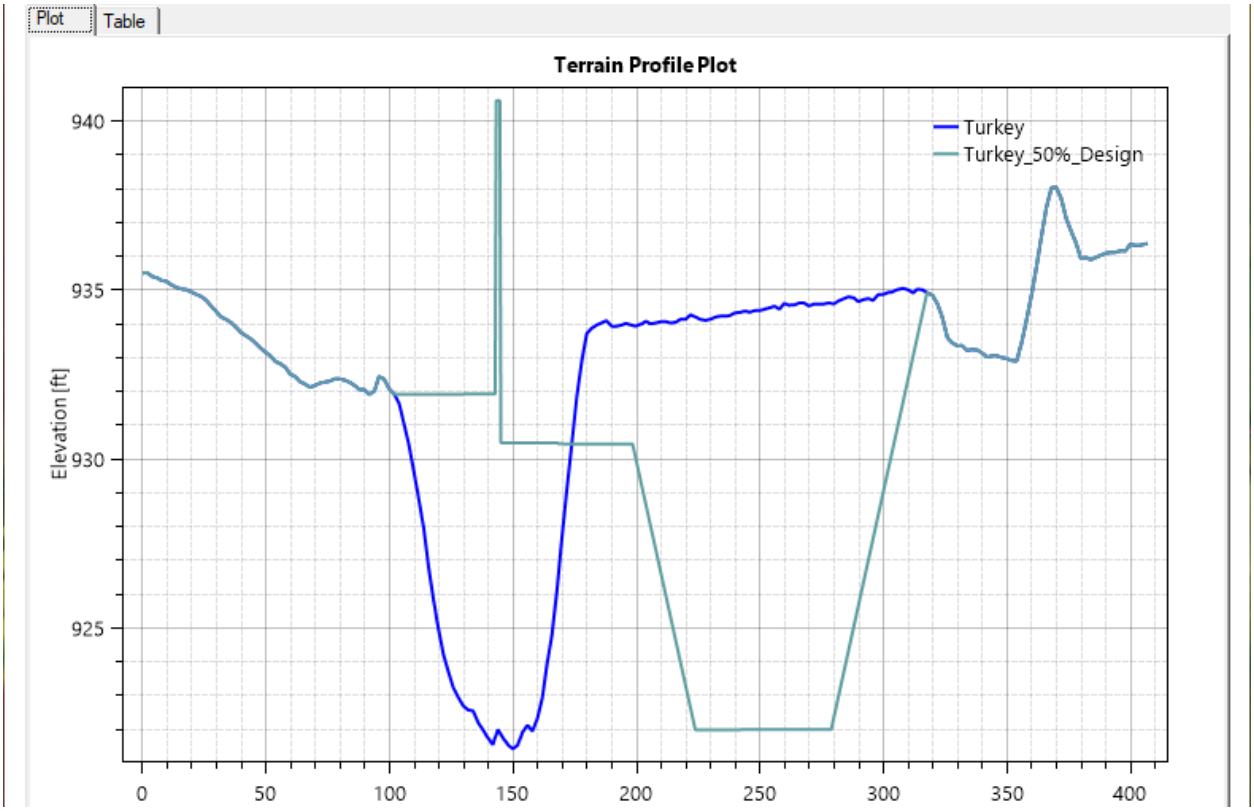


Figure 4.15. Cross Section of Tributary Benching



4.7 Previous Studies

To be added

4.8 References

Hershfield, D. M. (1961). *Rainfall Frequency Atlas of the United States, Technical Paper No. 40*. Weather Bureau, U.S. Department of Commerce, Washington, D.C.

Larkin. (2005). *Northeast Johnson County Watershed Study.* Larking Group Inc., Kansas City, MO.

NOAA. (2013). *Precipitation-Frequency Atlas of the United States, NOAA Atlas 14, Volume 8, Version 2: Southeastern States*. National Oceanic and Atmospheric Administration, National Weather Service, Silver Spring, MD.

USACE. (2025). *Turkey Creek and Indian Creek July 2025 Flood High Water Marks.* U.S. Army Corps of Engineers, Kansas City District: P:\ED-HH\Turkey Creek\2025 Flood Event

USACE. (2024). *HEC-RAS 2D User's Manual, Latest.* U.S. Army Corps of Engineers, Hydrologic Engineering Center: https://www.hec.usace.army.mil/confluence/rasdocs/r2dum/latest

USACE. (2024). *HEC-RAS Hydraulic Reference Manual, Latest.* U.S. Army Corps of Engineers, Hydrologic Engineering Center: https://www.hec.usace.army.mil/confluence/rasdocs/ras1dtechref/latest

USACE. (2018). *Turkey Creek Summer 2017 Flood Events.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (2002). *Hydraulic Design of Flood Control Channels, EM 1110-2-1601.* U.S. Army Corps of Engineers, Washington, D.C.

USACE. (1996). *Risk-Based Analysis for Flood Damage Reduction Studies, EM 1110-2-1619*. U.S. Army Corps of Engineers, Washington, D.C.

USGS. (2005). *Land Subsidence in the United States, Fact Sheet 2005-3080*. U.S. Geological Survey.

Stantec. (2024). *Turkey Creek and Brush Creek 2D Watershed Hydrologic and Hydraulics Report, DRAFT.* Stantec, Overland Park, KS

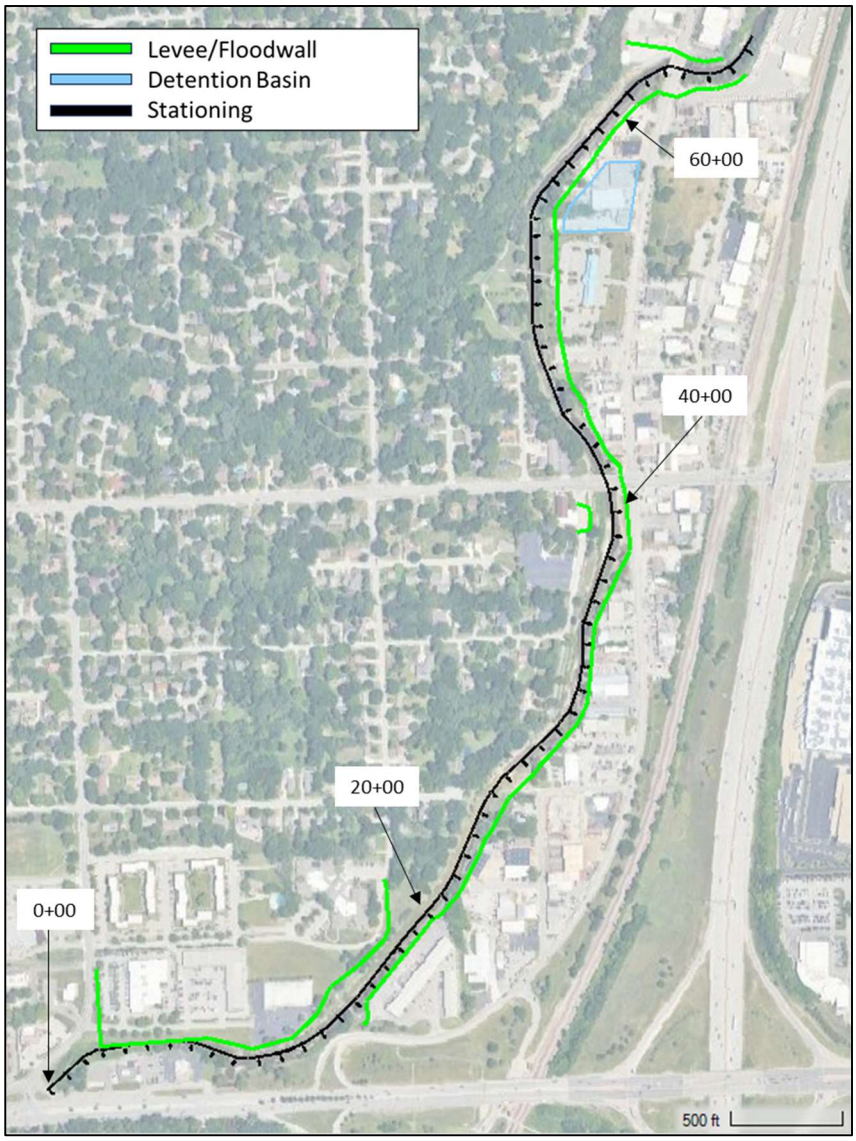
5.0 INTERIOR DRAINAGE

This project includes improvements to interior drainage infrastructure. The areas of specific improvements are provided below:

- Drainage Structures
    - Left Bank—Station (Sta.) 3+00 to 20+00, 39+00 to 40+50, and 62+00 to 66+00
    - Right Bank—Sta. 14+50 to 66+25
- Detention Basin—Sta. 54+00.

The proposed levee alignment and detention basin are displayed in Figure 5.1.

Figure 5.1. Project Features and Stationing



5.1 Design Methodology

This section addresses the general description of the design methodology. This section provides standards, criteria, and available data.

5.1.1 Frequency and Criteria

Engineering Manual (EM) 1110-2-1413, *Hydrologic Analysis of Interior Areas*, dated 24 August 2018, EM 1110-2-1415, *Hydrologic Frequency Analysis*, dated 5 March 1993, and EM 1110-2-2902, *Conduits Pipes and Culverts Associated with Dams and Levee Systems,* dated 31 December 2020 represent the applicable design criteria for the interior drainage analysis.

American Public Works Administration Specification Section 5600 (2011 Version), *Storm Drainage Systems & Facilities* (APWA), is the local design criteria for the Kansas City metropolitan area, including both Johnson County and the City of Merriam in Kansas. APWA states that property not reserved or designed for conveying storm water shall have a system to convey the 10-year (10% AEP) rainfall event when the total drainage area is two acres or more.

For modifications to existing storm drainage crossings under the proposed levee/floodwall, the proposed drainage crossings were sized to convey flows equal to or greater than the capacity of the existing system. The proposed drainage crossings were sized to minimize the induced impacts for a 1% AEP rainfall event where possible. These proposed drainage structures were also sized to a minimum of 24 inches as a part of inspection best practices.

Due to the scope and size of this project, the APWA criteria for detention basin sizing is not applicable. APWA requires the detention basin to be sized for the 1% AEP, 24-hour design storm for development projects. A development project is defined by APWA as, “Any activity, including subdivision, that alters the surface of the land to create additional impervious surfaces, including, but not limited to, pavement, buildings, and structures.” This is not applicable to this scope. The goal of the interior drainage is to focus on what best serves the site and has the least flood impact, while being cost effective. The detention basin storage will be determined by the 10% AEP, 24-hour precipitation event to match the current storm sewer infrastructure in place. The non-federal sponsor concurs with this assessment.

All elevations in this chapter are referenced to the North American Vertical Datum of 1988 (NAVD88) in feet. North American Datum (NAD) 1983 State Plane Coordinate System (SPCS) for Kansas North was used as the spatial datum.

5.1.2 Enclosed Pipe Systems

Design guidance for pipe velocities, depths of cover, and invert offsets were obtained from the APWA standard. The guidance for closed pipe systems was as follows:

- Velocity within the system shall be between 3 and 20 feet per second.
- Minimum depth of cover shall be 18 inches.
- Pipe crowns entering a drainage structure shall be at or above the crown of the pipe exiting from the structure and provide a minimum fall of the invert in the structure of 0.2 feet.

Manning’s n values were determined from the HEC-RAS Hydraulic Reference Manual – Table 6-1 Manning's "n" for Closed Conduits Flowing Partly Full as shown in Table 5.4.

The diameter and slope of new pipes was selected to meet flow and velocity targets for that span’s cumulative flow rate and to minimize diameter changes along the longitudinal run.

5.1.3 Construction of Drainage Structures

Current design criteria for Turkey Creek includes installation of outfall structures near the normal flow line of the creek with a flap gate. Storm system crossings of levees and floodwalls will not have double closures as required by EM 1110-2-2705. This is because the risk to life safety is negligible and the likelihood of a flap gate getting pinned fully open is low as identified in the Semi Quantitative Risk Assessment (SQRA). Even if a flap gate is pinned open, the backflow that gets through is not substantial. These crossings will not have the landside manholes and closures and will only have passive closure devices at the outfall to Turkey Creek as documented in the Design Guidance Deviation for Secondary Closure for Storm Drainage Pipes Memorandum. Life safety and backflow sensitivity testing are documented in the 2025 SQRA.

Existing storm water pipes through the levee or floodwall alignment that are not a part of the design are to be removed or abandoned and grout filled. Installation of a headwall and flap gates at the existing triple 7-foot by 7-foot reinforced concrete box (RCB) drainage structure at Sta. 38+60 is included as part of the drainage structure design.

These design criteria are maintained to ensure protection of the leveed area is passive, as Turkey Creek is extremely flashy, and the sponsor cannot ensure personnel will be on site in time to close gate wells at all times of the day. See Figure 5.2 for the intensity of historic precipitation events and Section 5.3 for more detailed information on historic events.

5.1.4 Data Available

Bridges, culverts, and channel cross-sections were taken in the field by Stockwell Engineers Inc. between May and July 2023. All structures along the study streams were surveyed and incorporated into the models as 2D connections.

LiDAR data was provided by Johnson County, Kansas, through the Automated Information Mapping System (AIMS) for Turkey Creek and Brush Creek (2-foot resolution). The LiDAR acquired from AIMS was collected in 2020. For the portion of the Turkey Creek watershed located in Wyandotte County, Kansas, supplemental LiDAR was downloaded from Kansas' Data Access & Support Center (DASC) at 1-meter resolution. The LiDAR acquired from DASC was collected in 2018. The DASC LiDAR was reprocessed to match the resolution of the Johnson County LiDAR and were mosaicked together. LiDAR was used to obtain ground elevations for areas outside the survey limits and to draw approximate drainage areas.

The City of Merriam, Kansas, provided a shapefile of existing stormwater pipes for the approximate storm sewer location for the city. AIMS was used to fill in any gaps in the existing pipe network.

The high-water mark collection results for the July 17th, 2025, precipitation event are stored in Appendix E.1.

Structural flooding elevations (Table 5.11) were aggregated between multiple sources. These included a survey from Stockwell Engineers Inc. which contained elevations for nine structures of interest. The rest of the flooding elevations were estimated from AIMS and the previously mentioned LiDAR.

5.1.5 Rainfall Depth Analysis

Johnson County, Kansas, funded Dr. Bruce McEnroe, a technical expert on the hydrology within Johnson County, to evaluate the appropriate precipitation and temporal distribution for the watershed studies within the county. The Rainfall for Flood Studies: Indian and Tomahawk Creeks Report (Appendix M.1), completed in November 2023, also evaluated the appropriate

rainfall distribution for the flood frequency watershed analysis. Based on Dr. McEnroe's evaluation, a 12-hour nested hyetograph with a 3-hour peak was determined as an appropriate temporal distribution for Johnson County, Kansas.

Johnson County concluded that a single rainfall depth derived from NOAA Atlas 14 be applied to all watershed studies in the county. Those depths were derived from KDOT's rainfall tables, which are averaged by county and summarized in Table 5.1. These annual maximum rainfall depths range from 0.03 inches higher for the 50% AEP event to 0.16 inches higher for the 0.2% AEP event when compared to Atlas 14 depths.

Table 5.1. 12-hour Duration Rainfall Depths for Johnson County, Kansas.

	Annual Exceedance Probability Event						
	50%	10%	4%	2%	1%	1%+*	0.2%
12-hour Rainfall Depth (inches)	3.17	4.74	5.82	6.71	7.65	8.56	10.04

*1%+ AEP is the high end of the 90 percent confidence interval for the 0.01 AEP event.

A hydrologic analysis for the Turkey Creek watershed was completed using HEC-1 in May 2005. These HEC-1 outputs provide 24-hour duration rainfall events. The HEC-1 output files provide the cumulative rainfall percentage and the total precipitation over the 24-hour period. Based on those provided inputs, rainfall depths can be determined as seen in Table 5.2.

Table 5.2. 24-hour Duration Rainfall Depths for Johnson County, Kansas

	Annual Exceedance Probability Event						
	50%	10%	4%	2%	1%	0.5%	0.2%
24-hour Rainfall Depth (inches)	3.67	5.52	6.76	7.76	8.8	9.9	11.4

The HEC-1 24-hour duration rainfall depths was used for this analysis. This was to better meet standards for the detention basin sizing and match the analysis in Part 04–Hydrology and Hydraulics.

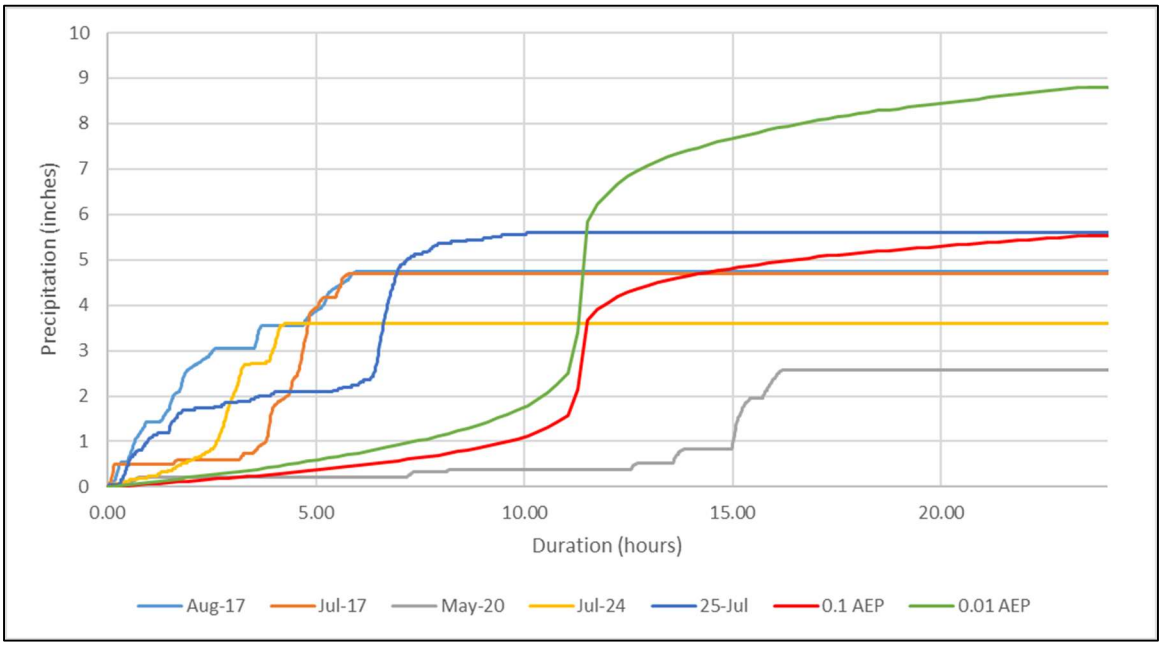
Real precipitation events were also included in the interior drainage hydraulic modelling. These were used to provide historic examples of how long it takes for the water to rise out of existing banks as well as more comparisons between existing and proposed conditions. These include events in July 2017, August 2017, May 2020, July 2024, and July 2025 and can be seen in Figure 5.2. These events cover a range of 2-to-50-year return periods as shown in Table 5.3. These events were chosen due to their precipitation intensity and their recency. The City of Overland Park's StormWatch gages were used to develop the precipitation input for these events, utilizing the StormWatch precipitation gages and incremental rainfall for periods of interest.

Table 5.3. Historic Storm Event Information

Storm Event	**Total Precipitation (in)**	**Precipitation Duration (hrs)**	**Return period (years)**

July 2017	4.7	5.8	10 to 25
August 2017	4.8	6.6	10 to 25
May 2020	2.6	3.7	2 to 5
July 2024	3.6	4.2	5 to 10
July 2025	5.6	9.8	25 to 50

Figure 5.2. Cumulative Precipitation for Theoretical and Real Precipitation Events – TU03 Gage.



5.2 Interior Drainage Hydraulic Modeling

This section addresses the general description of the existing and proposed conditions and their respective hydraulic modeling. Hydrologic Engineering Center River Analysis Software (HEC-RAS) version 6.7 beta 5 was used for this analysis.

The hydrologic calculations were developed directly in the HEC-RAS model using a rain-on-grid, 2D approach. For more information on hydrologic analysis, reference Appendix E.7.

The subbasin modeling was developed using the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) version 4.11. The HEC-HMS model included the complete watershed contributing to the upper portion of Turkey Creek. LiDAR data, referenced in section

5.1.4, provided the existing terrain of the watershed. Break points were placed in HEC-HMS to represent gravity drain locations. Using HEC-HMS, streams were identified and subbasins were initially delineated. Subbasins that flowed into the reach of interest were selected and exported to the HEC-RAS model. This model was used exclusively for subbasin delineation and was only interested in the surface interactions and did not account for subbasin delineation with respect to pipe connectivity between watersheds.

The Stantec Hydrologic Engineering Center River Analysis System (HEC-RAS) model was pulled into version 6.7 beta 5. The rain-on-grid 2D component used precipitation data to determine the flows and water surface elevations for the Turkey Creek watershed. Subbasin numbering corresponds to the drainage structure number. Subbasins were adjusted using HEC-RAS particle tracing and existing stormwater drainage. Figure 5.3 shows the subbasins within the entire Upper Turkey Creek watershed and Figure 5.4 show the subbasins within the project area. The Upper Turkey Creek watershed drains approximately 7,850 acres to the downstream end of the project.

Due to the flashy nature of Turkey Creek, the modeling strategy sought to keep the coincident timing of the interior drainage systems tethered to the stages in Turkey Creek. Therefore, Stantec's rain-on-grid, 2D model was selected as the base existing conditions model. The rain-on-grid in Stantec's model applies rainfall across the entire Upper Turkey Creek watershed (pink polygon in Figure 5.3). For theoretical annual exceedance probability events, one precipitation hyetograph is applied for the entire watershed. For historical events, a group of gage locations used the Thiessen polygon method to implement recorded precipitation hyetographs across the watershed. See Appendix M.2 for hyetograph data.

Stantec's model includes major bridges and culverts. Stantec's model does not include a modeled stormwater system. See Appendix E.7 for more information regarding Stantec's base model.

The hydraulic calculations were developed directly in the HEC-RAS model. The base of the hydraulic model was the HEC-RAS model provided by Stantec Inc. For more information on the base hydraulic model, reference Stantec's July 2024 Turkey Creek and Brush Creek 2D Watershed Hydrologic and Hydraulics Report. Further hydraulic developments were made in the Stantec Base model.

The bridge modeling methods in the interior design hydraulic model match the bridges described in Part 04 – Hydrology and Hydraulics.

Circular drainage structures were modelled as pipe networks with no negative flow flaps and other shapes were modeled as 2D connections featuring culverts, with no negative flow flaps and using the normal 2D Equation Domain, so weir coefficients are not impactful.

Figure 5.3 Upper Turkey Creek Watershed



Figure 5.4. Turkey Creek Interior Drainage Project Area



5.2.1 Calibration & Validation of Base Model

Stantec's rain-on-grid, 2D model was calibrated using 2 historic storm events in the watershed, August 22, 2017, and May 28, 2020. The City of Overland Park's StormWatch gages were used to develop the precipitation input for the calibration events, utilizing the StormWatch precipitation gages and incremental rainfall for periods of interest. These gages were also used to calibrate the model based on the recorded stages during the historic flood events throughout the watershed. See the locations used for calibration in Figure 5.5.

Figure 5.5. Stormwater Gage Locations for Calibration.



The final calibration parameters include Manning's n calibration regions along Turkey Creek to vary the roughness values between the deeper flow in the channel and the shallow flow in the uplands. The final abstraction ratio for the basin was set to 0.1. The base curve numbers were derived using the NRCS TR-55 method and not altered during calibration. Channel roughness along Turkey Creek ranged from 0.025 to 0.035 with the lower n values along the stretch of Turkey Creek along Johnson Drive and Antioch Road. No roughness modifications were made from the Stantec rain on grid model. The upland n values were set to represent shallow overland flow Manning's n values. Table 5.4 summarizes the calibrated Manning's n- values by land cover.

Table 5.4. Calibrated Manning's n-value by Land Cover.

Description	Base n-value	Calibrated Floodplain n-value	Calibrated Upland n-value
Trees	0.085	0.060	0.140
Natural Channel	0.035	0.035	0.035
Concrete Channel	0.015	0.015	0.015
Pavement	0.020	0.015	0.020
Grass	0.030	0.030	0.140
Buildings	1	1	1
Barren Land	0.025	0.025	0.045
Open Water	0.025	0.025	0.030

The final calibrated results from the August 2017 and May 2020 historical events are summarized in Table 5.5 and Table 5.6. The absolute average of the difference in WSE between the calibrated model and the observed WSE is 1.1 and 1.0 feet for the August 2017 and May 2020 events respectfully. The average of the difference in WSE between the calibrated model and the observed WSE is 0.95 and -0.75 feet for the August 2017 and May 2020 events respectfully.

Table 5.5. August 2017 Calibration Results.

Gage	Calibrated Model WSE Peak (ft)	Observed Peak WSE (ft)	Difference (ft)
TU2	954.5	953.2	1.3
TU3	937.9	937.5	0.4
TU4	907	904.5	2.5
TU6	884.4	883	1.4

Gage	Calibrated Model WSE Peak (ft)	Observed Peak WSE (ft)	Difference (ft)
TU7	859.8	860.3	-0.5
TU8	841.6	841	0.6
		Absolute Average	1.1
		Average	0.95

Table 5.6. May 2020 Calibration Results.

Gage	Calibrated Model WSE Peak (ft)	Observed Peak WSE (ft)	Difference (ft)
TU2	953.9	953.7	0.2
TU3	935.6	937.3	-1.7
TU4	904.8	904.2	0.6
TU6	882.6	882.7	-0.1
TU7	857.9	860	-2.1
TU8	839.5	840.9	-1.4
		Absolute Average	1.0
		Average	-0.75

To validate this calibration, high water marks were collected for the July 17th, 2025, precipitation event. See Appendix E.1 for documentation of the high-water mark collection. Validation results from the July 2025 historic event are summarized in Table 5.7. The absolute average of the difference in WSE between the calibrated model and the observed WSE is 0.8 feet, and the average is -0.37 feet. These values falls within the accepted ranges of the prior calibrated events, so no further calibration was required.

Table 5.7. July 2025 Validation Results.

HWM	Calibrated Model WSE Peak (ft)	Observed Peak WSE (ft)	Difference (ft)
TU2 (HWM 1)	956.95	956.75	0.2

HWM	Calibrated Model WSE Peak (ft)	Observed Peak WSE (ft)	Difference (ft)
TU4a (HWM 2)	909.36	910.59	-1.23
TU4b (HWM 3)	909.12	910.60	-1.48
TU5a (HWM 4)	906.86	907.40	-0.54
TU5b (HWM 5)	906.46	907.36	-0.9
TU5c (HWM 6)	905.98	904.97	1.01
TU6 (HWM 7)	890.78	891.22	-0.44
TU8 (HWM 8)	846.81	846.42	0.39
		Absolute Average	0.8
		Average	-0.37

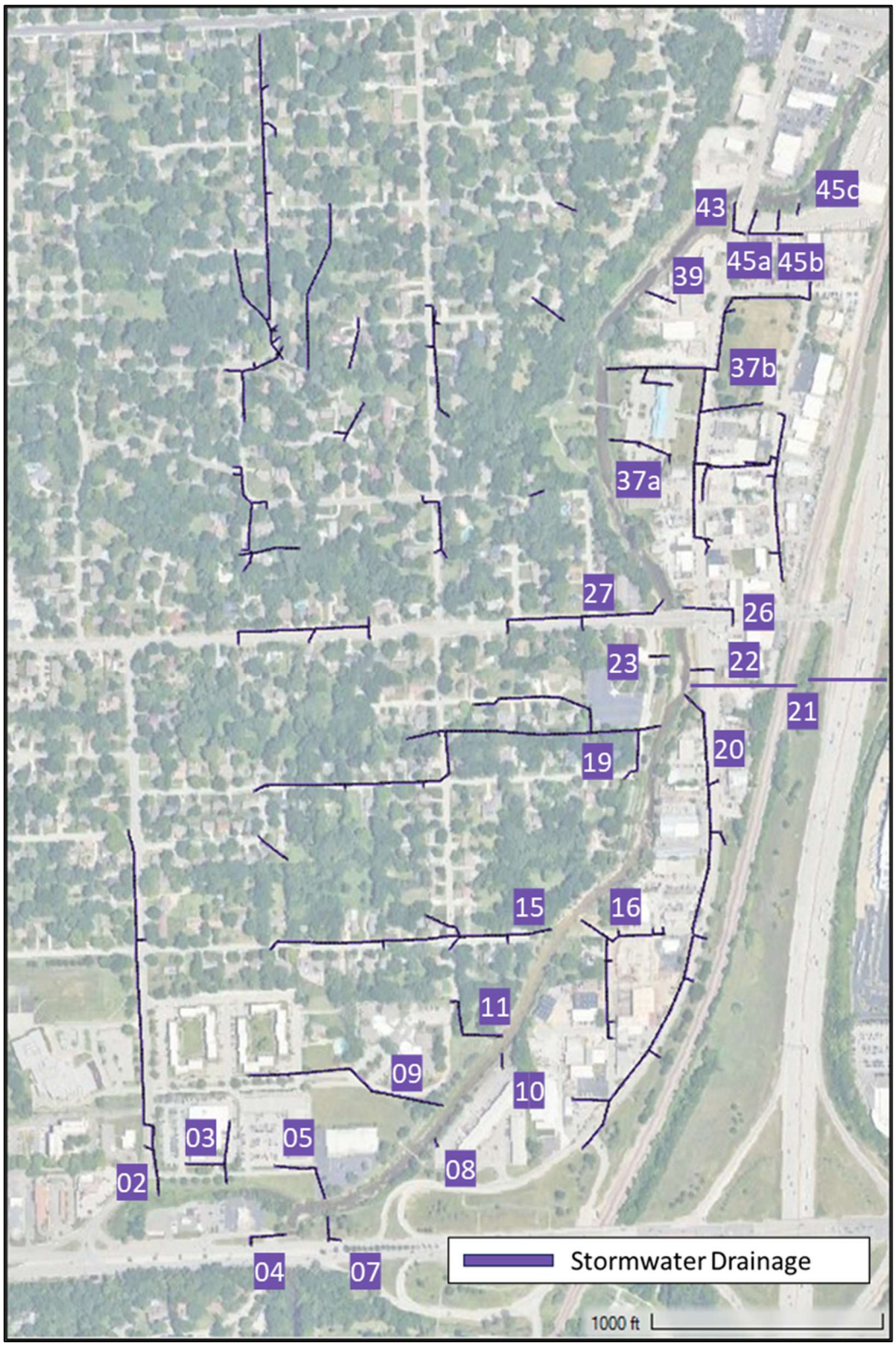
5.2.2 Existing Conditions Hydraulic Model

Stantec's base model provided the basis for the existing conditions geometry for the Turkey Creek watershed. No levee or floodwall currently exists in the Turkey Creek watershed. There is an existing depression with approximate dimensions of 100 feet by 300 feet, a maximum depth of 3 feet, and an approximate average depth of 2 feet on the right bank north of Merriam marketplace where the detention basin will be placed as part of the proposed design.

The existing conditions geometry includes the addition of the existing stormwater system within the project reach as shown in Figure 5.6. These existing structures are based on the previously mentioned shapefile of existing stormwater pipes, as well as AIMS data discussed in Section 5.1.4. See Appendix M.3 for full existing conditions pipe network data.

This geometry was used as an existing conditions baseline for hydraulic properties, including water surface elevation and depth.

Figure 5.6. Turkey Creek Existing Conditions Storm Pipe Network.

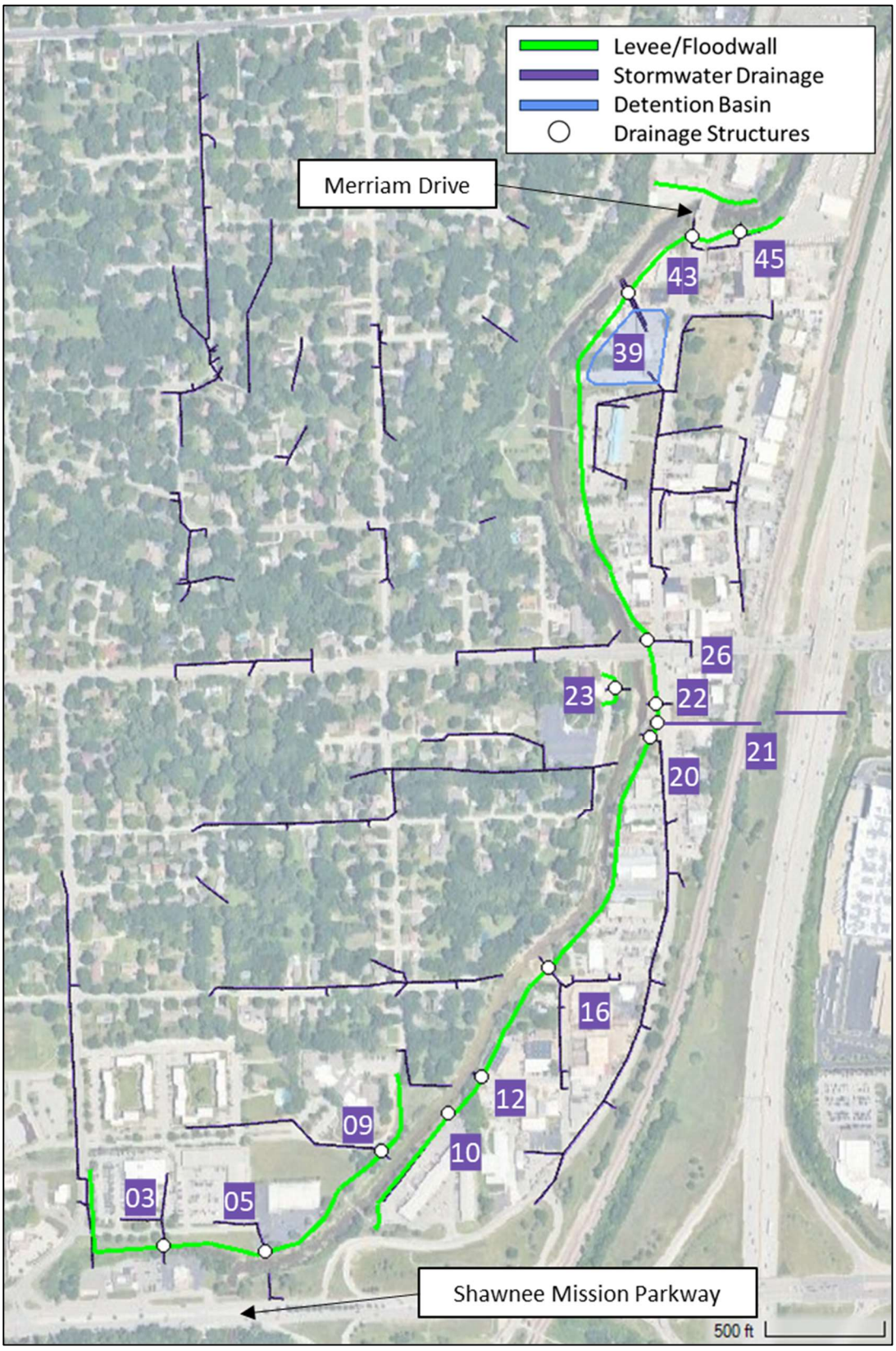


5.2.3 Proposed Conditions Hydraulic Model

The proposed work consists of construction of levee and floodwall sections, interior drainage structure additions or alterations, and a dry detention basin. This work will be completed along Turkey Creek in the City of Merriam, Kansas, from Shawnee Mission Parkway on the south (upstream) end to Merriam Drive just north of West 57th Street on the north (downstream) end.

The current recommended plan has identified a total of 14 locations where stormwater drainage lines would cross under the proposed levee or floodwall. Table 5.8 provides a summary of proposed storm drainage structures and Figure 5.7 provides locations of the drainage structures in the project area. See Appendix M.3 for full proposed conditions pipe network data.

Figure 5.7. Upper Turkey Creek Proposed Conditions Storm Pipe Network.



5.2.4 Interior Drainage Structure Design

The interior drainage structures consists of 24-inch to 60-inch RCP, as well as the existing triple 7-foot-by-7-foot box culverts, designed to drain using open-channel gravity flow. Table 5.8 contains information regarding the hydraulic model drainage system pipe design. All drainage structure pipes have a flow flap blocking all backflow from Turkey Creek into the levee interior. Pipe design was completed to meet requirements stated in Section 5.1.2 and to minimize induced impacts of the 1% AEP rainfall event on the interior area. Pipe design was limited to the conduits crossing under the levee or floodwall or connection of two drainage systems into one to reduce the number of conduits under the levee or floodwall. All structures meet the specified design criteria from section 5.1.2 and meet the goal to minimize induced impacts to the extent feasible. All conduits under the levee and floodwall alignment have a Manning's n value of 0.015 to represent reinforced concrete pipes and entrance and exit loss coefficients of 0.5 for square cut. System calculations showing pipe sizes, invert elevations, slopes, and maximum velocities are included in Appendix M.4

Table 5.8. Drainage Structure Pipe Design.

Structure Number	HEC-RAS Conduit Name	Outlet Pipe Diameter (feet)	Landside Invert Elevation (feet)	Riverside Invert Elevation (feet)	Length (feet)	Slope*
ST-03	ST-03 1	2	927.25	922.01	123.18	4.25%
ST-05	ST-05 1	3	919.3	918.98	29.31	1.09%
ST-09	ST-09 1	4	915.64	914.76	63.91	1.38%
ST-10	ST-10 1	2	912.59	912.03	41.96	1.33%
ST-12	ST-12 1	4	914.37	910.33	65.09	6.21%
ST-12B	ST-12 2	4	914.37	910.33	56.16	7.19%
ST-16	ST-16 1	2	911.13	905.93	141.22	3.68%
ST-20	ST-20 1	4	901	898.52	90.17	2.75%
ST-21	SA/2D: TTE_100	Triple Box 7' x 7'	905.65	900.23	436.92	1.24%
ST-22	ST-22 1	2	900	897.64	54.14	4.36%
ST-23	ST-23 1	3	903.36	900.68	76.23	3.52%
ST-26	ST-26 1	2	900	897.6	60.18	3.99%
ST-39	ST-39 1	4	889.75*	889.23*	198.19	0.26%
ST-39B	ST-39 2	4	889.75*	889.23*	215.52	0.24%

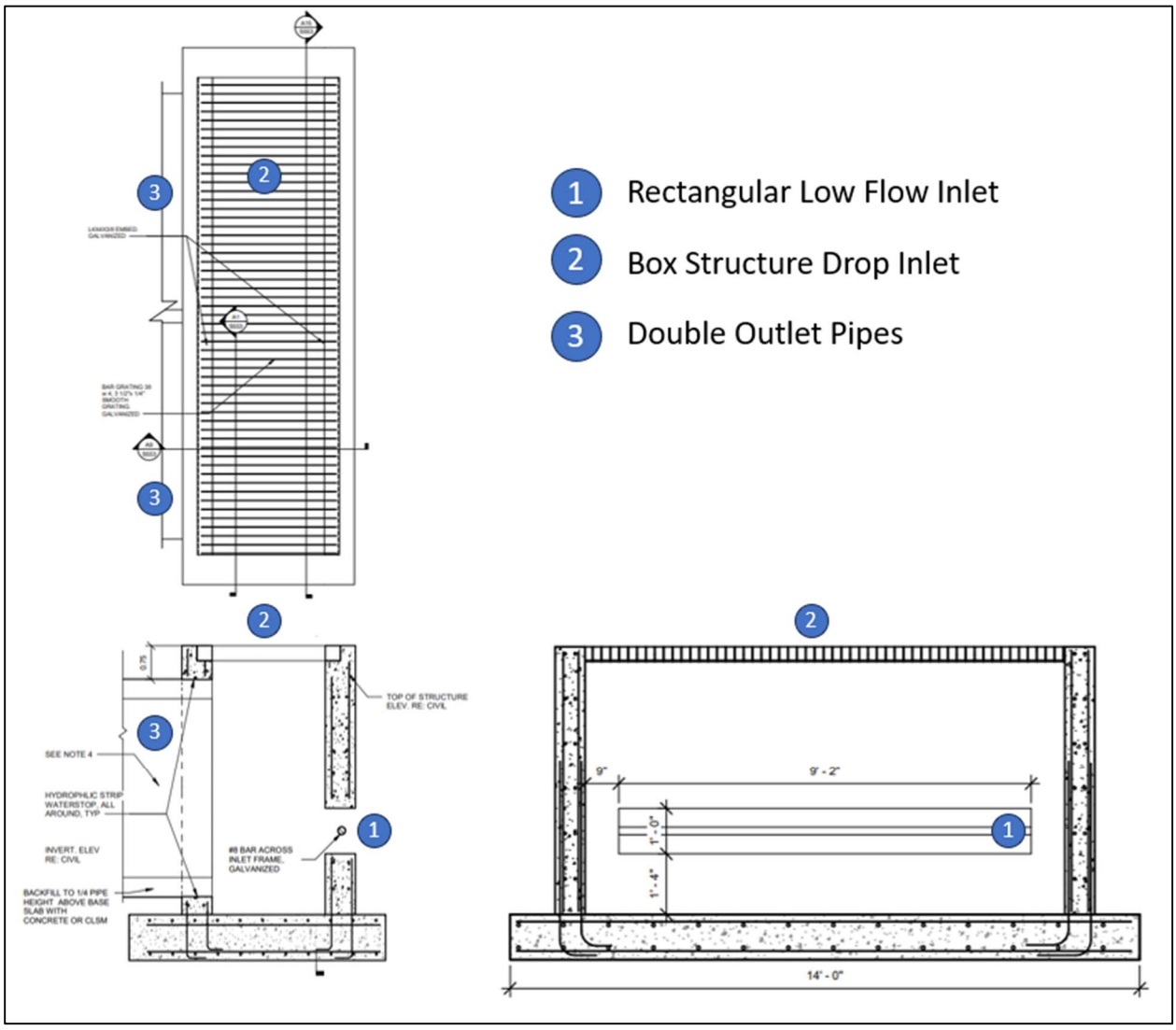
Structure Number	HEC-RAS Conduit Name	Outlet Pipe Diameter (feet)	Landside Invert Elevation (feet)	Riverside Invert Elevation (feet)	Length (feet)	Slope*
ST-43	ST-43 1	2	895	889.65	118.4	4.52%
ST-45	ST-45 1	2	893.77	890.79	61.34	4.86%

*Pipe lengths and slopes in this table reflect information from hydraulic modelling, not civil design. Hydraulic pipe lengths should be near the value provided by civil, slopes may vary due to differences in pipe length. Limitations in hydraulic modelling of pipe networks may also lead to some deviations from civil design values. These deviations are not expected to significantly impact results.

5.2.5 Detention Basin Proposed Design

An 11.0-acre-foot grassed detention basin will be constructed, with a footprint of approximately 72,300 square feet, an invert elevation of 893.75 feet, and a maximum depth of 12.25 feet. The detention basin drainage outlet structure (ST-39) is described as follows. A 12-foot X 5.5-foot box structure drop inlet at 897.75 feet with an open grate, a 1 foot X 9 foot rectangular low flow inlet at 893.75 feet, connected to two 48” diameter RCP outflow pipes with flap gates. See Figure 5.8 for structural drawings of the detention basin drainage outlet structure.

Figure 5.8. Detention Basin Drainage Outlet Structure.



This detention basin is proposed to collect stormwater runoff originating from locations landward of the levees and floodwall from the northern right bank area of the project. The structure is located north of Merriam Marketplace, shown in Figure 5.4.

This detention basin volume is based off of the ability to store the 10% AEP rainfall event below the invert elevation of the contributing storm water system. A 10% AEP rainfall event was selected for design instead of the 1% AEP rainfall event required in APWA due to detention basin size limitations and inapplicability of the site development definition. The change in design criteria was made with sponsor concurrence and is further explained in Section 5.1.1. This event was determined to have 5.52 inches of excess precipitation from the 2005 HEC-1 study as discussed in Section 5.1.5. To determine the footprint of the detention basin, a polygon was created with sides located 20 feet within the boundaries of the property lines and levee toe to provide the maximum footprint, per APWA Section 5608.2, and as shown in Figure 5.9. Side slopes of 3:1 (H:V) were implemented on the non-levee sides and a side slope of 5:1 was implemented on the levee side, per APWA 5600 Section 5608.5.A.4. A slope of 1% runs along the floor of the detention basin, leading to a low point in the northwestern corner at 893.75 feet,

where the outlet is located. The existing storm water system was altered to have one outfall into the detention basin on the southeast corner. This allowed for the highest possible outfall elevation, in turn providing the highest possible bottom of the basin. The hydraulic components can be seen in Figure 5.10. The detention basin was designed to store the 10% AEP event under gravity flow conditions, which lead to a storage capacity of 2.74 acre-feet of water below the storm water entrance invert (896.6 feet). The 10% AEP detention basin sizing was deemed acceptable by the sponsor.

Figure 5.9. Proposed Detention Basin Location.

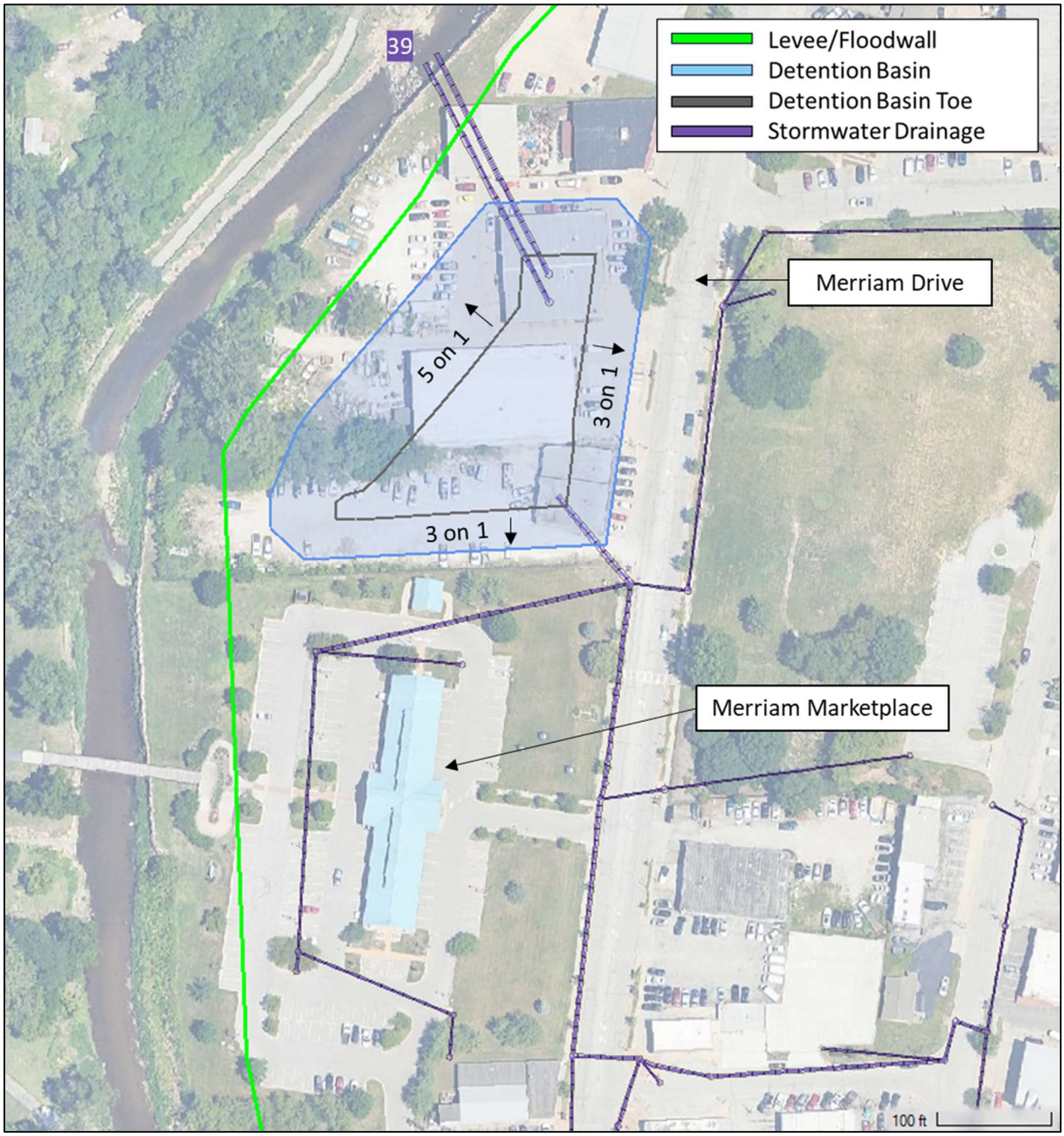
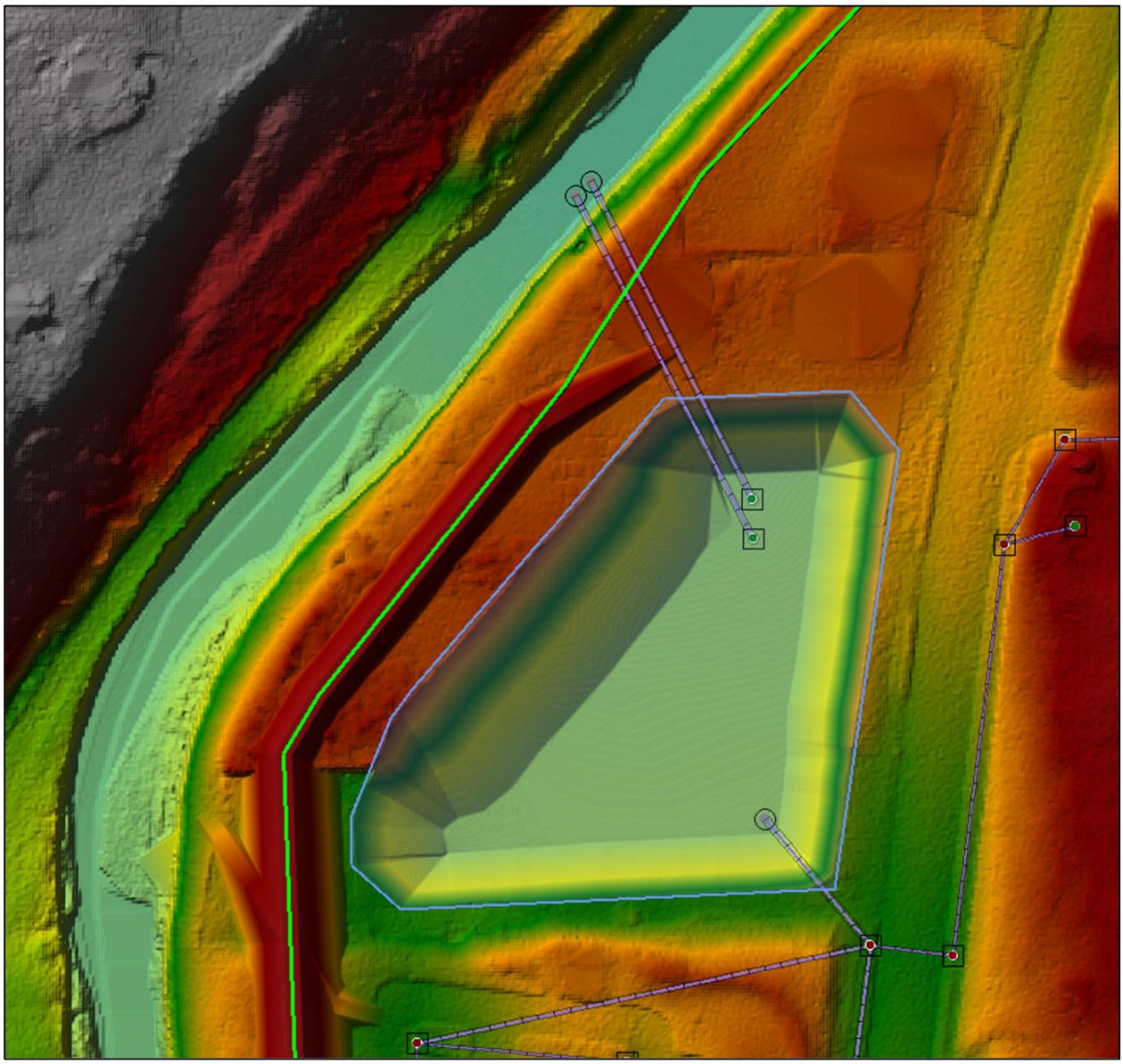


Figure 5.10. Detention Basin Implementation in the Hydraulic Model.



5.3 Coincident Frequency Analysis

A coincident frequency analysis was ruled out during the SQRA due to the small watershed size as documented in Section 4.4.2 of the SQRA. It was determined that since the contributing drainage area upstream of the project was aligned south to north and approximately 10.57 square miles, that the typical west to east storm pattern will hit both the contributing area and the interior area. Since the same system hits both Turkey Creek and the interior, it is assumed the same frequency event occurs on both the creek and the interior drainage.

5.4 Rate of Rise Analysis

A group of notable historic storm events with gridded precipitation was reviewed for rate of rise characteristics: July 26-28, 2017, August 21-23, 2017, May 27-29, 2020, July 01, 2024, and July 17, 2025. The historic storm events were used to determine realistic maximum hourly rate of rise (Table 5.9) and warning time (Table 5.10). Warning time is defined as the time from the first rainfall to the time when the channel WSE causes the drainage structure flap gates to close.

Table 5.9. Historic Storm Event Maximum Hourly Rate of Rise (ft/hr).

Storm Event	ST-03	ST-05	ST-09	ST-10	ST-12	ST-16	ST-20	ST-21	ST-22	ST-23	ST-26	ST-39	ST-43	ST-45
July 2017	10.0	9.0	10.2	8.2	7.4	9.6	11.5	11.6	11.5	11.5	11.8	12.8	12.3	11.0
August 2017	5.7	5.0	6.0	8.4	4.3	5.1	7.7	7.8	7.8	7.6	8.4	9.0	8.9	8.1
May 2020	3.0	3.8	4.5	7.4	4.3	3.4	6.1	6.1	6.1	6.2	6.7	6.2	5.8	5.2
July 2024	8.3	8.0	9.6	6.8	6.4	7.1	11.6	11.6	11.7	11.9	12.6	12.7	12.2	11.0
July 2025	14.2	12.9	14.4	9.3	10.4	13.0	17.5	17.5	17.4	17.4	18.0	19.4	18.7	16.9

EM 1110-2-2902, Conduits Pipes and Culverts Associated with Dams and Levee Systems lists the closure type requirements for different flood rise conditions. The flood rise condition for the project was considered fast since there was less than 12 hours of warning time as shown in Table 5.10. This requires the use of passive closure structures.

Table 5.10. Historic Storm Event Warning Time (hrs).

Storm Event	ST-05	ST-10	ST-12	ST-16	ST-20	ST-21	ST-22	ST-26	ST-39
July 2017	4.57	4.53	4.43	4.45	-	-	4.5	4.5	4.42
August 2017	-	-	-	-	-	-	-	-	3.85
May 2020	-	-	-	-	-	-	-	-	6.47
July 2024	-	-	-	-	-	-	-	-	0.7
July 2025	-	-	-	-	1.58	0.78	1.57	1.57	0.75

*Drainage structures not included in this table do not reach closure conditions, and in turn do not register a warning time across all historic events.

5.5 Analysis of Existing and Proposed Conditions with Theoretical Storm Events

Table 5.11 shows structural flooding and subbasin drainage area. Structural flooding is defined as the WSE required for the first building in the subbasin to be touched by water. These structural flooding elevations were determined using survey data where possible and a combination of AIMS and LiDAR data where survey data was not available, as referenced in Section 5.1.4. These structural flooding elevations were used to evaluate critical subbasins. The subbasin drainage areas can be seen in plan view in Figure 5.4. The drainage structure numbering system matches the subbasin numbering system throughout this report.

Table 5.11. Subbasin Properties.

Subbasin Number	Structural Flooding Elevations (feet)	Drainage Area (acres)
ST-03	936.0	3.8
ST-05	933.4	3.7
ST-09	932.4	16.3
ST-10	929.8	5.4
ST-12	923.0	2.2
ST-16	917.0	5.7
ST-20	912.0	22.9
ST-21	926.0	434.2
ST-22	912.0	2.2
ST-23	916.1	1.2
ST-26	910.0	1.3
ST-39	903.9	26.2
ST-43	909.5	1.4
ST-45	908.0	4.8

The two scenarios analyzed for the existing and proposed conditions for interior drainage are the 10% AEP, 24-hour rainfall event and 1% AEP, 24-hour rainfall event.

Two specific goals for the proposed design were to ensure the identified conduit size and location were sufficient to convey flow; and (where possible) to not induce increased water surface or structural flooding due to the presence of the levee alignment.

The design team attempted to reduce the peak WSE in all the subbasins so there were no areas with increases in peak WSE. However, due to rapid flap gate closure occurring due to the channel WSE being greater than the interior WSE, any feasible increase in pipe capacity could not solve the problem. An increase in detention capacity would be required to reduce the peak WSE. However, detention options were not feasible on a real estate or cost basis, apart from the location north of Merriam Marketplace. The design team settled on flowage easements to

mitigate the increase in peak WSE. See Section 5.7 for more information on the flowage easement areas.

The data tolerance threshold used for the analysis is 0.1 feet. This includes numbers that round to 0.1 feet, so any value greater than or equal to 0.05 feet meets the data tolerance threshold.

The 10% AEP theoretical event results in Table 5.12 show that five of the fourteen subbasins experience an increase in peak WSE from existing to proposed conditions. However, structural flooding is reduced from four subbasins in the existing conditions to two subbasins with the proposed conditions. See Figure 5.11 for WSE change mapping for the 10% AEP event.

The 1% AEP theoretical event results in Table 5.13 show that three of the fourteen subbasins experience an increase in peak WSE from existing to proposed conditions. However, structural flooding is reduced from nine subbasins in the existing conditions to seven subbasins with the proposed conditions. See Figure 5.12 for WSE change mapping for the 1% AEP event.

Table 5.12. 10% AEP Theoretical Event Results.

Drainage Structure Number	ST-03	ST-05	ST-09	ST-10	ST-12	ST-16	ST-20	ST-21	ST-22	ST-23	ST-26	ST-39	ST-43	ST-45
Change in WSE (ft)	0.39	-0.54	0.19	-0.09	-0.38	0.19	-0.34	-0.03	-0.30	-0.67	-0.40	-7.11	0.08	0.05
Maximum Existing Depth (ft)	0.68	1.46	0.38	0.57	1.15	0.54	0.50	6.76	0.60	1.61	1.18	2.54	0.65	0.00
Maximum Proposed Depth (ft)	1.07	0.95	0.59	0.46	0.77	0.68	0.14	6.73	0.32	0.97	0.79	5.96	0.73	0.05
Duration of Proposed WSE > Existing WSE (hours)	24.1	22.3	21.6	2.3	21.1	5.7	23.1	0.9	0.4	1.1	15.5	0.0	22.6	0.2
Duration of Drainage Structure Closure (hrs)	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.0	0.8	4.4	0.0	0.0
Does Structural Flooding Occur with Existing Conditions	NO	NO	NO	NO	NO	NO	YES	NO	NO	NO	YES	YES	NO	YES
Does Structural Flooding Occur with Proposed Conditions	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	YES	NO	NO	YES
Maximum Channel Rate of Rise (ft/hr)	13.2	4.1	14.6	9.1	9.7	12.7	15.9	17.4	17.2	17.3	18.0	5.4	18.9	16.9

Table 5.13. 1% AEP Theoretical Event Results.

Drainage Structure Number	ST-03	ST-05	ST-09	ST-10	ST-12	ST-16	ST-20	ST-21	ST-22	ST-23	ST-26	ST-39	ST-43	ST-45
Change in WSE (ft)	-2.01	-3.06	0.35	-1.38	-0.07	-1.95	-3.82	2.11	-3.80	-1.10	-3.80	-4.39	-1.67	0.43
Maximum Existing Depth (ft)	3.14	6.38	1.51	2.90	2.55	2.84	4.54	12.13	5.06	2.70	5.94	8.60	4.09	0.86
Maximum Proposed Depth (ft)	1.13	3.05	1.86	1.51	2.48	0.85	0.70	14.24	1.28	1.62	2.15	14.74	2.43	1.29
Duration of Proposed WSE > Existing WSE (hours)	22.4	22.4	22.5	2.4	21.5	5.1	21.7	1.9	0.6	0.9	11.0	0.0	20.1	2.0
Duration of Drainage Structure Closure (hrs)	1.5	1.4	0.9	0.7	0.2	1.4	1.9	0.0	1.9	0.0	1.8	3.1	0.9	0.3
Does Structural Flooding Occur with Existing Conditions	NO	YES	NO	NO	YES	YES	YES	NO	YES	NO	YES	YES	YES	YES
Does Structural Flooding Occur with Proposed Conditions	NO	NO	NO	NO	YES	NO	YES	NO	YES	NO	YES	YES	YES	YES
Maximum Channel Rate of Rise (ft/hr)	19.3	20.4	17.0	11.9	13.6	15.8	18.8	20.0	19.9	20.0	20.5	22.2	22.8	20.4

Figure 5.11. 0.1 Annual Exceedance Probability, 24-hour rainfall event: Change in Water Surface Elevation from Existing Conditions to Proposed Conditions (Map Value=Existing Conditions-Proposed).

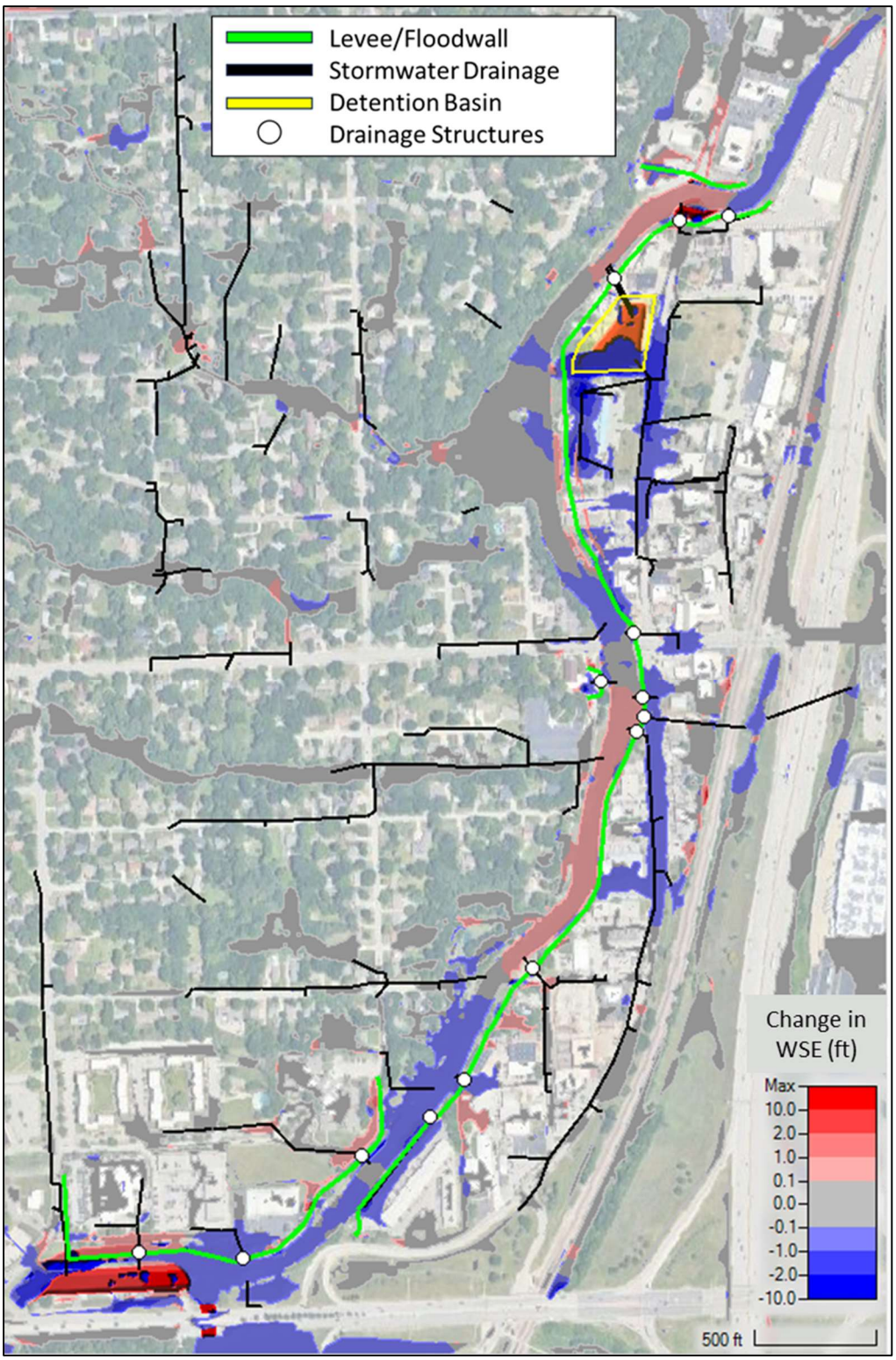
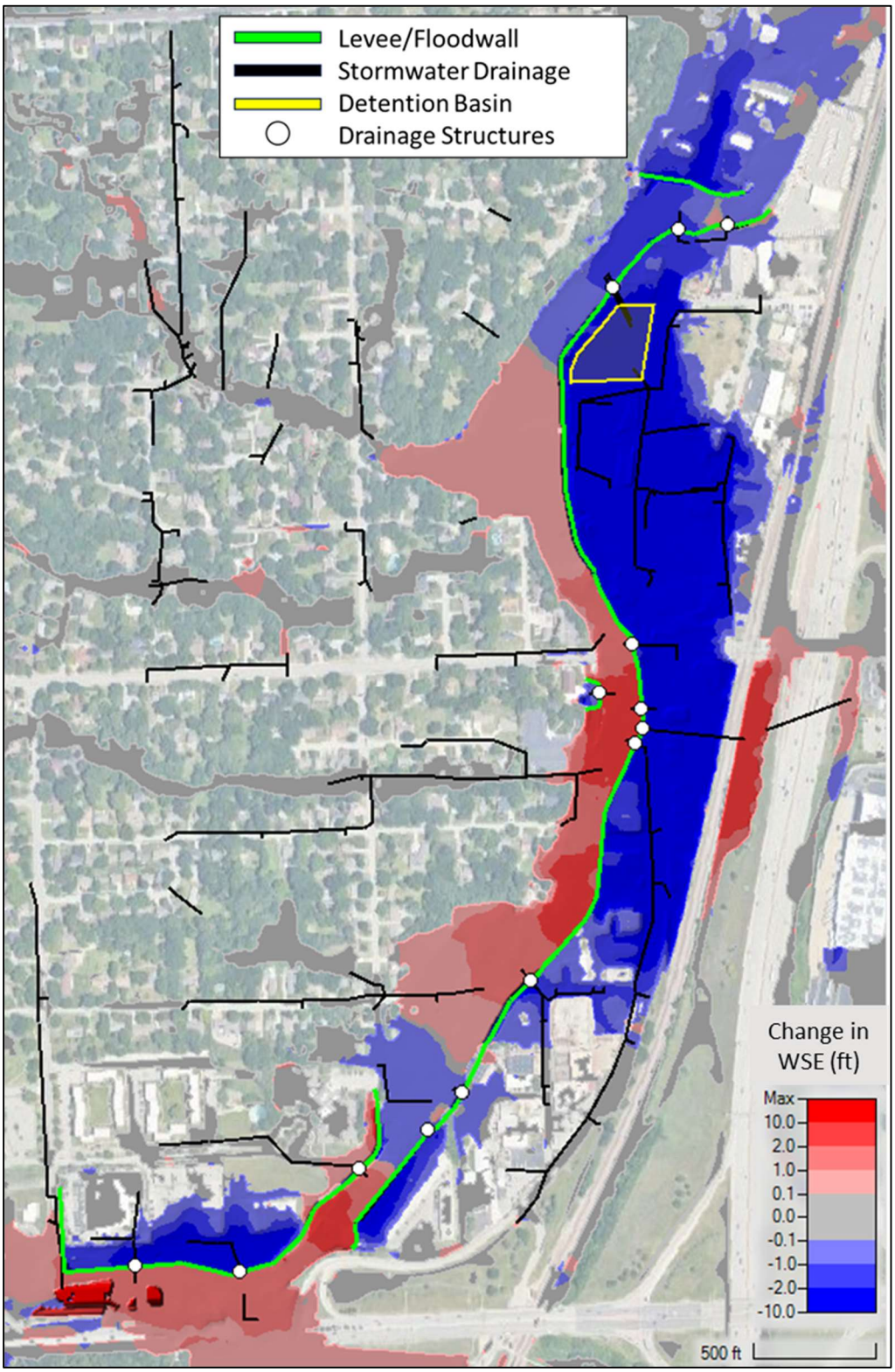


Figure 5.12. 0.01 Annual Exceedance Probability, 24-hour Rainfall Event: change in Water Surface Elevation from Existing Conditions to Proposed Conditions (Map value=Existing Conditions-Proposed)



5.6 Induced Impacts Mapping

Interior areas were reviewed for induced impacts from peak WSE and duration. Induced impacts from an increase in peak WSE were mitigated where possible by increasing drainage structure capacity or providing detention volume. The remaining areas with increased peak WSE have been deemed either unfeasible or not cost effective to mitigate as part of design.

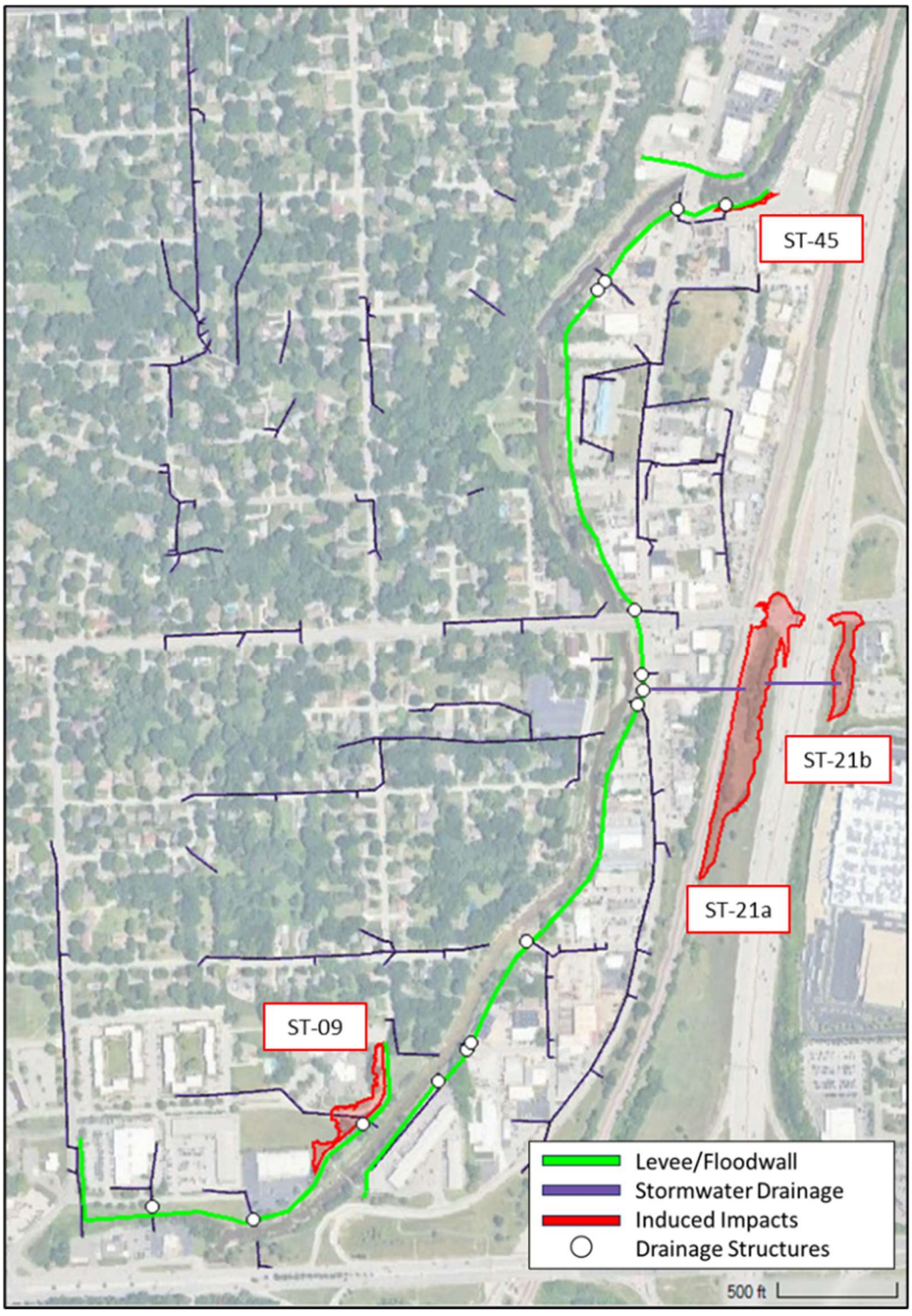
See Table 5.14 for acreage of induced impacts and Figure 5.13 for the locations of induced impacts in plan view. Induced impact areas were clipped to interior locations where the water surface elevation increases rounded to greater than or equal to 0.1 feet. 0.1 feet was selected because it accurately reflects hydraulic trends without introducing the oscillations at extremely shallow flow. The induced impact areas are comprised of areas of new inundation and increased water surface elevation. The 1% AEP rainfall event was used as the basis for determining the induced impacts. Additionally, impacts from permanent easements are considered in the area determination.

Interior areas with an increase in duration of inundation are captured in the “Duration of Proposed WSE > Existing WSE (hours)” row of Table 5.12 and Table 5.13. These interior areas were determined to provide a relative benefit since the peak inundation WSE is lower than without the project. This relative benefit may counter a potential takings claim.

Table 5.14. Induced Impact Areas.

	ST-09	ST-21a	ST-21b	ST-45	Total
Interior Induced Impact Area (acres) w/ Project 1% AEP	0.63	2.87	0.65	0.03	4.18

Figure 5.13. Peak Induced Impact Mapping.



5.7 Interior Drainage References

USACE. (2018). *Hydrologic Analysis of Interior Areas, EM 1110-2-1413.* U.S. Army Corps of Engineers, Washington, D.C.

USACE. (1993). *Hydrologic Frequency Analysis, EM 1110-2-1415.* U.S. Army Corps of Engineers, Washington, D.C.

USACE. (2020). *Conduits Pipes and Culverts Associated with Dams and Levee Systems, EM 1110-2-2902.* U.S. Army Corps of Engineers, Washington, D.C.

American Public Works Administration. (2011). *Storm Drainage Systems & Facilities, Specification Section 5600.*

NOAA. (2013). *Precipitation-Frequency Atlas of the United States, NOAA Atlas 14, Volume 8, Version 2: Southeastern States*. National Oceanic and Atmospheric Administration, National Weather Service, Silver Spring, MD.

USACE. (2024). *HEC-RAS 2D User's Manual, Latest.* U.S. Army Corps of Engineers, Hydrologic Engineering Center: https://www.hec.usace.army.mil/confluence/rasdocs/r2dum/latest

USACE. (2024). *HEC-RAS Hydraulic Reference Manual, Latest.* U.S. Army Corps of Engineers, Hydrologic Engineering Center: https://www.hec.usace.army.mil/confluence/rasdocs/ras1dtechref/latest

Stantec. (2024). *Turkey Creek and Brush Creek 2D Watershed Hydrologic and Hydraulics Report, DRAFT.* Stantec, Overland Park, KS

McEnroe, B. (2023). *Rainfall for Flood Studies: Indian and Tomahawk Creeks*. Johnson County Public Works, Johnson County, KS.

This page was intentionally left blank for duplex printing.

6.0 SUBSURFACE CONDITIONS

This section discusses subsurface investigation and soils laboratory testing and presents subsurface conditions and engineering soil properties for design of the Upper Turkey Creek FRM Project.

6.1 Regional Geology and Geomorphology

This section discusses regional geology near the Upper Turkey Creek FRM Project.

6.1.1 Location

The Turkey Creek Flood Control Project is in the Osage Plains region of the central lowland physiographic region. The generally rolling to level upland has been eroded by Turkey Creek and associated drainageways to create vertical relief of more than 200 feet between the highlands and lowlands of the Turkey Creek channel. Bedrock is exposed in numerous places along the valley walls and in the creek bottom.

6.1.2 Soils

The soils in the Turkey Creek flood plain area are mostly alluvial deposits overlying eroded bedrock surface. The surficial soils generally are comprised of unclassified fill overlying lean clay with layers of clayey gravel found at depth near the bedrock surface. The native alluvial silts and clays belong to Onawa-Haynie Association. The hillside soils in the valley are derived from loess and residual soils of the Knox-Ladoga Association.

The native lean clay deposits (CL) are described as medium to stiff and very stiff consistency materials with liquid limits less than 50 percent and plasticity indices varying from about 13 to 29 percent. Natural moisture contents reported in the exploration data vary from 21 to 29 percent.

There are occasional layers of fat clay deposits within these natural deposits. The fat clays have liquid limits on the order of 52 percent.

Groundwater in the region is generally encountered a few feet above the bedrock surface and fluctuates with ground surface elevation. A relatively thin layer (approximately 4 feet) of clayey gravel immediately above the bedrock is a water bearing layer.

6.1.3 Fills during the Channelization of Turkey Creek

Some fills from the excavation were randomly placed in the study area. Fills from I-35 rock cuts were placed along the left bank and the construction of a low-level levee and railroad yard on the right bank resulted in fills of unclassified materials. The fills have been in place for many years and appear to have consolidated under their own weight.

Some evidence of dumped debris along the right bank and within the flood plain area exists. This dumped debris is generally unsuitable material for use in engineered fills.

6.1.4 Bedrock

Bedrock is exposed in numerous places along the valley walls and in the creek bottom. Bedrock consists of sedimentary bedrock of the Pennsylvanian Age. The rock units along the creek bottom and the valley walls are alternating shales and limestones of the Bronson and Linn subgroups of the Kansas City Group. The high hilltops outside the study area are part of the Zarah subgroup.

The valley walls are eroded into the formations of the Linn subgroup, which are predominantly shales with several limestones. From the top down, the rock units are the Iola Limestone Formation, the Chanute Shale Formation, the Drum Limestone Formation, and the Cherryvale

Formation. The Cherryvale Formation consists of the following members in descending order: Quivira Shale, Westerville Limestone, Wea Shale, Block Limestone, and Fontana Shale. Of particular note is the Westerville Limestone, which in the upper portion is highly variable in lithology, usually consisting of cross-bedded oolitic limestone but may contain varying amounts of shale. The lower 3 to 4 feet is consistently hard, dense limestone. Vertical faces of the Westerville Limestone in this area may not be stable over the long term and the unit may be water bearing.

The valley floor is cut into the formations of the Bronson Subgroup, which are predominately limestones with several shales. From the top down, the rock units are the Dennis Limestone Formation, the Galesburg Shale Formation, and the Swope Limestone Formation. The Dennis Formation consists of the Winterset Limestone Member and the Stark Shale Member. The Swope Limestone Formation consists of the Bethany Falls Limestone Member, the Hushpuckney Shale Member, and the Middle Creek Limestone Member. The upper several feet of the Bethany Falls Limestone consist of a nodular limestone in a shale matrix and is referred to as the *peanut zone*.

The intervening Galesburg Shale Formation consists of Canville limestone (usually absent) and the Galesburg Shale. Differentiating the Galesburg Shale from the overlying Stark Shale is sometimes difficult. The two shales are frequently grouped together as the Stark-Galesburg Shale even though they are from different formations. Review of KDOT profiles for I-35 in the project vicinity indicates a noticeable thickening of this unit to the north, with the appearance of the Canville limestone.

The present Turkey Creek alignment generally flows on or near the top of the bedrock surface. The exposed bedrock in the channel is either shale or Winterset Limestone in the upper reaches while Bethany Falls Limestone is present in the lower reaches near the tunnel entrance.

6.2 Historical Subsurface Investigations

Prior to the most recent subsurface investigation in 2023, several subsurface investigations were completed in the general area, such as for I-35 or for other reaches of Turkey Creek. Two soil borings completed in 2002 are the only borings performed in the current project area for which records have been located. The following subsurface investigations were conducted.

- Corps of Engineers Borings of 1959—Most of these borings were drilled to the top of bedrock or auger refusal. Boring locations and ground elevations are very approximate and were obtained by interpolating between contours of the most recent topographic maps.
- Corps of Engineers Borings of 1989 and 1994–95—These borings included coring of the bedrock. Boring locations and ground elevations were based on actual ground surveys. Limited laboratory tests on typical overburden soils were performed. Standard penetration tests were available for the overburden soils. Groundwater conditions were noted.
- Geologic exploration conducted for the I-35 Highway Project by the Kansas and Missouri Departments of Transportation—Included geologic profiles along the highway and subsurface information at various bridge crossings.
- Feasibility Study (Borings in 2002, Study published in 2015)—The Feasibility Study was conducted over a long time and a large area. As part of the Feasibility Study, an Integrated Environmental Assessment included investigation of 20 square miles of the Upper Turkey Creek watershed. Several pilot borings were conducted along Turkey Creek with the majority placed near the Kansas River located at U.S. 69 Highway and I-

35 and eastward. Two borings drilled in 2002 by USACE, labeled DC-4 and DC-5, were conducted in the current project area.

- These two soil borings revealed mostly silty clay fill and weathered shale to auger refusal on limestone at approximately 13 feet below ground surface. The subsurface investigation continued with rock coring to between 25.9 and 27.6 feet below ground surface.
- Various Terracon projects in the general vicinity.

6.3 2023 USACE Subsurface Investigation

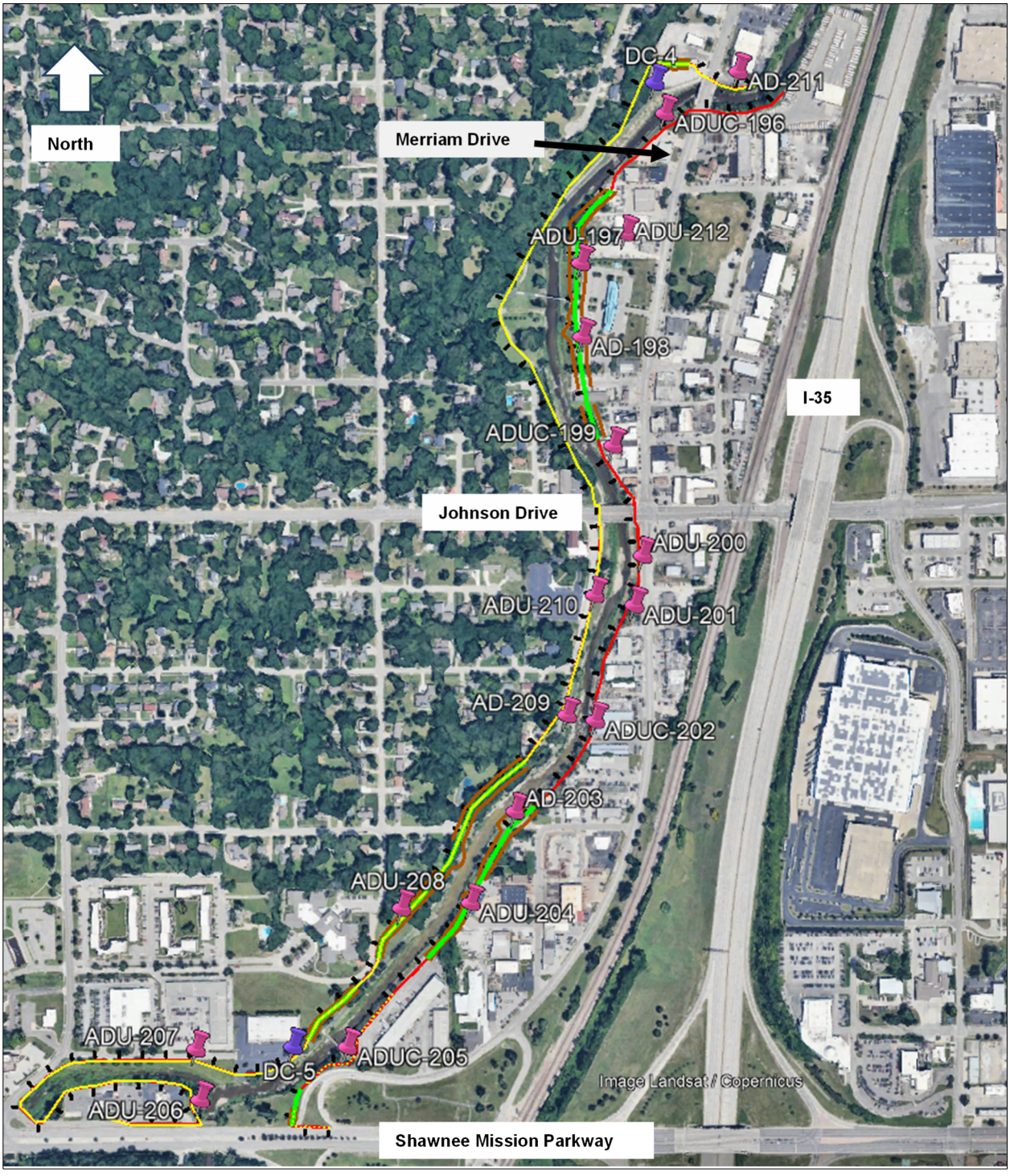
In 2023, USACE conducted a subsurface investigation to collect subsurface samples for the design and construction of the floodwall and levee system on Upper Turkey Creek. A total of 17 borings were conducted as part of this investigation. Boring logs are included in Appendix A. Split spoon samples were taken every 5 feet until refusal. Relatively undisturbed Shelby tube samples were taken at predetermined depths.

The target depth for all borings was in a range between 12.9- and 23-feet deep, which was based on historical borings in the area. While most soil samples met their target depth, some borings reached refusal between 6 and 8 feet in depth due to the shallow bedrock, resulting in crushed Shelby tubes.

Groundwater was encountered in 7 of the 17 borings and ranged from 10.3 to 21 feet below grade. In five of the seven borings where groundwater was encountered, it was found within 2.5 feet of bedrock. During this investigation, performed between January and March, the water level in Turkey Creek was seasonally low. Groundwater is expected to vary seasonally with precipitation and creek levels.

Figure 6.1 shows an overview map of boring locations on aerial photograph. Feasibility Study Borings DC-4 and DC-5 are shown in purple, while the 2023 borings are shown in pink. The levee alignment shown is approximate and for general reference only; the project plans should be referenced for the current proposed levee alignment. Boring locations are also shown on existing conditions plans and proposed features plans in the plan set.

Figure 6.1. Boring Locations.



6.3.1 Environmental Contamination

Environmental contamination (petroleum) was found within borings ADU-212 and ADU-201. Boring ADU-212 contamination was old and deteriorated contamination from a leaking underground storage tank (LUST) site that was closed by the Kansas Department of Health and Environment (KDHE). Random fill was also found in the borings and included large pieces of railroad ties. It is assumed that the former tank pit was backfilled with readily available materials at the time.

Boring ADU-201 soil contamination was not a known KDHE site. The source of the contamination is unknown. The current building was not constructed until the mid-1980s, but historical aerials show a possible former gas station in the 1940s-50s. The contaminated soil was contained, and a soil sample was submitted to Terracon for lab analysis of hydrocarbons and volatile organic compounds. Lab analysis revealed the soil was above default target levels and above residential levels for medium-range hydrocarbons but below non-residential standards. There are no residences nearby on this side of Turkey Creek, therefore non-residential standards would apply. The drummed soil was placed on site and was picked up by Safety Kleen®.

6.3.2 Drilling and Sampling Methods

Recent borings were completed with a Diedrich D-90 truck-mounted auger/rotary rig with hollow stem augers. In the historical and recent USACE investigations, both disturbed and undisturbed samples of the overburden soils were obtained at 3-to-5-foot intervals. Samplers used were 1 3/8-inch standard split spoon and 3-inch or 5-inch, thin-wall Shelby tubes. The bedrock was cored with an NX size core barrel. In-between core samples, the borings were advanced through the bedrock using rock bit drilling.

6.3.3 Disturbed Soil Samples Collection

Standard Penetration Testing (SPT) using a split-spoon sampler was performed in general accordance with ASTM D 1586 to obtain disturbed samples at predetermined depths. The split-spoon sampler was driven a maximum depth of 1.5 feet for each sample. A 140-pound safety hammer was dropped approximately 30 inches to advance the sampler. The number of blows for each six-inch increment was recorded on the boring logs. The blow count value, or SPT N-value, corresponds to the sum of blows for the last two six-inch increments. Each SPT was driven the full penetration depth or until refusal, whichever occurred first, with refusal defined as a blow count of 50 for less than 6 inches of sampler penetration. Material collected from the split-spoon sampler was transferred into labeled glass jars and tightly capped for storage and later transport to the soils laboratory for testing.

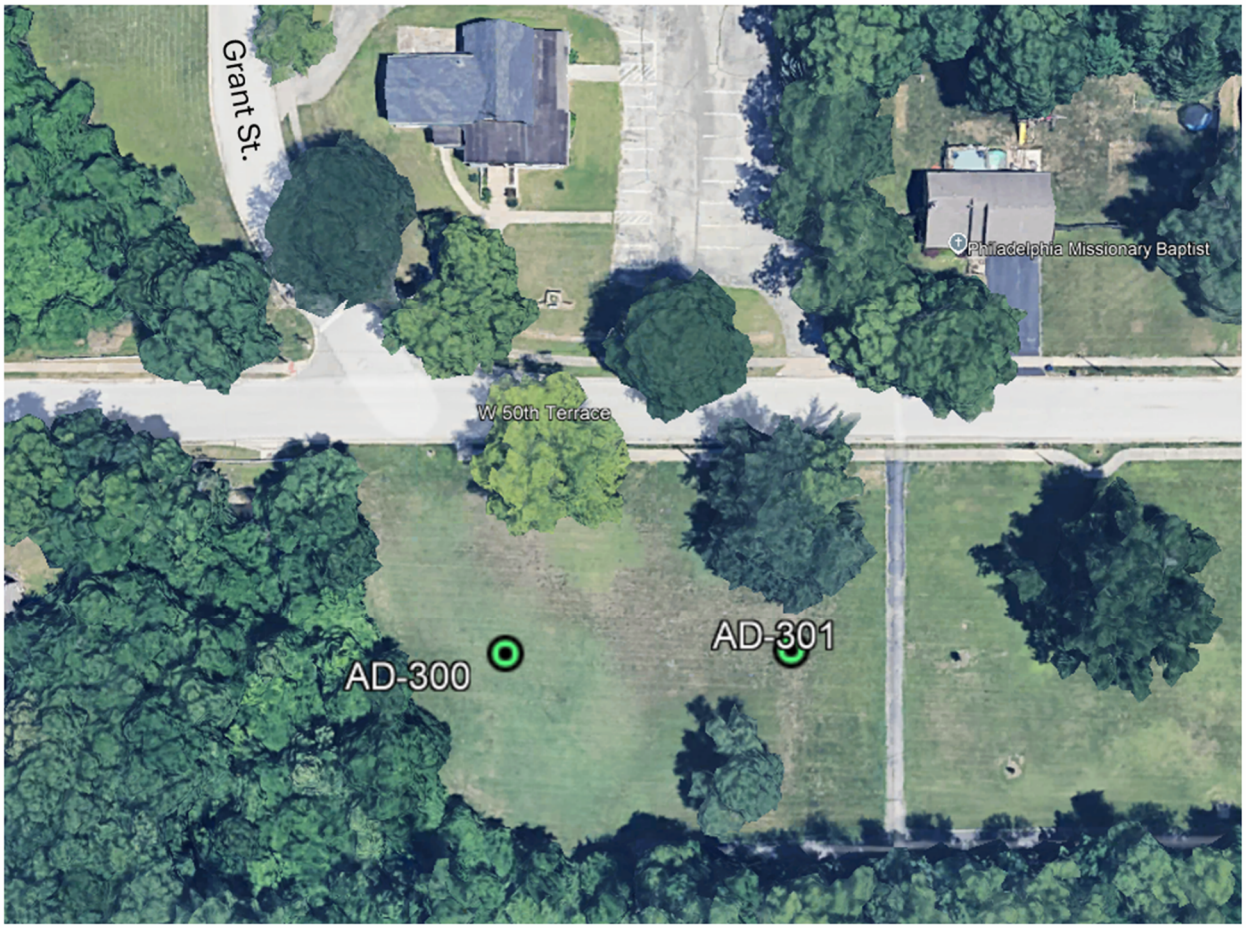
6.3.4 Undisturbed Soil Samples Collection

Relatively undisturbed samples were collected by pushing 3-inch or 5-inch outside diameter Shelby tubes in general accordance with ASTM D 1587. 3-inch Shelby tube samples were left in the tubes, with the open ends capped and taped, then sent to the lab for extruding and testing. 5-inch Shelby tube samples were extruded in the field and waxed for increased preservation.

6.4 2025 USACE Subsurface Investigation

Two additional soil borings (Borings AD-300 and AD-301) were drilled by USACE at a second proposed offsite borrow area, an open lawn area on the south side of W 50th Terrace and near Grant Street in Merriam, Kansas. This site appears to be relatively level and USACE is not aware of any current planned development at the site.

Figure 6.2. Boring Locations within 2nd Potential Borrow Area .



Borings extended to 15 feet below grade and encountered lean to fat clays and gravelly fill. Split spoon samples and bulk samples were taken for laboratory testing.

Laboratory testing was recently completed and is being incorporated into the final boring logs.

6.5 Future Subsurface Investigation

If the project is funded to continue, borings in proposed I-wall areas will be used to verify depth of bedrock to confirm if sheet piles can be driven to design depths without encountering bedrock, if predrilling through a small amount of rock will be required for sheet pile installation, or if T-walls need to be used in place of I-walls due to shallow bedrock. The I-wall area borings will be drilled to bedrock, expected at 10 to 25 feet below grade. It is proposed that approximately 10 borings be drilled in I-wall areas.

6.6 Historical Laboratory Testing

6.6.1 General

There is no record for historical laboratory testing on soil samples from the current project area. Laboratory tests were performed on soil samples from other reaches of Turkey Creek in the historical subsurface investigations by the USACE Missouri River Division Laboratory in Omaha, Nebraska. The tests generally included index tests and shear strength tests. The general results are summarized here for context.

6.6.2 Soil Classification Tests

Water content and Atterberg Limits tests were provided to classify soils and evaluate site stratigraphy. Based on the available data, the surficial soils are generally lean clays with liquid limits ranging from 32 to 49 percent and plasticity indices ranging from 13 to 29. The natural moisture content varied from about 21 to 30 percent.

6.7 Recent Laboratory Testing

6.7.1 General

Laboratory testing of soil samples collected from the borings along the levee alignment performed in 2023 was completed by Terracon and included Atterberg limits, sieve analysis, Unified Soil Classification System (USCS) classification, and moisture content. Additionally, Shelby tube density, consolidated-undrained (CU) and unconsolidated-undrained (UU) triaxial shear strength tests, and consolidation tests were performed on undisturbed soil samples extracted from Shelby tubes. Three rock cores were tested for compressive strength.

Laboratory testing of soil samples collected from the borings performed at the potential borrow site on the south side of W 50th Terrace near Grant Street in Merriam, Kansas was performed by the USACE St. Louis District. These tests included Atterberg limits, sieve analysis, USCS classification, and moisture content. In addition, bulk samples were tested for maximum dry density and optimum moisture content (standard Proctor). Remolded samples were created by compacting bulk samples to 95% of the maximum dry density, the compaction specified for levee embankment construction. CU triaxial shear strength testing was performed on remolded samples.

Appendix A contains the individual test data sheets and a summary table of these tests. Test results are also included on individual boring logs. Atterberg limit, sieve analysis, and USCS classification results were used to verify field soil classifications. Shelby tube density results were used to verify soil unit weights. Triaxial soil shear strength tests were used to establish soil strength parameters for global slope stability and for floodwall foundation design. Consolidation tests were used to estimate settlement of the in-situ soils due to levee construction.

Not all tests could be conducted as planned. Some of the Shelby tube samples had low recovery, were disturbed, or had substantial amounts of rock, making triaxial tests impossible. A second round of testing was attempted with additional Shelby tube samples to have more triaxial test results; however, these samples were also not usable.

6.7.2 Soil Classification Tests

Water content and Atterberg limits tests were provided to classify soils and evaluate site stratigraphy. Based on the available data, the surficial soils are generally lean clays with liquid limits ranging from 38 to 58 percent (only one sample over 50 percent) and plasticity indices ranging from 16 to 36 (only one sample over 28). The natural moisture content varied from about 21 to 30 percent.

Surficial soils at the borrow area on W 50th Terrace are generally lean clays with liquid limits ranging from 45 to 55 and plasticity indices ranging from 23 to 34. The natural moisture content varied from about 17 to 26 percent.

6.7.3 Unit Weight

Table 6.1 provides the unit weight of eight Shelby tube samples. The unit weight for the AD-198 sample appears significantly higher than expected, perhaps due to gravel in the sample.

Table 6.1. Soil Unit Weight.

Sample	Dry Unit Weight (per cubic foot [pcf])	Water Content (percent)	Moist Unit Weight (pcf)
ADU-197, 5 feet–7 feet	113.1	14.6	129.6
ADU-197, 10 feet–12 feet	95.8	26.0	120.7
AD-198, 10 feet–12 feet	124.2	28.9	160.1
ADUC-202, 5 feet–7 feet	96.6	23.2	119.0
ADU-204, 2 feet–4 feet	99.7	22.2	121.8
ADU-206, 5 feet–7 feet	103.1	18.1	121.8
ADU-207, 5 feet–7 feet	92.5	25.0	115.6
ADU-208, 1 foot–3 feet	94.3	25.4	118.3
Average (excluding ADU-197)	99.3	22.1	121.2

6.7.4 Consolidated-Undrained Triaxial Shear Strength

Table 6.2 provides CU shear strength test results on four samples along the levee alignment and Table 6.3 provides CU test results for two bulk samples from the borrow site on W 50th Terrace, remolded to 95 percent of the standard Proctor maximum dry density.

Table 6.2. Consolidated Undrained Test Results.

Sample	USCS	Liquid Limit	Effective Cohesion (psf)	Effective ϕ (degrees)	Total Cohesion (psf)	Total ϕ (degrees)
AD-198, 10 feet–12 feet	CL	38	0	37.4	0	23.9
ADUC-202, 5 feet–7 feet	CL	42	409.0	27.9	227.5	20.2
ADU-206, 5 feet–7 feet	CL	40	0	22.4	0	17.8
ADU-208, 1 foot–3 feet	CL	45	123.8	32.0	0	20.2
Average			133.2	29.9	56.9	20.5
Standard Deviation			192.9	6.3	113.8	2.5
Average minus one standard deviation			0*	23.6	0*	18.0

*negative value (see 6.9.2 and 6.9.4 for additional discussion)

Table 6.3. Consolidated Undrained Test Results – Borrow Site (Remolded).

Sample	USCS	Liquid Limit	Effective Cohesion (psf)	Effective ϕ (degrees)
AD-300, bulk	CL w sand	46	288.0	30.0
AD-301, bulk	CL	45	288.0	30.0

6.7.5 Unconsolidated-Undrained Triaxial Shear Strength

Table 6.4 provides UU triaxial shear strength test results on three samples. There was not enough usable sample for ADU-204 to run more than two tests.

Table 6.4. Unconsolidated-Undrained Test Results.

Sample	Confining Pressure (psi)	Cohesion (psf)	ϕ (degrees)
ADU-197, 10 feet–12 feet	7	1,237.0	0
	14	1,504.8	0
	28	2,494.1	0
	Average	1,745.3	0
ADU-204, 2 feet–4 feet	2.8	1,995.8	0
	10.4	2,766.2	0
	Average	2,381.0	0
ADU-207, 5 feet–7 feet	4.2	1,042.6	0
	9.7	1,478.9	0
	20.8	1,778.4	0
	Average	1,433.3	0
Average of Averages		1,853	0
Standard Deviation		483	0
Average minus one standard deviation		1,370	0

6.7.6 Consolidation Testing

Table 6.5 provides consolidation test results on five samples. The deeper sample had a smaller Overconsolidation Ratio (OCR) and larger compression index C_c, recompression index C_r, and initial void ratio e_o. The other four samples and the overall average were in a relatively close range. As C_c and C_r are in the numerator for calculation of consolidation settlement, it is conservative to add the standard deviation to the average. As e_o is in the denominator, it is conservative to subtract the standard deviation from the average.

Table 6.5. Consolidation Test Results.

Sample	USCS	Preconsolidation Pressure (tons-force per square foot [tsf])	Overburden (tsf)	OCR	C_c	C_r	e_o
ADU-197, 5 feet–7 feet	Gravelly CL	3.5	0.39	8.9	0.11	0.01	0.503
AD-198, 10 feet–12 feet	CL	3.2	0.88	3.6	0.23	0.03	0.876
ADU-204, 2 feet–4 feet	GC w sand	2.9	0.18	16.1	0.15	0.02	0.639
ADU-206, 5 feet–7 feet	CL w sand	4.1	0.37	11.1	0.16	0.03	0.667
ADU-208, 1 foot–3 feet	CL	2.3	0.12	19.2	0.11	0.01	0.791
Average					0.15	0.02	0.695
Average plus Std Deviation for C_c and C_r, Average minus Std. Deviation for e_o					0.20	0.03	0.57

6.7.7 Rock Core Compressive Strength

Table 6.6 provides rock core compressive strength test results for four samples.

Table 6.6. Rock Core Compressive Strength.

Sample	Elevation (feet)	Specimen Type	Compressive Strength (pounds per square inch [psi])
ADUC-196, 28.3 feet–28.7 feet	882.2–881.8	Shale	60
ADUC-202, 12.2 feet–12.6 feet	900.8–900.4	Limestone	12,560
ADUC-205, 15.1 feet–15.5 feet	913.7–913.3	Limestone	8,540
ADUC-205, 18.4 feet–18.8 feet	910.4–910.0	Shale	40

6.8 Subsurface Conditions

The levee site is generally comprised of undocumented fills underlain by alluvial lean clays. The alluvial soils also include silty lean clays, gravelly lean clays, and occasional clays with sand. Fat clay was only encountered in one sample. The lower portion of the alluvial lean clays, immediately above the bedrock surface, contains clayey gravel deposits. The clayey gravel is water bearing.

Fill soils contained occasional gravel and trace amounts of asphalt, brick, or concrete fragments but appeared to mainly be composed of lean clay. Petroleum odor was encountered at two locations, as noted in Section 6.3. Although the transition is difficult to pinpoint due to similar appearances where there is no obvious extraneous material, fill soils appear to extend approximately 5 to 10 feet below grade. Fill soils had medium stiff consistency on average in the soil borings performed for this investigation.

Natural alluvial soils also had medium stiff consistency on average.

Unlike most levee system locations in the Kansas City District, a continuous pervious zone of foundation sand in between the impervious blanket and bedrock was not encountered in the soil borings.

Subsurface rock was classified as weathered to competent shale and occasional limestone, which ranged from soft to moderately hard. Weathered shale was encountered at depths ranging from as shallow as 10.3 feet to as deep as 25 feet. Competent shale was encountered as shallow as 21.2 feet and as deep as 37 feet. Competent limestone was encountered as shallow as 8.7 feet and as deep as 13 feet. Soft limestone was only encountered in one boring location, at a depth of 18.5 feet.

6.9 Engineering Soil Property Development for Design

Design soil parameters used for underseepage and slope stability analyses are based on boring logs, laboratory testing, and typical properties based on soil classification.

6.9.1 Underseepage Design Soil Parameters

Table 6.7 shows design soil parameters for underseepage analysis.

Table 6.7. Underseepage Design Soil Parameters.

Property	Report Design Basis
Saturated Unit Weight of Impervious Blanket γ_{sat}	115 pcf
Permeability Ratio, Sand foundation material to impervious blanket (K_f/K_b)	400
Hydraulic Conductivity (k_{sat}) of Sand	0.003 feet/second

NWK typically uses saturated unit weight of 115 pcf for foundation materials. Shelby tube density tests show 115 pcf to be on the lower end of test results, but it is believed to be representative for the clayey fill and natural soils which do not contain gravel.

The permeability ratio of silty clay blanket material to a pervious substratum is given as 400 in the Kansas City District Underseepage Guidance.

Hydraulic conductivity values for sand are based on typical values for saturated soils (see Das, Geotechnical, p. 164). This value was used in feasibility analyses and design for the South Topeka Floodwall Modification Project. Assumed values were compared to permeability of pervious sand foundation for Missouri River Levee L-476, where 0.0026 feet/second (227 feet/day) was used (see DM No. 1 for 476-L, p. I-15).

6.9.2 Steady State Stability Design Soil Parameters

Effective (drained) soil strengths from CU testing were used for selecting steady state stability design soil parameters. Due to sampling difficulties, only four sets of tests were performed. The four resulting effective internal friction angles (22.4, 27.9, 32.0, and 37.4 degrees) were variable. Due to sample disturbance and gravel in samples, only two of the four test sets had reasonable stress-strain curves for all three samples tested at various normal stresses in each test set. These two test sets had effective results with internal friction angles and cohesion, 27.9 degrees and 409 psf for one and 32 degrees and 123 psf for the other.

As lab results were limited, the friction angle was also confirmed through correlations with Plasticity Index (PI) in UFC 3-220-10 (see Figures 8-13 through 8-15, Change 1, 11 March 2025). The fully softened friction angle derived from the correlations is taken to be equal to the normally consolidated peak value. Based on five consolidation test results on samples ranging from 1 to 12 feet below grade, the onsite soils are overconsolidated (overconsolidation ratios of 3.6 to 19.2). Normally consolidated strengths should be conservative for overconsolidated soils.

The average PI from 25 jar samples tested in 2023 (excluding four non-plastic samples) was 21.9. These correlations (in Figure 6.3, Figure 6.4, and Figure 6.5) show the average correlation for this PI results in a drained friction angle of 30 degrees and the average minus one standard deviation results in a drained friction angle of 28 degrees.

Figure 6.3. PI – Phi Correlation 1.

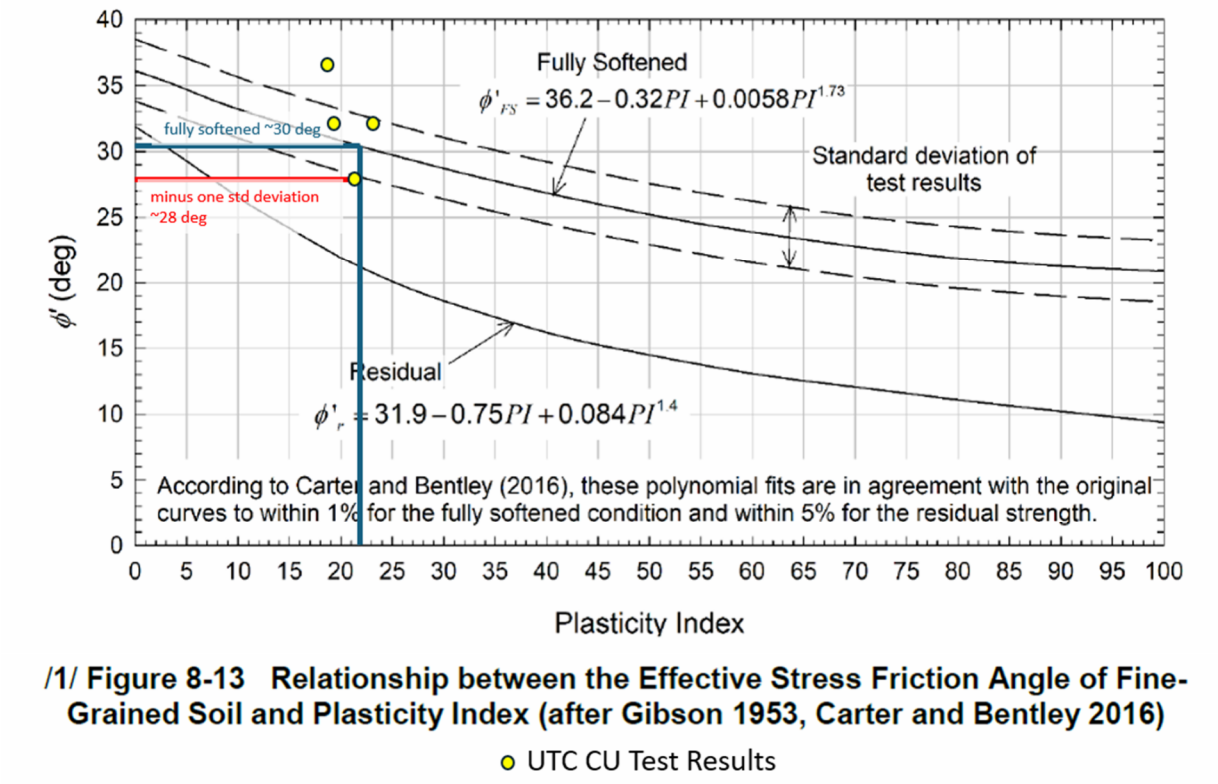


Figure 6.4. PI-Phi Correlation 2

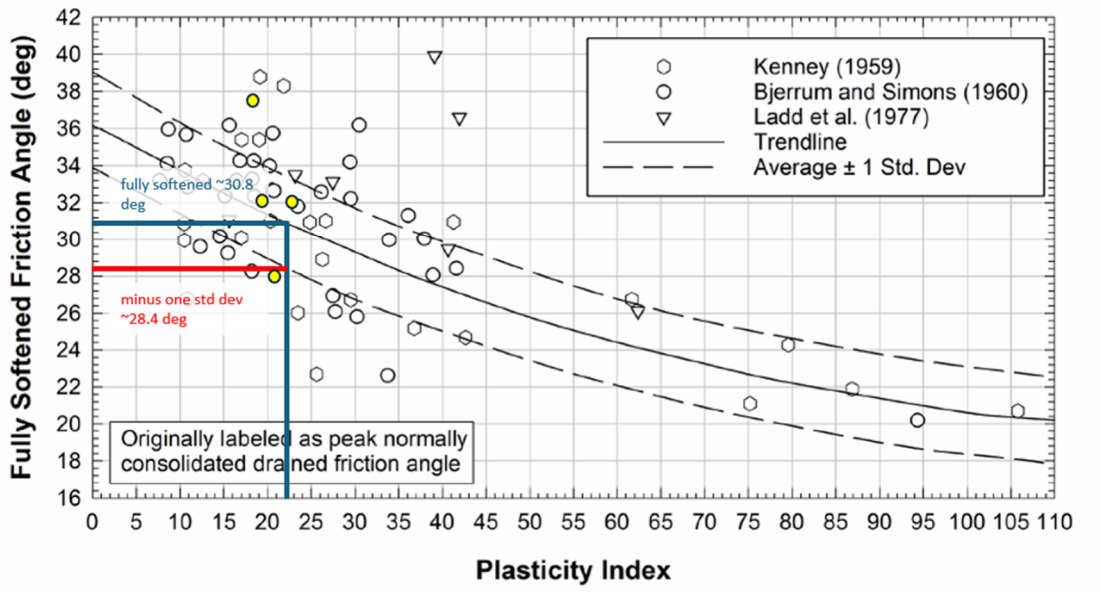


Figure 6.5. PI-Phi Correlation 3

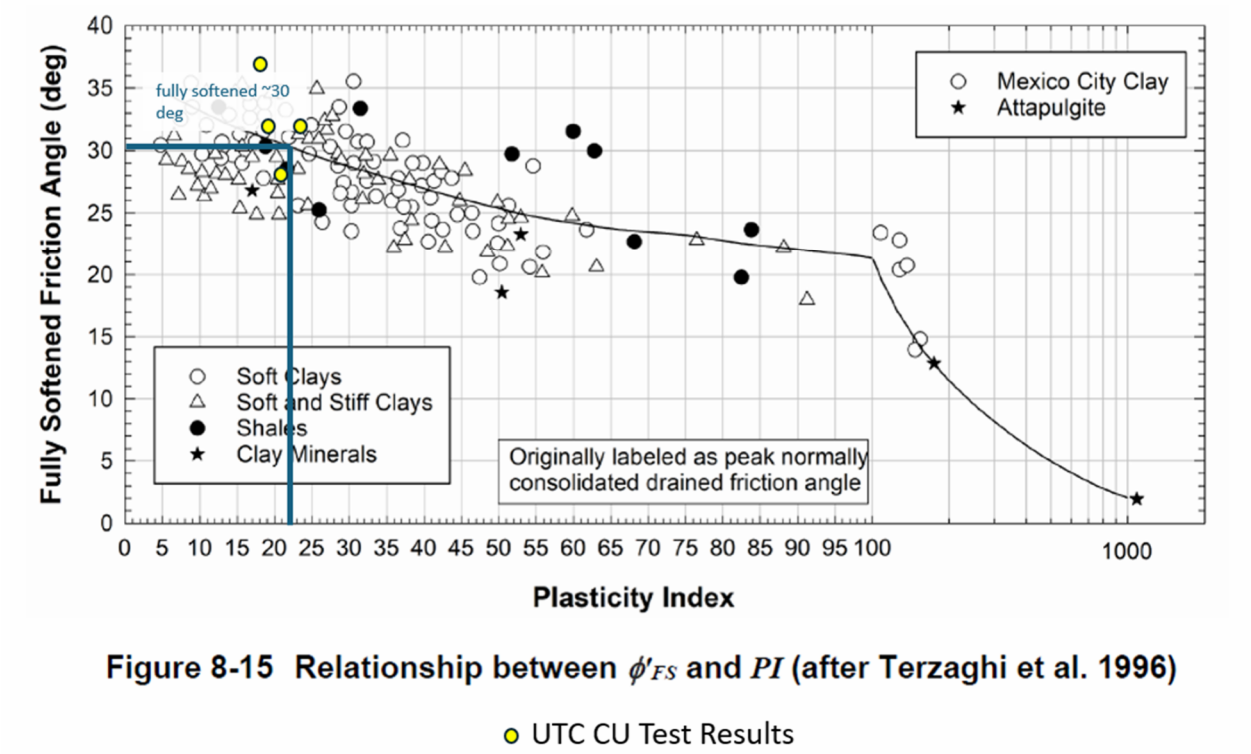


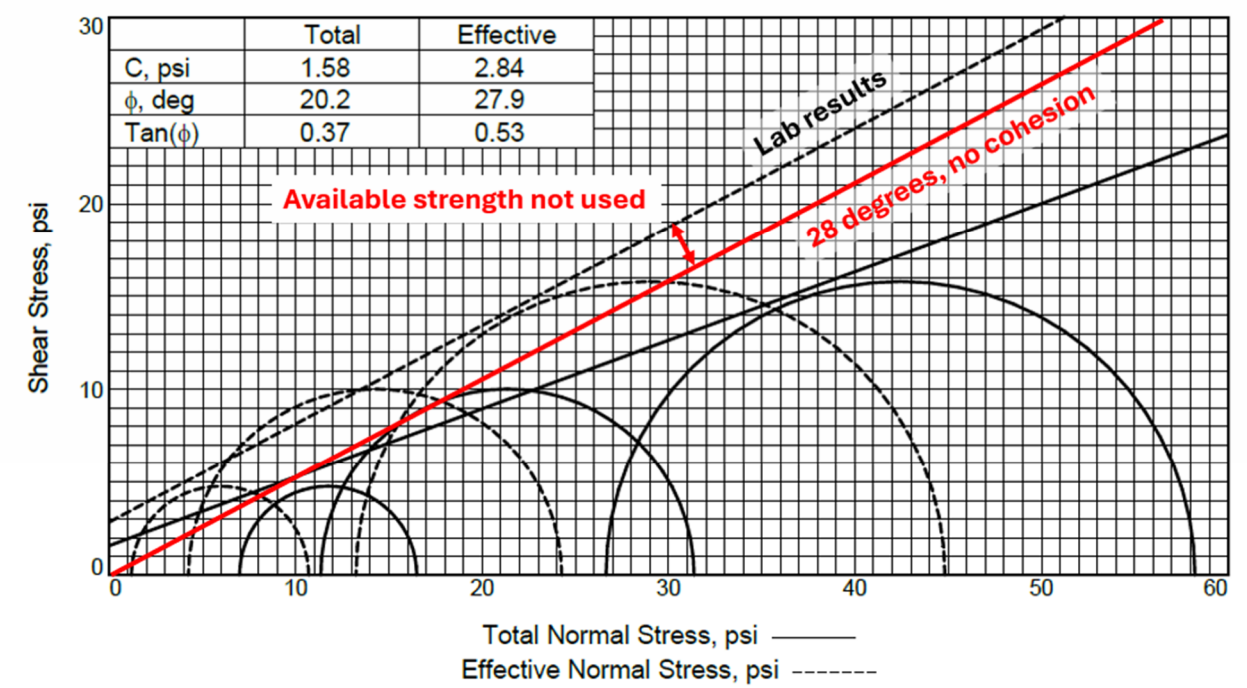
Figure 8-15 Relationship between ϕ'_{FS} and PI (after Terzaghi et al. 1996)

For drained strength parameters for structural stability calculations, I-wall rotational stability, drilled shaft calculations, and for the first stage of rapid drawdown, a drained friction angle of 28 degrees with no cohesion was conservatively used.

For steady state global slope stability calculations, using 28 degrees with no cohesion was resulting in unacceptable factors of safety for design. The Kansas City District typically does not use cohesion for drained strength parameters, but using an internal friction angle of 28 degrees with no cohesion does not allow the analysis to use all of the strength available from the test results. Using 28 degrees with no cohesion for steady state analyses was resulting in stability berms being required for a levee that is unlikely to see sustained loading. See

Figure ***6.6*** for the CU test result with the lower internal friction angle (from Boring ADU-202 from 5 to 7 feet below grade).

Figure 6.6. Lab Results vs No Cohesion Plot



The GeoStudio software used for slope stability analyses allows the use of a normal-shear strength function to define the drained shear strength. A curve was defined following the lab results at higher normal stresses and roughly following the curve of the first Mohr's circle back to a cohesion intercept of zero for lower stresses.

The normal-shear points were defined as shown and then adapted to a curvilinear function in GeoStudio (see Figure 6.7). This allowed for use of more of the strength from the laboratory test result while still using a cohesion intercept of zero.

Figure 6.7. Drained Strength Function

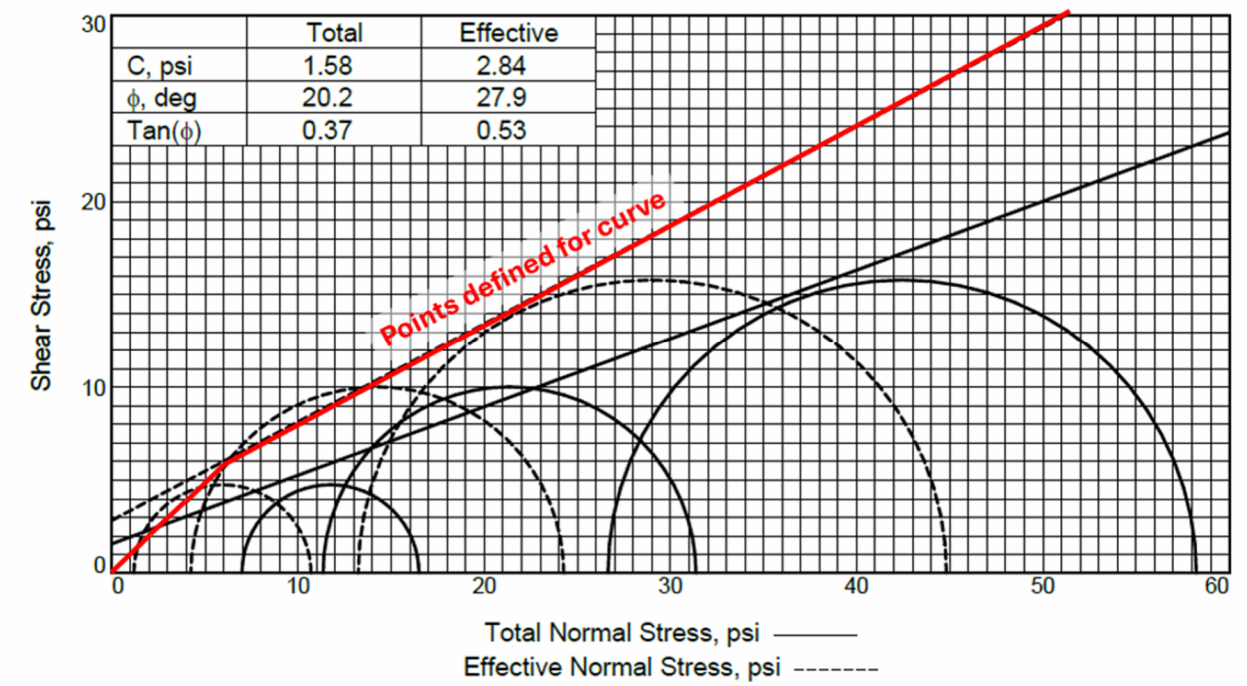
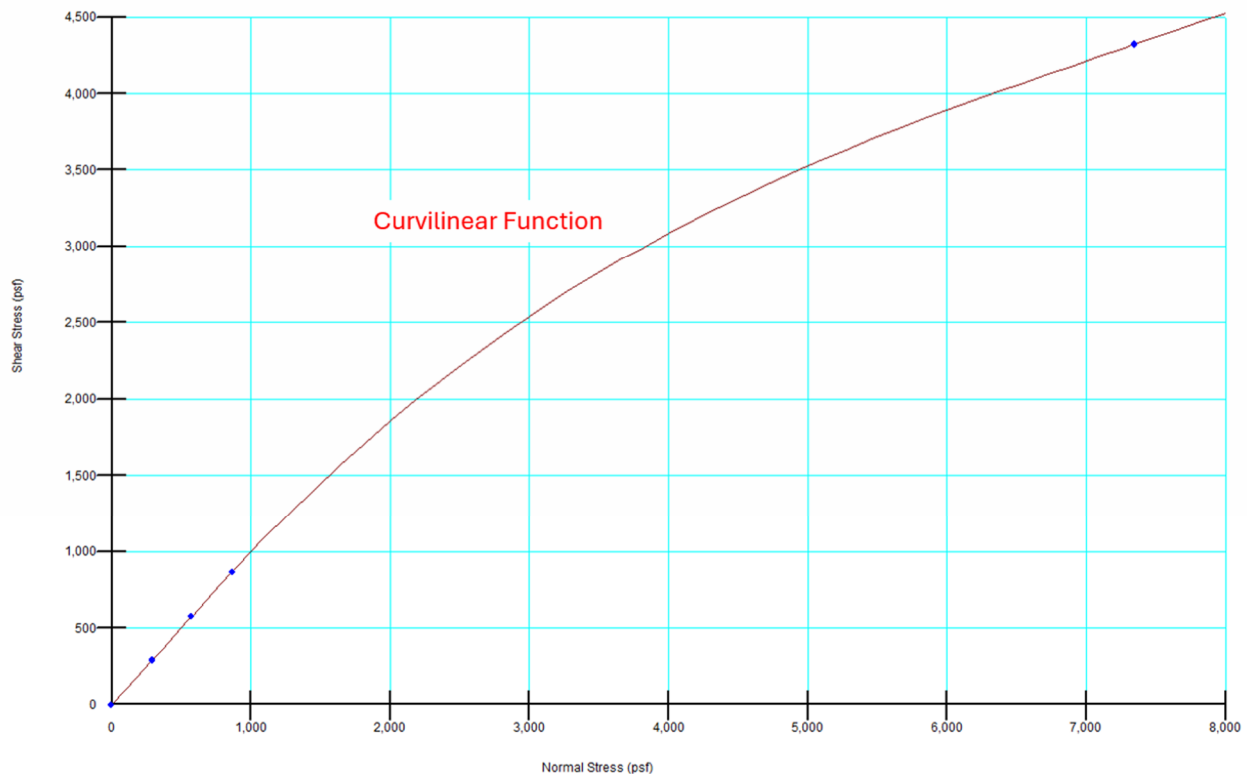


Figure 6.8. Curvilinear Strength Function.



Soil properties for compacted fill in the embankment for steady state use the same curvilinear function as their strength is expected to be no worse than that of the in-situ materials.

Remolded CU triaxial shear strength tests were performed on bulk samples from the potential borrow site at W 50th Terrace. These samples were compacted to match the specified compaction for embankment materials (95 percent of the standard Proctor maximum dry density). Both bulk samples had an effective friction angle of 30 degrees. An effective friction angle of 28 degrees was selected for use for the embankment. It is unknown what borrow sources will be used, but the remolded CU tests show this friction angle is reasonable for the type of soil and the degree of compaction that would be used for the embankment.

Although a continuous pervious zone of foundation sand was not encountered in the soil borings, a pervious stratum has been included in underseepage and stability analyses, as described in Section 7.1.4. Properties are based on typical properties of foundation sands at other levee design projects such as the Argentine Levee Raise in Kansas City, Kansas.

The curvilinear function shown above was used for drained strengths of impervious fill and blanket for slope stability calculations in GeoStudio. For other applications, including drained bearing capacity, the strengths in Table 6.8 were used.

Table 6.8. Steady State/Drained Material Properties.

Material	Unit Weight (pcf)	Cohesion (pcf)	Phi (degrees)
Impervious Fill (Embankment)	120	0	28
Impervious Blanket	115	0	28
Pervious Stratum	115	0	31

6.9.3 End of Construction Stability/Undrained Bearing Capacity Design Soil Parameters

Soil strength properties for end of construction stability or undrained bearing capacity are based on the undrained soil strengths from UU testing (see Table 6.9). Due to sampling difficulties, only three sets of tests were performed. The average cohesion minus one standard deviation was 1,370 psf (see Table 6.4). Based on the limited number of tests, a cohesion of 1,000 psf was chosen for design for both the embankment and impervious blanket.

Table 6.9. End of Construction/Undrained Material Properties.

Material	Unit Weight (pcf)	Cohesion (pcf)	Phi (degrees)
Impervious Fill (Embankment)	120	1000	0
Impervious Blanket	115	1000	0
Pervious Stratum	115	0	31

6.9.4 Rapid Drawdown Stability Design Parameters

Table 6.11 shows soil strengths for rapid drawdown stability. The effective stress plots of the CU triaxial shear strength testing with pore pressure measurements were used for selection of drained shear strengths for in-situ soils for steady state analysis/first stage calculations (Basic

Strength on SLOPE/W). The total stress plots of the same test were used for selection of undrained shear strengths for second stage calculations (R Envelope on SLOPE/W). The average minus one standard deviation for R Envelope friction angle was 18 degrees (see Table 6.2). Only one of the four test results showed cohesion. As noted above, due to sample disturbance and gravel in samples, only two of the four test sets had reasonable stress-strain curves for all three samples tested at various normal stresses in each test set. One of these showed a cohesion of 227.5 psf.

The analysis method for rapid drawdown requires R Envelope cohesion to be more than the cohesion used for the first stage/steady state calculations. A cohesion value of a few hundred psf is typically used for the R Envelope for rapid drawdown at Kansas City District projects based on more extensive testing on similar materials, see Table 6.10. An R Envelope cohesion of 200 psf was selected for design.

Table 6.10. Rapid Drawdown Design Parameters – Nearby Projects.

Material	Unit Weight (pcf)	Basic Cohesion (pcf)	Basic Phi (degrees)	R Envelope Cohesion (pcf)	R Envelope Phi (degrees)
Argentine Impervious Fill (Embankment)	120	0	26	300	17
Argentine Impervious Blanket	115	0	26	300	17
Armourdale/CID MO CL Fill	120	0	32	150	22
Armourdale/CID MO Alluvial ML	110	0	29	850	18
Armourdale/CID MO Alluvial CL	115	0	31	300	20
Armourdale/CID MO Alluvial CH	120	0	23	600	11
Average				416	17.5

Table 6.11. Rapid Drawdown Material Properties.

Material	Unit Weight (pcf)	Basic Cohesion (pcf)	Basic Phi (degrees)	R Envelope Cohesion (pcf)	R Envelope Phi (degrees)
Impervious Fill (Embankment)	120	0	28	200	18
Impervious Blanket	115	0	28	200	18
Foundation Sand	115	0	31		

6.9.5 Settlement Parameters

The average values from consolidation testing with one standard deviation in the conservative direction were used for settlement calculations (see Table 6.12). See lab results in Table 6.5.

Table 6.12. Consolidation Parameters.

C_c	C_r	e_o
0.20	0.03	0.57

The design overconsolidation ratio (OCR) was varied by depth incorporating the consolidation testing results from this project and from similar alluvial clay soils at the Argentine, CID Missouri, and Armourdale levee raises nearby along the Kansas River (see Figure 6.9, test results in Table 6.13, and larger plot of the figure in Appendix B.5).

As the Upper Turkey Creek alluvial clay soils had similar behavior to the alluvial clay soils at the other projects at shallow depths, a power trendline was created for all of the test results to develop design OCR vs. Depth for shallow and deeper depths (see Table 6.13).

Figure 6.9. OCR vs Depth Trendline.

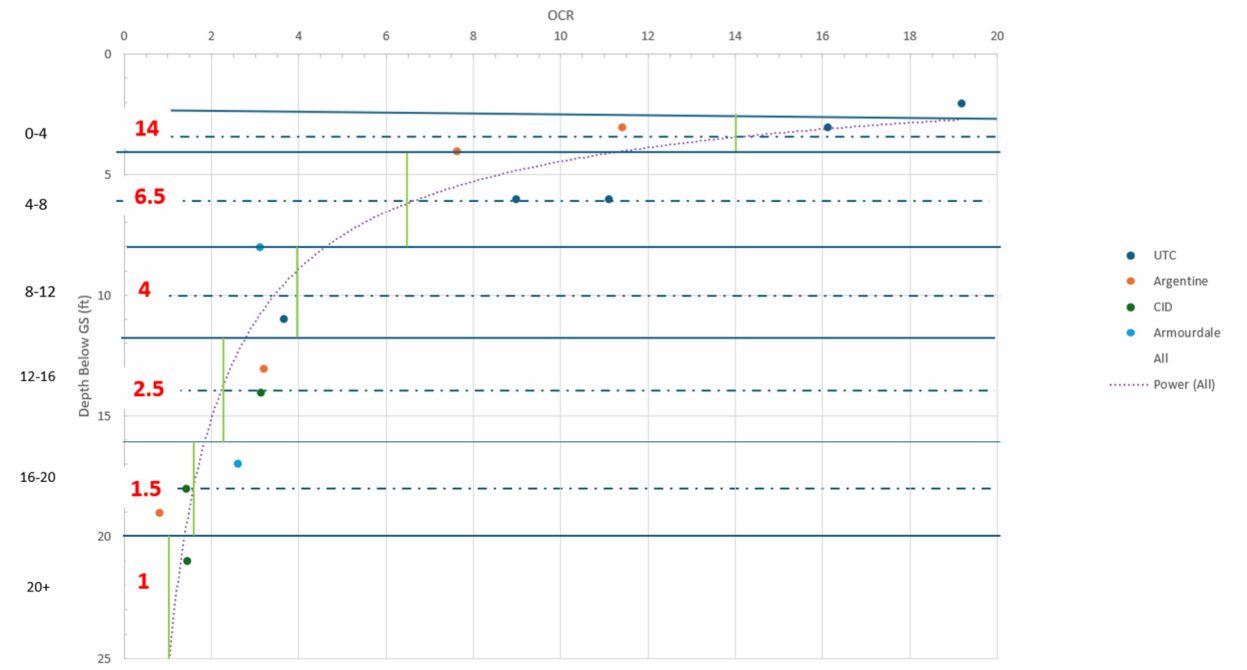


Table 6.13. Design OCR vs. Depth.

Depth below Grade (feet)	OCR
0-4	14
4-8	6.5
8-12	4
12-16	2.5
16-20	1.5
20+	1

6.9.6 Seismic Parameters

A soil seismic site class of D was used for initial assumptions for floodwall design. This corresponds to a shear wave velocity for medium dense sand or stiff clay in the top 100 feet. Depth of bedrock varied from 6.5 to 27.8 feet below grade, with an average depth of 14 feet and an average depth plus one standard deviation of 20 feet. The upper soils generally had medium stiff consistency. Based on shallow bedrock, using site class D with an average shear wave velocity corresponding to a stiff clay is satisfactory.

7.0. GEOTECHINCAL

Part 7 discusses levee embankment design and geotechnical aspects of floodwall design, including underseepage control, global slope stability, settlement, and slope protection. Local stability of floodwalls, such as sliding and overturning, and structural considerations for floodwalls are included in Part 9.

The levee system stationing used in this section refers to the August 2025 alignment provided by the Kansas City District Engineering Division, Geotechnical Branch, Civil Design Section (EDG-C). The levee alignment and stationing are constantly being adjusted during design. The river centerline stationing is also provided as an alternative means of designating locations in the levee system. This stationing is expected to remain consistent through the design project while levee stationing may continue to change.

In addition, the levee and floodwall cross sections analyzed for underseepage and slope stability are based on the August 2025 alignment. Portions of the left bank system have since been removed to improve interior drainage as part of Refinement 4. Other levee and floodwall top elevations and locations have had numerous small adjustments and will continue to be refined. The cross sections analyzed based on the August 2025 alignment are considered representative of the levee and floodwall heights, geometries, and soil conditions in the current design.

7.1. Underseepage

The following sections discuss underseepage criteria, design methodology, and recommended improvements.

7.1.1. Design Criteria

The underseepage requirements currently used in Pre-construction Engineering and Design (PED) work in the Kansas City District are contained in a memorandum dated 8 April 2015 with subject, “Missouri River Levee System (MRLS) R471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria; Revised per USACE-HQ Comments.” This memorandum was reviewed and approved by the Northwestern Division and USACE Headquarters for use in PED projects in the Kansas City District. The Northwestern Division also approved the memorandum for use as *Regional Criteria*, to be applied to design of underseepage berms for other projects in Northwestern Division districts until revisions to EM 1110-2-1913 are completed. Appendix B contains the full document.

Significant requirements in the memorandum include:

- Vertical Gradient Factor of Safety (FS) of 1.6 required at landside levee toe with water at the top of levee, as opposed to feasibility requirement of FS of 1.1 with water at top of levee and FS of 1.5 with water at design surface, three feet below top of levee
- Vertical Gradient FS of 1.0 required at berm toe as opposed to feasibility requirement of 1.1 beyond the berm toe
- Horizontal gradient ratio of 1.6 required at underseepage berm toe
- Generally recommended minimum berm width of 150 feet for new berms

Horizontal seepage gradient, or creep ratio, relates to the likelihood of sand boils at the levee or berm toe piping backwards to create a conduit beneath the levee. The critical horizontal gradient is considered to be 1/5 for fine sands and silts, typical of the Missouri River floodplain, as discussed in the referenced criteria. For earthen levees, the length of creep is defined as the distance from the effective seepage entrance to the first exit point for seepage; the landward toe

of the underseepage berm. This horizontal seepage gradient requirement is factored into the minimum required berm width. Table 7.1 summarizes the requirements.

Table 7.1. Underseepage Criteria.

Design Issue	Northwestern Division 2015 Criteria[1]
Levee Landside Toe Vertical Underseepage Gradient FS (for feature design)	$FS_{i\text{-}v}$>1.6, water at levee top
Underseepage Berm Toe Vertical Underseepage Gradient FS	$FS_{i\text{-}v}$>1.0, water at levee top
Underseepage Berm Toe Horizontal Underseepage Gradient[2] Ratio	$i_{h\text{-}c}/i_h$>1.6, water at levee top
Minimum Berm Thickness—Levee Toe	3 feet
Minimum Berm Thickness—Berm Crown[3]	1 foot
Berm Thickness Increase for Shrinkage and Consolidation	Adjust to account for settlement[4]
Berm Slope for Surface Drainage	Grade for positive drainage away from levee

[1]Requirements taken from Table 6 in referenced memorandum

[2]From referenced criteria: $i_h=H/(X1+L2+W)$

[3]Berm crown is the toe of the berm at the full required width. From this point (where the berm is 1-foot thick) the berm should slope down at a 1V:3H slope to existing grade.

[4]In practice, a 25 percent increase is used if no consolidation data is available.

The above criteria are generally applied to design of new levees or proposed levee modifications. Although the criteria were developed for Missouri River, the floodplain materials along Upper Turkey Creek are similar and the criteria may also be applied there.

7.1.2. Analysis Method

The general procedure outlined in EM 1110-2-1913, *Design and Construction of Levees – Appendix B.4,* (2000) was to calculate the FS for vertical underseepage gradient for the natural blanket and the excess head at the landside levee or floodwall toe. The design water surface for underseepage analysis is the top of levee or top of floodwall elevation. The excess head at the landside toe is assumed to be acting at the bottom of the blanket.

Variations from the manuals follow:

- Established hydraulic conductivity ratios for the Missouri River drainage basin (of which Upper Turkey Creek through the Kansas River is a tributary) relating to different blanket material types were used for underseepage calculations instead of horizontal and vertical hydraulic conductivity values for the pervious zone and impervious blanket zone material types, respectively. The hydraulic conductivity ratios were selected based on values from the Kansas City District Underseepage Guidance and the 2015 Regional Underseepage Criteria (see Appendix B). These values have long been established as appropriate for the floodplains within the Kansas City District. These values were developed from the 1962 Missouri River Division Conference in Omaha, Nebraska, based on experience with the Missouri River during the 1952 flood.

- The analysis assumes an infinite landside blanket unless a known exit point or seepage block is present.
- Rigorous blanket thickness transformation was generally not performed except to discount silty sand and coarser material below the blanket silts and clays. Sand lenses above the bottom of the blanket are included in the blanket thickness if they appear to be isolated from the lower pervious zones.
- If an underseepage berm is present or being designed, only half of the berm width is included in the L2 term when calculating FSi at the toe of the berm. This generally adds a small amount of conservatism to the calculations.

While underseepage berm requirements are provided here for reference, note that no underseepage berms are proposed for this project.

7.1.3. Design Assumptions

The design top of levee or floodwall is based on the profile provided by EDG-C.

The design water loading is to the top of levee or floodwall. While Upper Turkey Creek is not expected to remain at top of levee elevations for prolonged periods, this design was performed to bracket that worst case scenario.

Ground surface elevations are based on topographic survey by others provided by EDG-C. Elevations are based on the NAVD 88 vertical datum.

The subsurface profile is based on two historical soil borings (from 2002) and 17 recent soil borings (from 2023) performed within the project area.

The levee was divided into representative reaches for analysis based mainly on top of bedrock elevation (see Figure 7.1). Soil stratigraphy and site geometry, including distance from the levee system alignment to the creek, were relatively consistent throughout the system while depth to bedrock increased from upstream to downstream (south to north). In general, the cross section with the tallest levee or floodwall was analyzed within a particular reach. Bottom of blanket elevations and top of bedrock elevations for design cross sections are based on the nearest borings or interpolated between borings (see subsurface characterization in the Boring Profile in Appendix B.1).

Figure 7.1 Levee Design Reaches.



Table 7.2 provides the approximate reach boundaries with river centerline stationing.

Table 7.2 Levee Design Reaches.

Reach	UTC Centerline Stationing
1	0+00 to 17+60
2	17+60 to 29+50
3	29+50 to 38+60
4	38+60 to 57+80
5	57+80 to 66+00

The landside ground elevation was taken as the elevation at the landside levee toe or floodwall location. In general, the topography is relatively flat at the system alignment and then slopes up landward of the levee, especially on the left bank (west side of Turkey Creek). The riverside elevation is taken at the riverside levee toe or floodwall location and is generally the same as the landside ground elevation. The topography then slopes down to the creek bank.

The riverside seepage length (L1) was taken as the distance from the riverside levee toe or floodwall location to the creek bank. The soil borings did not show a consistent pervious layer between the impervious blanket and bedrock, so the seepage entrance was taken as the creek bank rather than a bottom of blanket elevation along the riverside slope.

The levee seepage length (L2) was taken as the levee width. A 10-foot crest width and 1V:3H side slopes were assumed. For floodwalls, L2 was taken as the footing width. I-wall areas within T-wall areas were not analyzed separately for underseepage.

A permeability ratio of 400 was used for ratio of hydraulic conductivity of the foundation sand to hydraulic conductivity of the impervious blanket. This corresponds to a typical value for a silty lean clay material in the Kansas City District underseepage guidance.

The soil stratigraphy along the proposed levee system alignment is different from most Kansas City District projects in that a continuous pervious layer or foundation sand was not encountered between the impervious blanket and bedrock. Bedrock is also relatively shallow. The blanket theory spreadsheet used for underseepage calculations in the Kansas City District and used to calibrate the seepage model for slope stability calculations in GeoStudio is based on the presence of a pervious zone to transmit seepage where underseepage pressures can build up at the bottom of the impervious blanket. Although the soil borings did not show a continuous obvious pervious zone, it is unconservative to assume that seepage will not be transmitted and underseepage pressures will not build up at the bottom of the impervious blanket. It is assumed that seepage will flow through voids and joints in the bedrock or through pervious zones of the upper weathered rock or gravelly clay.

As this means of seepage is difficult to quantify and bedrock in the blanket theory spreadsheet represents a relatively impervious layer, a pervious blanket was created between the impervious blanket and bedrock in the spreadsheet and the sensitivity of the results was checked based on varying the thickness of that pervious layer (varying the top of bedrock elevation). The top of bedrock elevation was adjusted in 10-foot increments until the change in FS for vertical underseepage gradient was less than 0.05. This adjusted worst-case, top of bedrock elevation was used in the underseepage analysis to account for any seepage pressures that could build up at the bottom of the impervious blanket. This adjusted top of bedrock elevation was also used in the slope stability models.

The top of bedrock elevation was adjusted for both the levee sections and floodwall sections.

7.1.4. Blanket Theory Analysis and Recommendations

Table 7.3 shows underseepage analysis results for various reaches.

Table 7.3. Underseepage Analysis.

Station	Protection	Vert. Gradient Factor of Safety (1.6 required)
Left Bank		
Reach 1		
13+20/UTC 0+50	Tallest Floodwall	**0.63**
26+80/UTC 14+50	Tallest Levee	2.61
Reach 2		
41+40/UTC 28+50	Tallest Levee	1.74
Reach 3		
44+40/UTC 31+80	Tallest Levee	1.66
44+80/UTC 32+40	Tallest Floodwall	1.55
Reach 4		
51+80/UTC 40+00	Tallest Floodwall	3.69
Reach 5		
76+60/UTC ~61+00	Tallest Levee	2.66
78+20/UTC 63+10	Tallest Floodwall	2.27
Right Bank		
Reach 1		
6+80/UTC 15+25	Tallest Floodwall	**0.95**
Reach 2		
18+20/UTC 26+80	Tallest Levee	**0.69**
19+00/UTC 27+60	Tallest Floodwall	**1.30**
Reach 3		
28+40/UTC 36+75	Tallest Floodwall	**0.68**
Reach 4		
33+80/UTC 41+75	Tallest Floodwall	1.70
41+40/UTC 49+50	Tallest Levee	1.73
Reach 5		
52+60/UTC 61+60	Tallest Floodwall	2.02

Although the adjusted top of bedrock elevation for worst case seepage results in low factors of safety for vertical underseepage gradient in some areas, the duration of loading above the levee toe is approximately two hours for a 1% AEP event or an overtopping event (see Figure 4.8). For infrequent short duration loadings, the levee foundation soils are unlikely to develop backward erosion piping. In addition, the soil profile generally consisted of nonerodable clay soils to bedrock; continuous foundation sand layers that are needed to form a pipe were not observed in the borings performed for this investigation. Underseepage berms and relief wells are not recommended.

An inspection trench will be excavated along levee centerline for all levee reaches. For levees 6 feet or higher, the trench will be 6 feet deep. For shorter levees, the depth of the trench will be

equal to the levee height. The trench width should be approximately 6 feet to allow for compaction equipment.

The inspection trench will allow for detection of any continuous pervious zones, voids, or unacceptable fill material. The trench will be backfilled with the excavated material compacted in lifts. If unacceptable material is encountered, it will be replaced with impervious fill to ensure there are no continuous seepage paths.

An inspection trench is not recommended for floodwall areas. Construction of the floodwall footing and key will require excavation to similar depths, allowing for inspection of the upper soils.

7.1.5. Floodwall Toe Drain

A toe drain will provide a filtered exit to prevent internal erosion of the floodwall subgrade soils from a concentrated leak at the footing-soil interface. The floodwall toe drain will consist of a 15-inch diameter perforated pipe wrapped in geotextile with coarse and fine aggregate. The drain will be routed to discharge through drainage structures or through standpipes at the ground surface. The toe drain will be used in T-wall areas but not in I-wall areas.

7.1.6. Internal Erosion in I-Wall Areas

Internal erosion in I-wall areas was evaluated by considering line of creep seepage along the sheet pile. This method evaluates whether the seepage path is long enough that internal erosion in that material would be unlikely to progress from the ground surface at the landside toe of the I-wall back to the ground surface at the riverside toe or to a vertical crack on the riverside of the tip of the sheet pile. A spreadsheet was used to determine Lane's weighted creep ratio for both the case with no crack and case with vertical crack on the waterside. Results are provided in Appendix B.

Best Practices in Dam and Levee Safety Risk Analysis states "To assess the likelihood of progression of backward erosion piping, the creep ratio for the reservoir level under consideration is compared to the minimum (or safe) creep ratio for the piping material. Progression of backward erosion would be expected if the creep ratio is less than the minimum creep ratio" (see page D-6-E-5, Version 4.1, USACE, July 2019).

The design sheet pile depths for rotational stability satisfied the critical creep ratio for a medium clay, which represents the known in-situ soil conditions.

7.2. Slope Stability

Section 7.2 discusses slope stability criteria, design methodology, and recommended improvements.

7.2.1. Design Criteria

Slope stability criteria are contained in EM 1110-2-1913, April 2000, and EM 1110-2-1902, October 2003. The design water surface is assumed to be the top of levee or floodwall for steady seepage analysis. The water surface is also assumed to be at the top of the levee or floodwall for end of construction analysis and for the first stage of rapid drawdown. Per the referenced engineer manuals, the factors of safety and shear strength parameters shown in Table 7.4 are required.

Table 7.4. Slope Stability Criteria.

Criteria	End of Construction	Long-Term Steady Seepage	Rapid Drawdown
Required FS for New/Improved Levees	1.3	1.4	1.0–1.2
Shear Strength Free draining soils	Use drained strength	Use drained strength	Use drained strength
Shear Strength Low permeability soils	Use undrained strength	Use drained strength	1st stage: drained 2nd stage: undrained 3rd stage: the lower of the two

The end of construction case represents undrained conditions for impervious embankment and foundation soils; excess pore water pressure is present because the soil has not had time to drain since being loaded.

Steady seepage case occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated, and a condition of steady seepage occurs.

Rapid drawdown represents the condition when a prolonged flood stage saturates at least the major part of the upstream embankment and then falls faster than the soil can drain. This causes excess pore pressure to develop, which may result in the riverside slope becoming unstable.

Earthquake loadings are not normally considered in levee stability analysis because of the low probability of earthquakes coinciding with periods of high water (see Section 6-5d, EM 1110-2-1913 [2000]).

Upper Turkey Creek is a flashy stream, which historically has rarely had water outside the channel for prolonged periods of time. Larger historical events such as the 1998 flood, however, demonstrate that more significant flooding is possible. For this design, it was assumed for the worst-case scenario that water could remain at the top of levee or floodwall long enough for steady seepage conditions to develop.

7.2.2. Method of Analysis

Slope stability of the levee embankment and floodwall reaches was evaluated for selected representative cross sections from those used in underseepage calculations. Those cross sections were then applied to similar reaches elsewhere in the levee system. Slope stability was analyzed using Seequent's GeoStudio SLOPE/W program using Spencer's method.

Cross sections were developed based on levee alignments and surveyed site topography provided by EDG-C. Landside and riverside levee slopes of 1V:3H were used for design.

A levee crest width of 10 feet was used for design. It is understood that the City of Merriam, Kansas, may desire to have larger crest widths in certain areas to allow for more room on the trail for bicyclists and pedestrians. Sensitivity analyses showed that larger crest widths do not have negative impacts on slope stability.

Seepage analysis was performed on GeoStudio's SEEP/W program to provide pore water pressure conditions for use in the stability analysis in SLOPE/W. In general, a water total head boundary condition equal to the top of the levee or floodwall elevation was applied to the

riverward slope and riverward edge of the model and a water total head equal to the landside ground surface elevation was applied to the landward edge of the model. In some cross sections where the landward ground surface slopes up significantly, even exceeding the top of levee elevation, the ground surface elevation at the landside levee toe was used for the water total head boundary condition at the landward edge of the model.

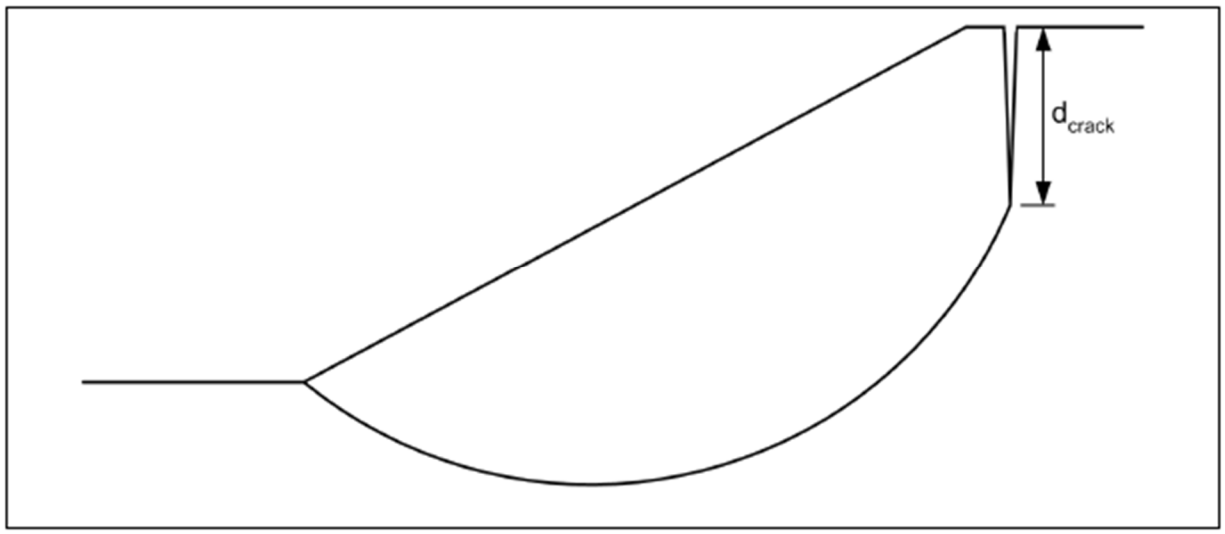
A potential seepage face boundary condition with water rate equal to 0 cfs was applied to the landside levee slope and landside ground surface and a no-flow boundary was placed at top of bedrock elevation with water rate equal to 0 cfs. The riverward extent of the model was taken as halfway into Upper Turkey Creek (approximately 25 feet). The landward extent of the model was taken as 2,000 feet landward of levee centerline, at which point there appeared to be no influence on the calculation results.

Hydraulic conductivity of 0.003 feet/second was used for foundation sand. A permeability ratio of 400 was used in the underseepage blanket theory spreadsheet calculations. This equates to a hydraulic conductivity of 7.5×10^{-6} feet/second for blanket materials and impervious embankment fill, using 0.003 feet/second for sand. The hydraulic conductivity of the embankment fill and impervious blanket was varied until the water total head at the bottom of blanket at the levee toe in the SEEP/W model matched the blanket theory calculations.

Critical slip surfaces were determined for a global stability breach with slip surface entry for landside slope failures on the riverside slope and slip surface including the entire crest. Slip surfaces not including the entire crest are considered maintenance breaches, which could be repaired before leading to levee breach. Federal levees are monitored frequently during high water events by both the local levee sponsor and USACE. Small slope breaches on the landside slope have occurred on Kansas City District levees after large rain events or during high water events in the past and emergency repairs or permanent repairs have been made before the slide progressed significantly. Multiple slides would need to occur in succession, enlarging the breach surface from an initial landside slope breach, while the river level remained high, before levee breach could occur.

Vertical tension cracks were added near the upper end of slip surfaces to eliminate tensile stresses from analyses (see Figure 7.1 from Figure C-22 in EM 1110-2-1902). When soils at the crest of a slope have cohesion, as in the end of construction case where undrained strengths are used, the analysis method results in negative values for the normal and shear forces, implying that the soils have tensile strength. Because few soils have tensile strength that can be relied on for slope stability, tensile stresses were eliminated from the analyses by adding water-filled tension cracks. Crack depths were estimated based on the undrained soil strength and then adjusted to not be greater than the height of the embankment and to not result in lower factors of safety. Tension cracks were added to the end of construction analyses. As a function was used for drained strength in the steady state analyses and cohesion was used in the second stage R Envelope strengths for rapid drawdown, steady state and rapid drawdown cases were spot-checked for negative normal and shear forces in the slip surface slices by viewing column information in GeoStudio. No negative normal or shear forces were found, so tension cracks were not added to those analyses.

Figure 7.2 Vertical Tension Crack.

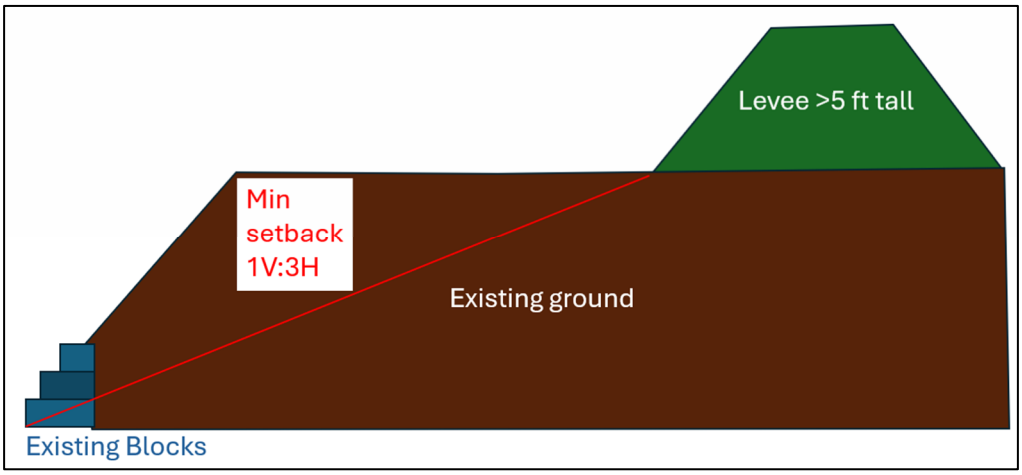


7.2.3. Analysis Results and Recommendations

Steady seepage, end of construction, and rapid drawdown stability were evaluated for representative levee and floodwall cross sections.

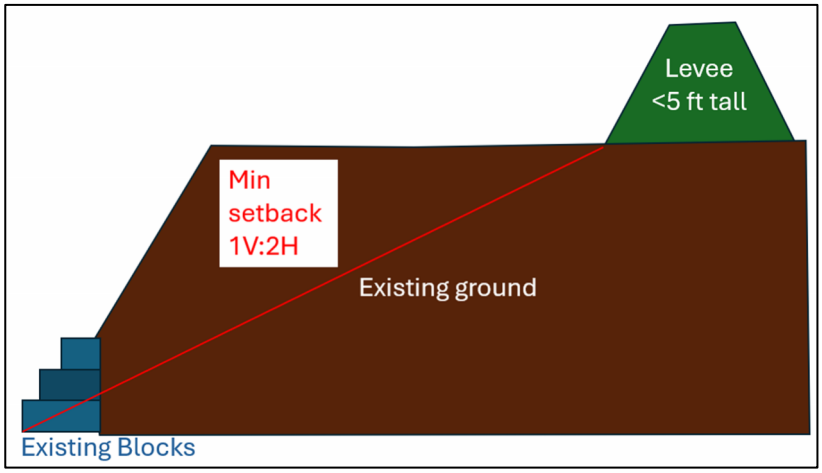
In general, the levee alignment was set so that a 1V:3H projected slope from the riverside levee toe would not intersect the face of the channel slope (limestone block wall or creek bank) above bottom of channel (**Error! Reference source not found.**).

Figure 7.3. General Levee Minimum Setback.



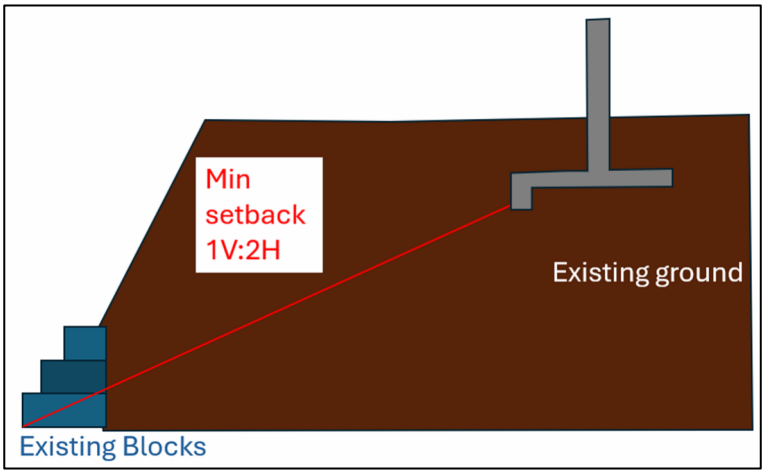
For levees less than or equal to 5 feet tall, the projected slope from the riverside levee toe to the face of the channel slope was allowed to be as steep as 1V:2H (Figure 7.4).

Figure 7.4. Minimum Setback for Levees Less than 5 feet Tall.



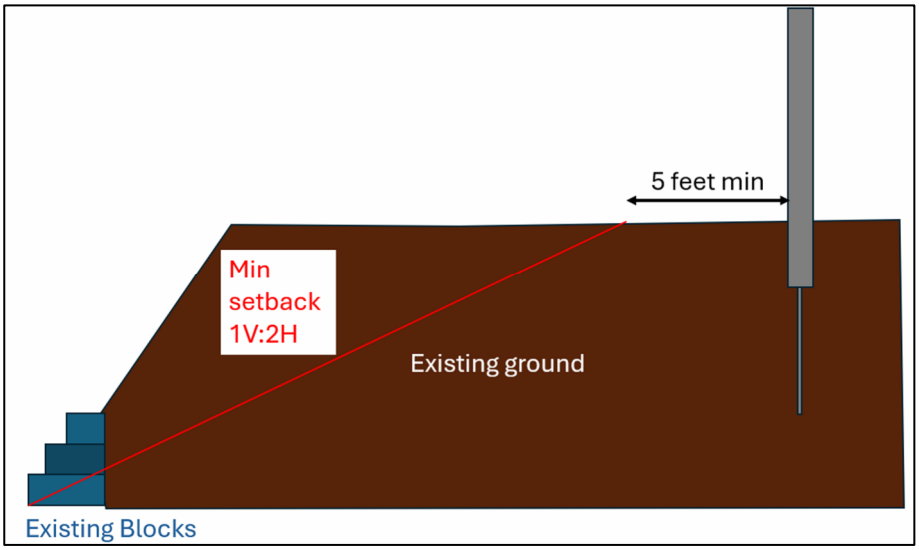
T-wall alignment was set so that a 1V:2H projected slope from the footing key would not intercept the face of the channel slope (Figure 7.5).

Figure 7.5. T-wall Minimum Setback.



I-wall alignments were set so that a 1V:2H projected slope starting at the ground surface 5 feet riverward of the I-wall would not intercept the face of the channel slope (Figure 7.6).

Figure 7.6. I-wall Minimum Setback.



The levee and floodwall setbacks provide acceptable factors of safety for steady state seepage, end of construction, and rapid drawdown. In addition, they provide a buffer in case of scour, failure of the limestone block channel wall, or failure of the channel slope. The analysis results show that multiple flood events with no repairs would be required for the levee or floodwall to have a global slope stability failure. The setback would need to be reduced to approximately 1V:1.5H or steeper slope from the face of the channel slope to the levee; then another flood would be required to produce a rapid drawdown situation. Even then, the failure would occur at a time of low water, reducing the risk of flood damage in the leveed area.

7.2.3.1. Steady State Seepage

Table 7.5 summarizes the reaches analyzed for steady state stability. All levee reaches analyzed met the minimum required factor of safety of 1.4 for a setback of 1V:3H. Some levee reaches also met the required factor of safety at reduced setbacks.

No realistic failure surface could be determined for the floodwall areas based on the soil properties, shallow bedrock, and no slope down on the landside. The failure surface intercepted shallow bedrock. A conservative landside slope was added to a cross section with deeper bedrock (RB R4 F 33+80/UTC 41+75), and a feasible failure surface was obtained with a high factor of safety (1.7). This confirms that the floodwall areas have acceptable factors of safety for global slope stability. The resisting forces on the landside with no slope down are too much for a credible failure surface to be generated. Block-specified failure surfaces were generated, as well as circular failure surfaces, and results were similar.

No realistic failure surface could be determined when steepening the slope from the floodwall footing to the creek bank to 1V:1.5H or 1V:1H.

A setback of 1V:2H was selected for design for floodwalls.

Table 7.5. Steady State Stability Results.

Reach	Levee/ Wall Height (feet)	Steady State (Drained) Stability FS (1.4 Required)		
		Setback		
		1V:3H	1V:2H	1V:1.5H
Left Bank				
R1 F 13+20/UTC 0+50	11.1	NS	NS	NS
R1 L 26+80/UTC 14+50	4.8	2.7	2.7	2.4
R3 L 44+40/UTC 31+80	7.4	1.9	1.9	1.8
Right Bank				
R2 L 18+20/UTC 26+80	7.9	1.4	1.4	1.3
R4 F 33+80/UTC 41+75	10.1	NS	NS	NS
R4 L 41+40/UTC 49+50	10.1	1.9	1.9	1.8

NS - no solution

7.2.3.2. End of Construction

Table 7.6 summarizes the reaches analyzed for end of construction (with water-filled tension cracks). All levee reaches met the minimum factor of safety of 1.3, including at the reduced setbacks.

No realistic failure surface could be determined for the floodwall area at R4 F 33+80/UTC 41+75 based on the soil properties, shallow bedrock, and no slope down on the landside, even with tension cracks. When introducing an artificial landward slope, stability results could be generated. For the other floodwall area, a failure surface was generated for the case with

tension crack. Block-specified failure surfaces were generated, as well as circular failure surfaces, and results were similar. Based on high factor of safety or error messages and lack of sensitivity to the slope in the other runs, floodwall stability runs with reduced slopes were not generated.

Table 7.6. End of Construction Stability Results.

Reach	Levee/ Wall Height (feet)	End of Construction (Undrained) Stability FS (1.3 Required)		
		Setback		
		1V:3H	1V:2H	1V:1.5H
Left Bank				
R1 F 13+20/UTC 0+50	11.1	3.2		
R1 L 26+80/UTC 14+50	4.8	7.1	7.0	6.9
R3 L 44+40/UTC 31+80	7.4	4.5	4.5	4.5
Right Bank				
R2 L 18+20/UTC 26+80	7.9	3.3	3.4	3.4
R4 F 33+80/UTC 41+75	10.1	NS		
R4 L 41+40/UTC 49+50	10.1	4.1	4.1	4.1

NS - no solution

7.2.3.3. Rapid Drawdown

Table 7.7 summarizes the reaches analyzed for rapid drawdown. The levee results show factors of safety near the 1.0 minimum when the setback is reduced to 1V:1.5H.

No realistic circular failure surface could be determined for floodwall areas based on the soil properties and shallow bedrock. Block-specified failure surfaces were generated, and factors of safety were satisfactory. Based on high factor of safety or error messages and lack of sensitivity to the slope in the other runs, floodwall stability runs with reduced slopes were not generated.

Table 7.7. Rapid Drawdown Stability Results.

Reach	Levee/ Wall Height (feet)	Rapid Drawdown Stability FS (1.0 Required)		
		Setback		
		1V:3H	1V:2H	1V:1.5H
Left Bank				
R1 F 13+20/UTC 0+50	11.1	NS		
R1 L 26+80/UTC 14+50	4.8	1.7	1.4	1.2
R3 L 44+40/UTC 31+80	7.4	1.9	1.5	1.2

Reach	Levee/ Wall Height (feet)	Rapid Drawdown Stability FS (1.0 Required)		
		Setback		
		1V:3H	1V:2H	1V:1.5H
Right Bank				
R2 L 18+20/UTC 26+80	7.9	1.8	1.7	1.6
R4 F 33+80/UTC 41+75	10.1	2.9		
R4 L 41+40/UTC 49+50	10.1	1.5	1.2	1.0

NS - no solution

7.3. I-Wall Analysis

I-walls were considered as a cost-effective alternative to T-walls in areas where lower floodwall heights were sufficient and available right-of-way was limited between existing structures and the creek channel. The required penetration depth for rotational stability was the controlling factor and was used to determine where I-walls were feasible.

7.3.1. Rotational Stability

The USACE CASE program, CWALSHT, was used to analyze rotational stability for varying I-wall heights and establish the necessary sheet pile penetration depth. A maximum I-wall height of seven feet was determined per EM 1110-2-2502. Analyses were completed for wall heights from two to seven feet in half-foot increments. The analysis employed the same soil properties used in the T-wall and levee designs to maintain consistency. A bank slope of 1H:1.5V was used in all analyses to reflect the steepest bank slope existing along the alignment. Undrained and drained cases were considered. Early analysis indicated the drained case was the controlling case, as undrained conditions resulted in significantly shallower required penetration depths. An undrained example is included in Table 7.8 for comparison with the drained case for a 7-foot wall.

A sensitivity analysis was conducted to assess the influence of various input parameters. This included varying bedrock depths, evaluating scenarios with and without a pervious soil layer, and adjusting the distance between the wall and the streambank. Results indicated variations in bedrock depth influenced penetration depths the most. The model frequently encountered computational limitations when bedrock depth was reduced below a certain threshold. Removing the bedrock generally resulted in the most conservative penetration depths. Profile sensitivity was also examined by maintaining constant parameters while modifying the distance from the wall to the top of bank. This analysis concluded that the model was generally insensitive to these profile changes - doubling the distance from 10 to 20 feet reduced penetration depths by less than 0.5 foot on average.

Based on the comprehensive results of the CWALSHT analysis, a required sheet pile penetration depth was selected for each I-wall height, as summarized in Table 7.8.

Table 7.8. Required I-Wall Sheet Pile Penetration Depth for Varying Subsurface Conditions (ft).

I-Wall Height Above GS (ft)	No Pervious Layer		Pervious Layer at 15-20 ft BGS		No Bedrock (Continuous Clay Layer)		Selected Sheet Pile Penetration Depth
	Bedrock at 20 ft	Bedrock at 15 ft	Wall 10 ft from Bank	Wall 20 ft from Bank	Wall 10 ft from Bank	Wall 20 ft from Bank	
0.5	E	E	E	E	E	E	3
1.0	E	E	E	E	E	E	4
1.5	E	E	E	E	E	E	5
2	6.4	E	6.2	E	6.4	5.6	6
2.5	E	E	E	7.0	8.0	7.0	7
3	E	E	E	8.4	9.6	8.4	9
3.5	11.2	11.0	11.0	10.8	11.2	11.1	11
4	12.7	E	12.7	12.6	12.7	12.6	13
4.5	E	14.2	E	13.5	E	E	14
5	15.9	14.5	14.8	14.5	15.9	16.0	15
5.5	17.2	E	E	E	17.2	16.8	16
6	18.7	15.7	18.1	16.1	18.7	18.0	17
6.5	E	17.1	19.5	17.6	20.2	19.6	18
7	E (7.4)	18.0 (7.4)	E (E)	18.8 (E)	21.6 (7.4)	21.2 (5.1)	20

Note: "E" indicates the model encountered computational limitations and did not provide a solution. Undrained results for a 7-foot wall are in parentheses.

Wall heights below 2 feet produce errors in CWALSHT, used a linear projection (slope of 2) to select penetrations for 0.5, 1.0, and 1.5 ft wall heights.

7.3.2. Selection of I-Wall Ranges

Feasible I-wall ranges were determined based on the available penetration depths (existing ground surface to top of estimated bedrock). The flood protection alignment was provided by EDG-C and included the existing ground surface elevation, the proposed protection elevation, and type of protection (levee, floodwall, or none) for 20-foot segments. Bedrock depths were interpolated along the flood protection alignment based on available boring data, with separate interpolations performed for the left and right banks. Where borings were spaced more than approximately 1000 feet apart, the creek bed elevation was used to estimate bedrock depths. The height of protection above ground was determined from the provided survey data and rounded to the nearest 0.5 foot. The required penetration depth (per Table 7.8) was then assigned to each station based on the height of the protection (I-wall above-ground height). I-walls were recommended where the required penetration depth was equal to at least the

available penetration depth plus a one-foot buffer. Sections where available penetration was one to three feet less than the required penetration were identified as potentially I-wall suitable with the incorporation of grading, excavation, or a combination of the two. Floodwall sections with heights greater than seven feet were not considered. Recommended I-wall ranges are provided in Appendix B.

Critical sections within the floodwall ranges where I-walls were recommended were further analyzed to confirm required penetration depths were reasonable. Critical sections were selected based on wall heights and distances to the streambank—taller walls closer to the bank were prioritized. Separate CWALSHT analyses were completed for each critical section. The ground surface was modeled based on the lidar survey.

Additional sections were analyzed where I-walls were potentially suitable (as defined above). In these cases, the ground surface profile was modified to incorporate up to three feet of added fill at the base of the I-wall. Analysis of these reaches indicates that with the addition of fill, I-walls may be used. Further coordination with the Civil group would be needed to verify regrading is feasible in order to incorporate these I-wall reaches. Thus, I-walls are not shown in the current plans for these potentially suitable reaches.

Prior to I-wall construction, field verification of bedrock depths should be conducted.

7.4. Bearing Capacity for Floodwalls

Bearing capacity for floodwall footings was analyzed for both drained case using the design drained angle of internal friction of 28 degrees and for the undrained case using the design undrained cohesion of 1000 psf (see Appendix B.4), with the undrained case controlling. The floodwall footings were analyzed as continuous footings 7 to 20 feet wide with a spreadsheet using Terzaghi and Vesic methods. Length to width ratios for the floodwall footings exceed 5, a standard ratio for considering footings to be continuous. Floodwall segments generally run for several hundred feet; the shorter segments are for shorter walls with smaller footing widths. The design bearing capacity is 2000 psf.

7.5. Settlement

Section 7.5 discusses settlement criteria, design methodology, and recommendations.

7.5.1. Design Criteria

EM 1110-2-1913 states evaluation of the amount of post-construction settlement that can occur from consolidation of the embankment and foundation may be important if the settlement would result in loss of freeboard of the levee or damage to structures in the embankment.

The EM states many districts overbuild a levee or berm by a given percent to take into account anticipated settlement both of the foundation and within the levee or berm fill itself. The EM references overbuild of five to ten percent for semi-compacted fill, which is the anticipated degree of compaction for berms.

7.5.2. Consolidation Testing

Consolidation tests (ASTM D2435) were performed on undisturbed samples of foundation soils as part of the planned laboratory testing program. Average consolidation parameters with one standard deviation in the conservative direction were used for design, as discussed in Part 6. New compacted embankment soils were assumed to be relatively incompressible.

7.5.3. Levee and Floodwall Settlement

The maximum consolidation settlement of the clay subgrade soils due to construction of the new levee is expected to be around 3 inches. For predicted levee settlement of less than 6 inches, the Kansas City District does not typically recommend an overbuild.

The maximum consolidation settlement of the clay subgrade soils due to construction of the new floodwall is expected to be around 1 inch. Differential settlement between adjacent monoliths is expected to be less than ½ inch. Based on conversations with EDD-S about the concrete wall and flexibility of the water stop between monoliths, a maximum settlement of one inch is considered acceptable.

7.5.4. Pipe Settlement

Settlement of drainage structure pipes will be evaluated at a future submittal.

7.5.5. Recommendations

No overbuilds or mitigation are recommended for settlement.

7.6. Riverside Slope Protection

This section discusses riverside slope protection for the Upper Turkey Creek FRM Project.

7.6.1. Literature Review

Literature review to be completed for a future submittal.

Future Reference: K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\3.0 Project Data & Reference Info\3.4 Background Docs\Historical reports

7.6.2. Method of Analysis

The method of analysis was discussed previously in Section 4.7.

7.6.3. Criteria

Multiple criteria with respect to slope protection can be considered applicable to the Upper Turkey Creek Levee. Regarding the resilience of potentially erodible levee embankment materials, Table 2-5 of EM 1110-2-1601 provides permissible channel velocities for given surface materials and soil types and is presented in Figure 7.7. From this table, a mean velocity of 5 to 8 ft/sec would be assumed to represent an approximate threshold of resiliency for both the current and proposed levee raise, given the riverward soils and maintained sod cover. EM 1110-2-1601 also recommends a W50, weight at which 50% of the stone protection material is less than, (minimum) of 80 pounds for locations vulnerable to theft and/or vandalism. With the urban location of the levee, this recommendation is considered relevant to the project. Given the recommended minimum W50 value, the corresponding D50 value according to Table 3-1 (Figure 7.8 in this document) in EM 1110-2-1601, for a specific weight of 155 pcf, is approximately 12 inches, based upon the relationship D50=D30(D85/D15)0.33, presented in Section 3.7.b. of EM 1110-2-1601, where D85/D15 is assumed to be equal to 1.7. The corresponding D100 value according to Table 3-1 in EM 1110-2-1601, for a specific weight of 155 pcf, and 165 pcf is 21 inches.

EM 1110-2-1601 presents standardized gradations in Table 3-1 having a relatively narrow range in sizes (D85/D15 of 1.4 to 2.2); whereas the stability coefficient for incipient breach for Equation 3-3 in EM 1110-2-1601 is for a D85/D15 range of 1.7 to 5.2. A value of 1.7 was selected due to its placement within the documented range of values. This relationship was used to correlate the D30 (min) value presented in EM 1110-2-1601 Table 3-1 with the D50

(minimum), and corresponding W50 (minimum) value of 80 pounds recommended to address the urban placement of riprap.

Figure 7.7. Table 2-5 excerpted from EM 1110-2-1601.

Table 2-5
Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined Earth (slopes less than 5%)[2]	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

Figure 7.8. Table 3-1 excerpted from EM 1110-2-1601.

Table 3-1
Gradations for Riprap Placement in the Dry, Low-Turbulence Zones

D_{100}(max)	Limits of Stone Weight, lb[1], for Percent Lighter by Weight						D_{30}(min)	D_{90}(min)	
	100		50		15				
in.	Max	Min	Max[2]	Min	Max[2]	Min	ft	ft	
Specific Weight = 155 pcf									
9	34	14	10	7	5	2	0.37	0.53	*
12	81	32	24	16	12	5	0.48	0.70	
15	159	63	47	32	23	10	0.61	0.88	
18	274	110	81	55	41	17	0.73	1.06	
21	435	174	129	87	64	27	0.85	1.23	
24	649	260	192	130	96	41	0.97	1.40	
27	924	370	274	185	137	58	1.10	1.59	
30	1,268	507	376	254	188	79	1.22	1.77	
33	1,688	675	500	338	250	105	1.34	1.94	
36	2,191	877	649	438	325	137	1.46	2.11	
42	3,480	1,392	1,031	696	516	217	1.70	2.47	
48	5,194	2,078	1,539	1,039	769	325	1.95	2.82	
54	7,396	2,958	2,191	1,479	1,096	462	2.19	3.17	
Specific Weight = 165 pcf									
9	36	15	11	7	5	2	0.37	0.53	*
12	86	35	26	17	13	5	0.48	0.70	
15	169	67	50	34	25	11	0.61	0.88	
18	292	117	86	58	43	18	0.73	1.06	
21	463	185	137	93	69	29	0.85	1.23	
24	691	276	205	138	102	43	0.97	1.40	
27	984	394	292	197	146	62	1.10	1.59	
30	1,350	540	400	270	200	84	1.22	1.77	
33	1,797	719	532	359	266	112	1.34	1.96	
36	2,331	933	691	467	346	146	1.46	2.11	
42	3,704	1,482	1,098	741	549	232	1.70	2.47	
48	5,529	2,212	1,638	1,106	819	346	1.95	2.82	
54	7,873	3,149	2,335	1,575	1,168	492	2.19	3.17	
Specific Weight = 175 pcf									
9	39	15	11	8	6	2	0.37	0.53	*
12	92	37	27	18	14	5	0.48	0.70	
15	179	72	53	36	27	11	0.61	0.88	
18	309	124	92	62	46	19	0.73	1.06	
21	491	196	146	98	73	31	0.85	1.23	
24	733	293	217	147	109	46	0.97	1.40	
27	1,044	417	309	209	155	65	1.10	1.59	
30	1,432	573	424	286	212	89	1.22	1.77	
33	1,906	762	565	381	282	119	1.34	1.94	
36	2,474	990	733	495	367	155	1.46	2.11	
42	3,929	1,571	1,164	786	582	246	1.70	2.47	
48	5,864	2,346	1,738	1,173	869	367	1.95	2.82	
54	8,350	3,340	2,474	1,670	1,237	522	2.19	3.17	

Notes:
1. Stone weight limit data from ETL 1110-2-120 (HQUSACE, 1971 (14 May), "Additional Guidance for Riprap Channel Protection, Ch 1," US Government Printing Office, Washington, DC). Relationship between diameter and weight is based on the shape of a sphere.
2. The maximum limits at the W_{50} and W_{15} sizes can be increased as in the Lower Mississippi Valley Division Standardized Gradations shown in Appendix F.

7.6.4. Analysis Results

Discussion regarding the analysis results and a summary table of the recommended maximum riprap diameters can be found in Section 4.7.2.

7.6.5. Recommended Riprap Gradations

Riprap gradations for the minimum D100 sizes are provided in Table 7.9 based on Table 3-1 in EM 1110-2-1601 (1994) for specific weights of 155 and 165 pcf. Table 7.7.10 and Table 7.11

Table 7.9. Recommended Riprap Weights.

Approx. D100 (in)	W100 (lb)	W50 (lb)	W15 (lb)
36	877-2331	438-691	137-346
33	675-1797	338-532	105-266
21	174-463	87-137	27-69

Table 7.7.10. Recommended Maximum sizing for left bank.

Approx. UTC STA		Approx. Left Bank STA		Length (ft)	Initial D100 (in)	Recommended D100 (in)
Start	End	Start	End			
0+00	9+00	13+00	22+38	938	21	21
9+00	14+00	22+38	26+50	412	33	33
15+25	19+00	28+20	31+60	340	15	21
21+00	25+00	34+00	37+60	360	15	21
33+00	36+00	45+40	48+45	305	15	21
63+00	65+75	78+40	80+76	236	24	33

Table 7.11. Recommended Maximum sizing for right bank.

Approx. UTC STA		Approx. Right Bank STA		Length (ft)	Initial D100 (in)	Recommended D100 (in)
Start	End	Start	End			
15+25	19+00	6+80	10+47	367	15	21
19+00	26+00	10+47	17+60	713	15	21
30+00	33+00	21+40	24+75	335	30	33
33+00	48+00	24+75	40+00	1525	21	21
53+00	57+00	44+80	48+12	412	21	21
57+00	60+00	48+12	51+30	318	33	33
60+00	64+00	51+30	54+60	320	21	21

Approx. UTC STA		Approx. Right Bank STA		Length (ft)	Initial D100 (in)	Recommended D100 (in)
Start	End	Start	End			
64+00	65+75	54+60	57+78	318	42	36

7.6.6. Recommended Bedding Gradation and Thickness

No specific guidance for bedding design is available for levees. General requirements for dams were used. In general, for dams, the gradation of the bedding material should provide for retention of bedding particles by the overlying riprap layer and for retention of the material underlying the bedding layer. Bedding material should generally conform to Equations C-1 through C-3 from EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams (2004).

Figure 7.7.9. Bedding Gradation Criteria.

$$D_{15B} > 5D_{15E} \quad \text{(C-1)}$$

$$D_{15B} < 5D_{85E} \quad \text{(C-2)}$$

$$D_{85B} > D_{15R}/5 \quad \text{(C-3)}$$

where

D_{15B} = the 15 percent passing the size of the bedding

D_{85B} = the 85 percent passing the size of the bedding

D_{15E} = the 15 percent passing the size of the material to be protected

D_{85E} = the 85 percent passing the size of the material to be protected

D_{15R} = the 15 percent passing the size of the riprap

An intermediate filter layer may be required between the bedding and riprap to prevent washout of the bedding. Bedding layers over erosion-resistant clay materials need not be designed to meet the criteria of Equation C-1 or Equation C-2 but must still satisfy Equation C-3. Each design should produce a specification that defines material sources, gradations, and layer thickness to economically provide the riprap and bedding layers required to protect the embankment.

In addition, the bedding must not be so broadly graded as to be susceptible to internal instability. USBR guidance recommends bedding material be considered internally unstable when its coefficient of uniformity is less than 10 (Design Standards No. 13 Embankment Dams, Chapter 7 Riprap Slope Protection, Section 7.3.7.1, dated May 2014).

EM 1110-2-2300 states that bedding materials over erosion-resistant clay materials need not be designed to meet the criteria of Equation C-1 or C-2 but must still satisfy Equation C-3.

Bedding was sized to be retained by the maximum size riprap of 36 inches. A two-stage filter or geotextile for retaining clay-size embankment particles in the bedding layer is not recommended. The levee is not subject to frequent wave action. The gradient of flow is generally from river to riprap, to bedding, and then to embankment. In addition, the levee embankment will be erosion-resistant compacted homogenous impervious fill and levee and floodwall subgrade soils are generally composed of relatively nonerodable clay material. Bedding compatibility calculations are included in Appendix B.6.

It is recommended that bedding material meet the gradation requirements of Kansas Department of Transportation (KDOT) Stone for Filter Course, Type III Material (see KDOT Specification 1114 [2015]).

The recommended bedding gradation listed by percent finer is given in Table 7.12.

Table 7.12. Recommended Bedding Gradation.

Sieve/Size	Percent Finer
6” (152.4 mm)	100
5” (125 mm)	75-95
2” (50.8 mm)	40-60
3/8” (9.525 mm)	5-25

Based on local practice for levees in the Kansas City District and satisfactory performance history through several high-water events, bedding thickness will be set at six inches.

7.7. Levee System Overtopping Resilience

Overtopping resiliency measures will be considered in a future submittal

7.7.1. Earthen Levee Embankment Overtopping Resiliency

Turkey Creek is too flashy to effectively flood fight, so earthen levee embankment overtopping resiliency will be considered in a future submittal.

7.7.2. New Floodwall

Will be considered in a future submittal.

7.8. Levee Construction Materials

This section discusses levee construction materials.

7.8.1. Borrow Sites

See Part 8 for discussion of borrow material.

7.8.2. Fill Materials

On-site soils from required excavations, imported soils from provided borrow sites, or off-site borrow sites obtained by the contractor to be used as fill material shall be free of muck, frozen material, excess moisture, organic material, topsoil, rubbish, construction debris, chemical contamination, and rock larger than three inches. On-site soils from required excavations which are suspected by odor or appearance of petroleum contamination shall be handled in accordance with project specifications. Prior to placement in any excavation or embankment, fill material imported from provided borrow sites or borrow sites obtained by the contractor shall be verified to be free of chemical contamination by performing the environmental tests listed in the project specifications.

7.8.2.1. Impervious Fill

Impervious structural fill shall be used in the levee embankment and for backfill of any excavations within the levee critical area (within 500 feet of the landside toe and within 300 feet of the riverside toe of the levee or floodwall), except where otherwise specified. This includes backfill (other than pipe bedding) for installation or modification of drainage pipes and drainage structures. This material shall consist of satisfactory impervious material classified as lean clay (CL) or low plasticity silt (ML) in accordance with ASTM D2487. High plasticity clay (CH) and high plasticity silt (MH) are not suitable for backfill around structures due to their expansive properties. Sandy materials are also not acceptable in these areas as excavations for drainage structures extend into the natural blanket materials in areas where there is no filter zone. It is anticipated that impervious structural fill will be obtained from the required excavations or approved borrow sites.

7.8.2.2. General Construction Considerations

Grading areas should be cleared, grubbed, and stripped as outlined in the project specifications in preparation for fill placement. Areas to receive fill should be scarified to a depth of 6 inches and moisture conditioned. Levee and stability berm fill should be compacted by not less than four passes of compaction equipment suitable to the type of material being placed and shall be compacted to a minimum of 95 percent of the maximum dry density as determined by the contractor in accordance with ASTM D698.

8.0. BORROW AREAS

Soil fill for construction will be obtained from a combination of required excavations and offsite borrow sites.

The City of Merriam has identified two potential borrow sources for this project in Merriam, Kansas, one at Ikea Way and W 62nd Terrace in Merriam and one at W 50th Terrace near Grant Street. Subsurface investigation at the Ikea Way borrow site (which is unlikely to be available when this project resumes) is discussed in Appendix A. Subsurface investigation at the borrow site on W 50th Terrace is discussed in Part 6. Aerial photos and boring logs for both sites are included in Appendix A.

Construction of a multistory apartment building with a below grade parking garage is planned for the site on Ikea Way. Excavation at the site was planned to begin in December 2025 with any excess material stockpiled. The City of Merriam will inform USACE of how much material is stockpiled for use as borrow. As detailed in Appendix A, based on the visual classifications given in the boring logs and on the limited laboratory testing, the material is assumed to be suitable for use as impervious fill. Based on the current understanding of grading plans for the site, it is unclear if there will be a significant amount of borrow available from this site. It is unlikely that this borrow would be available long-term if this project is delayed.

No development is currently planned for the site on W 50th Terrace. This site is a relatively level open lawn area with trees on the outside. The boring logs indicated lean clay soils and gravelly fill. The gravelly fill is not suitable for use in levee embankments. The areal extent of the fill soil has not been determined. Laboratory testing and final boring logs are included in Appendix A.

If it is determined that additional or alternative borrow sites are required as the design progresses, USACE will work with the City of Merriam to identify additional sites. Otherwise, the construction contractor would need to find a commercial source.

8.1. Estimated Borrow Quantities

Based on the current design, roughly 15,000 – 20,000 cubic yards of levee fill is needed from another source for the project. This quantity assumes roughly half of the material excavated from the detention basin and any excess material from floodwall construction will be suitable material to use as levee fill.

This page was intentionally left blank for duplex printing.

9.0 STRUCTURES

9.1 General Design

This section presents the structural design criteria and assumptions for the Upper Turkey Creek flood management project, the primary function of which is flood protection in the Merriam, Kansas, area. Structures proposed for this project include floodwalls (soil-founded floodwalls$ and I-walls), potential retaining walls at properties of interest to the City of Merriam, a gatewell at the detention basin, and outfall structures and flapgates where stormwater conduits discharge into Turkey Creek.

9.1.1 Materials

9.1.1.1 Material Weights

Table 9-1. Design Material Unit Weights.

Material	Unit weight (pcf)
Water	62.4
Concrete	150
Steel	490
Aluminum	170
Riprap	130
Gravel	125

9.1.1.2 Material Design Strengths

9.1.1.2.1 Steel

Steel piles and sheet piles are ASTM A709 Grade 50. Structural steel shapes; Channels and angles are ASTM A36, HSS round and rectangular shapes are ASTM A500 and pipe sections are ASTM A53 Gr. B. For steel plates and bars, a minimum strength of ASTM A36 steel with a yield stress of 36 ksi is required. Where materials are required to have higher strengths, the drawings and specifications will clearly notate the changed ASTM strength designation for the material.

9.1.1.2.2 Aluminum

If aluminum is used for parts and components, or miscellaneous metals scope, it will be extruded shapes of either grade 6061 or 6063 T3 or T6 as applicable and as outlined by the drawings or specifications.

9.1.1.2.3 Stainless Steel

Stainless steel may be used for embedded plates in concrete. Stainless steel will be ASTM A240 grade 316 unless otherwise called out in the plans and specifications. Stainless steel grades and type are noted on the design drawings and in the specifications.

9.1.1.2.4 Reinforced Concrete

Unless noted otherwise in the drawings or specifications, concrete and rebar strengths used for design were f'_c=4,500 psi and f_y=60,000 psi. the reinforced concrete used will adhere to the requirements in EM 1110-2-2104 and EM 1110-2-2502 for rebar detailing and requirements for minimum and maximum areas of steel.

9.1.1.2.5 Existing Concrete

When information was not available for the design strengths for the existing concrete and rebar, the following values will be used for analysis: f'_c=3,000 psi and f_y=40,000 psi. The assumed existing concrete strengths are based on values in ASCE 41, *Seismic Evaluation and Retrofit of Existing Buildings*.

9.1.1.2.6 Soil Parameters

Table 9-2. Typical Soil Properties.

Property		Embankment	Foundation Soil
Saturated Unit Weight		120 pcf	115 pcf
Effective Unit Weight		57.6 pcf	52.6 pcf (buoyant weight)
Short Term/Undrained	Internal Friction Angle	0°	0°
	Cohesion	1,000 psf	1,000 psf
Long Term/Drained	Internal Friction Angle	28°	28°
	Cohesion	0 psf	0 psf
Allowable Bearing[1]		2,000 psf	

[1]FS=3.0 per ECB No. 2017-2. Includes water pressure influence due to flood loading. Refer to Appendix B for allowable bearing calculations.

9.2 Design Loads

9.2.1 Flood Loading

9.2.1.1 Design Flood Protection Requirements

Unless noted otherwise, floodwalls, gatewells, I-walls, conduits, bridge parapets included in flood protection, and outlet works structures were designed to the 100-year return period flood (0.01 AEP) elevation. The required 100-year (0.01 AEP) top of protection elevations were rounded up to the nearest inch. Tops of new floodwall stems do not have stepped elevations. Where needed, a single monolith with a sloped top accommodates the difference of about 6 inches in elevation from the required protection level and the top of the wall. Sloping transitions are made at least 2 ft from the end of the monolith section and maintain a 1 on 4 slope, the change in elevation is developed over a 2 foot run along the top length of the wall monolith. Sloping the protection elevation in lieu of a stepped transition at the joint was chosen so that during future inspections it could be readily observed if there were settlement issues between the two monoliths at the joint.

9.2.1.2 Hydraulic Loading Conditions

Three levels of performance were evaluated; serviceability, strength, and stability. Applicable load conditions for the flood protection were the *usual* and *unusual* load cases. The floodwalls are designed for a protection from the 100 year event (0.01 AEP); therefore, the Extreme loading conditions with a return period of 700 years do not apply to the design. The usual load flood loading applied to the structures is based on a WSE generated by a 10-year return period (0.1 AEP) flood event, and the unusual loading condition is generated by a flood event associated with the 100-year return (0.01 AEP), which is the top of protection pool.

The Serviceability loading condition considers the loading from the 100 year event as well. These loads are then scaled by a single load factor from Table 3-4 of EM 1110-2-2104 (2025 revision). EM 1110-2-2104 outlines design of Reinforced Concrete Hydraulic Structures (RCHS) using the Serviceability requirements as well as Strength design requirements. Reference attached calculations for the 12 foot tall T-wall that shows that the controlling loading for the T-wall systems will come from the Serviceability loading conditions. The remaining calculations for other T-Wall heights as well as the calculations for the I-wall materials are abridged to include only the Serviceability requirement checks since they govern the loading to the wall. The calculations package for the T-Walls shows that the serviceability checks will govern the design of the wall over the strength requirements, this is shown for the wall section that has the longest section of the stem that is exposed.

Stability limit states were used to satisfy stability requirements under drained and undrained loading conditions. Analyses are performed using unfactored loads to check stability failure modes described in EM 1110-2-2100 and to compute bearing pressure exerted on the soil as well as the uplift pressures on the foundation according to EM 1110-2-2906. The unfactored moments, shears, and thrusts on the foundation were then multiplied by the appropriate load factor to determine design forces used to establish the required section properties.

9.2.2 Impact Load (debris)

The inlet structure sizes which distribute water to this reach of Upper Turkey Creek will keep vegetation debris exposure to smaller sized logs. Mostly medium-sized, partial trees have the capacity to enter the waterway. It is anticipated that debris other than trees/logs will also be present, however within this reach of Turkey Creek, debris must enter the waterway by means of constricting culverts and concrete box culverts, so the size of debris is expected to be no larger than trees/logs. The current design of the left bank may have lengths of the alignment that are unprotected by levee or floodwall and thus would allow for debris to enter the channel. The area to the west of Turkey Creek is highly developed with minimal large vegetation. Likely only man made debris would enter the channel.

USACE criteria for debris loading is outlined in EM 1110-2-2502 in paragraph 6.12.3. the manual allows for a typical linear loading condition between 100 and 500 pounds per foot at the top of the flood protection level. For the T-walls of this project the Impact loading for the majority of the walls was limited to 100 pounds per foot at the top of the wall except where the angle of attack between the flow and the wall is steeper. This was applied at locations like the confluence of Turkey Creek and the tributary located at the upstream end of the project as well as the opposite bank of the Quaker Creek outfall. During the design, a hydraulic analysis was conducted and the flow vectors through this area show that even where the confluence of the tributary and Turkey Creek meet, the flows are still largely parallel to the face of the floodwalls. This analysis and additional descriptions are presented in the ‘Justification’ calculations provided at in the appendix of this report.

9.2.2.1 Debris for Faces Impeding Flow

The faces impeding flow include the upstream parapet faces on Johnson Drive, Merriam Drive Bridge structures, upstream faces of gatewells, and buttress structures perpendicular to the flow, which will support dam faces on walls above inlet structures which are parallel to the flow. The debris loads included in the design of these structures are point loads due to aberrant log impacts or a small uniform load of 500 pounds per linear foot (PLF).

The impact loading to impeding faces was chosen to be the maximum recommended loading from EM 1110-2-2502 where the water flow is perpendicular to the flow of water. This loading was compared to the loading calculated per ASCE 7-16 C5.4.5, *Impact Loads*, assuming 1,000 pounds contribute to the impact load. This is equivalent to a 12-inch diameter, 20-foot-long log, which is believed to be representative of approximately the largest log or tree expected in the river during a flood. This force considers either stopping the full impulse momentum of a 1,000-pound log hitting perpendicular to the structure or a portion of a larger log that hits at an angle. The velocities of the water flowing parallel to the channel were taken based on the HEC-RAS output at each specific structure's location. The resulting point load or the general line load is comparable to the suggested 500 pounds per linear foot from the Engineering Manual and thus the suggested loading is applied at the static WSE as a companion to the primary hydrostatic flood load.

9.2.2.1.1 Debris for Faces of Structures Not Impeding Flow

These locations include the floodwalls parallel to the channel and the vertical walls of the outfall structures as well gatewell faces other than the upstream side. For these structures, it is likely that debris impacts will be glancing, and by the rationale provided above the Impact loading is a minimum of 100 lbs per foot at the top of the wall. Reference also the section 9.2.2 discussing the impact loading from debris.

9.2.2.1.2 Debris for Pedestrian Bridges

The two pedestrian bridges located inside the project limits were evaluated for a horizontal debris impact loading of concentrated loading of 1,375 pounds based on calculations provided. The location of the impact loading was applied at the walkway level, and where the shear and moment were maximized. The loading was also increased by a load factor of 1.4 for unusual loads as described in Engineering Manual (EM) 1110-2-2502 and EM 1110-2-2107. The unfactored loading for impact was derived from the ASCE 7-16 commentary for flood impact loading. Section C5.4.5 outlines the analysis of debris striking structures located in floodwaters. Reference attached calculations package for specific considerations, impact loading based on 1,000 pound load striking the side of the bridge traveling at 1.6 feet per second and impacting the bridge for 0.03 seconds. The velocity and impact duration are based on the commentary from section C5.4, and the Maximum Response Ratio is based on the commentary of section C5.4 as well as section 12.8.2 of ASCE 7-16. From Equation 12.8-7 the variables are taken from Table 12.8-2 for 'All other structural systems.'

9.2.3 Ice Loads

While ice will occur in Turkey Creek during the coldest winter months, ice loads impacting structures are not anticipated to occur concurrently with a flood event high enough to load the stem of the structures. The historical flood season/period of record occurs in the spring and summer and thus ice loads will not be included.

9.2.4 Wave Loads

The EM 1110-2-2104 and EM 1110-2-2502 state that wave loads are computed as described in EM 1110-2-1100 accounting for the location of the structure, the characteristics of the hydraulic loading and the site conditions. Wave loads acting on these floodwalls will be insignificant because the fetch lengths are too small to generate a significant wave height. Wave loads are considered inconsequential and ignored in the floodwall design calculations.

9.2.5 Wind Loads

Wind loads were determined in accordance with ASCE 7-16. Section 9.2.5.1 lists the parameters. Wind loads are applied as vacuum pressures to the back side of the stems of floodwalls when serviceability is considered. The floodwall stems were not assessed for wind loading without the presence of water, indicative of conditions where the walls are not loaded with floodwater and are loaded by wind pressure as this loading condition is far lower than the Unusual loading condition and thus by inspection is acceptable.

- Exposure Category: C

9.2.5.1 Applied as a primary load:

- Wind Speed: 122 mph (Risk Category (RC) IV, MRI=1,700 years)
- Resulting pressure*: 30.4 pounds per square foot (psf)

9.2.5.2 Applied as a companion load with flood or construction loads:

- Wind Speed: 88 mph (ASCE 37-14, RC II, 0.8 Construction Factor)
- Resulting pressure*: 15.8 psf

*For continuous wall: K_z=0.85, K_{zt}=1.0, K_d=0.85, K_e=1, G=0.85, C_f =1.3

9.2.6 Construction Loads

The project specification for levee construction requires approved hand operated compactors and a maximum lift of four inches within four feet of structures. Outside of four feet, rolling compaction equipment and a maximum lift of eight inches is allowed. Construction surcharge loads are based on applicable construction equipment but not less than a generalized load of 250 psf. Construction surcharge loads are applied to both the driving side and the resisting side of the wall sections. Refer to Appendix D for the construction load used in each feature specific calculation.

9.2.7 Surcharge Loading

The majority of the floodwalls will not be loaded with surcharge loads from the land side that are significant enough to be considered as part of the design. Two specific locations, both along the right bank of the creek were analyzed with surcharge loading to the toe of the foundations. The first location on the right bank is the area where the floodwall will be adjacent to Merriam Drive, located south of the intersection of Johnson Drive and Merriam Drive. This location will experience heavy wheel loadings from the northbound and southbound lanes of Merriam Drive. Loads considered in this area were taken from the AASHTO LRFD manual and applied the approximate distance away from the foundation.

The second location along the right bank where surcharge loads were considered is located upstream of the first location at approximate creek station 34+00. Through this location there are several industrial facilities that store building materials near the proposed alignment of the floodwalls. This 10 foot tall wall was also checked for surcharge loading because during a visit to the site it was noted that one of the facilities was storing pallets of shingles near the wall. Due

to the weight of the building materials and the lack of space behind the facility the wall's foundation was checked for strength using the additional surcharge loading.

9.2.8 Seismic Loads

- Risk Category: IV
- Site Class: D–Stiff Soil (see 6.9.6)

Table 9-3. Seismic Parameters.

Notation	Value	Description
S_s	0.095	MCE_R ground motion for 0.2 second period
S_1	0.069	MCE_R ground motion for 1.0 second period
S_{MS}	0.152	Site-modified spectral acceleration value
S_{M1}	0.165	Site-modified spectral acceleration value
S_{DS}	0.101	Numeric seismic design value at 0.2 second spectral acceleration
S_{D1}	0.11	Numeric seismic design value at 0.2 second spectral acceleration

The inertial mass of this T-wall when accelerated by the design acceleration (SDS) produces a lateral force similar to the wind load applied to this structure (30 psf). The total lateral force using the equivalent lateral force procedure for a 10-foot stem wall is approximately 34 psf, or 340 pounds applied at the center of gravity of the 10-foot stem. This seismic response force produces less overall force on the T-wall than the static hydraulic force applied during the 0.01 AEP design load. The base and foundation of the T-wall is not included in the overall inertial mass since it is below the ground surface as seismic base shears is only distributed to mass above the base. The base of this structure can resist higher shear loads without sliding from applied static hydraulic loads, therefore these structures will not slide due to seismic loads.

9.3 Design Criteria

9.3.1 Stability

Refer to EM 1110-2-2100 (2005) Chapter 3 and B-3.c and ECB No. 2017-2. Table 9.4 shows these load cases that apply. From inspection of these load cases and considering the descriptions in EM 1110-2-2100 B-3.c (2), the only case that needs to be addressed in the stability analysis is the unusual case with water to the top of wall. With the Infrequent Flood (IF) and the Maximum Design Flood (MDF) probability of occurrence and wall loading equal, case I1 and case I2 are identical. Load Cases I3a and I3b are seismic load cases coinciding with the mean annual non-flood operating conditions. The mean annual river stage is below the floodwall; thus, the load cases need not be considered (see EM 1110-2-2100 B-3. c (2) (c) and (d)). Additionally, with site-modified ground accelerations less than 0.2g, seismic demand will not be greater than demand from the MDF. Load case I4, construction loads, will not control as construction loads near the wall are limited to 250 psf vertical load, which is much less than the design and allowable bearing pressures.

Table 9-4. Inland Flood Wall Stability Load Cases Adapted from EM 1110-2-2100 Table B-4.

Load Case	Loading Description	Classification
I1	Infrequent Flood (IF)	UN
I2	Maximum Design Flood (MDF)	UN/E
I3a	Coincident Pool+OBE	UN
I3b	Coincident Pool+MCE	E
I4	Construction	UN

Sliding (EM 1110-2-2100 Table 3-2) minimum FS is 1.5 for all applicable cases. For this use the T-walls are assessed as critical structures with ordinary site conditions and are loaded with the Unusual Loading Condition category.

Overturning (EM 1110-2-2100 Table 3-5) requires a minimum of 75 percent of the base be in compression for all applicable cases. The applicable cases for this analysis is the Unusual Loading Condition.

Allowable Bearing Capacity (ECB 2017-2 and EM 1110-2-2100) minimum FS for critical structures for Usual case is 3.5, for the Unusual case is 3.0, and for the Extreme case is 2.0. For this project, top of floodwall loading corresponds to the Unusual Load Condition category for a critical structure and a FS of 3.0 is used in the analysis.

9.3.2 Floodwall Bearing Capacity

Soil parameters were provided to the structural engineers for both the soil against and below the soil-founded floodwalls for all project soil-founded floodwall locations. The factors of safety shown in Table 9.5 from ECB 2017-2 Table 2 were used in the structural calculations to obtain allowable bearing capacity values for the various load cases.

Table 9-5. Stability Factors of Safety for Critical Structures.

Category	Usual Minimum FS	Unusual Minimum FS	Extreme Minimum FS
Bearing on Soil	3.5	3.0	2.0

The Unusual case FS for bearing governs the design for floodwalls in this project. The ultimate bearing capacity for soil was determined using EM 1110-2-1905, Equation 4-16 and Meyerhof parameters. Safety factors were applied to the ultimate bearing capacity measured, and not the estimated allowable bearing capacity. The allowable bearing capacity may be estimated according to EM 1110-2-1905 equation 1-2a, where the ultimate bearing is divided by an appropriate FS according to soil type and loading conditions. Section 4-3 of the EM allows for allowable bearing capacity to be estimated with a FS of 2 for cohesionless soils, and a FS of 3 for cohesive soils. The unusual case FS, and the allowable estimated-safety FS for cohesive soils were not taken together.

9.3.3 Concrete Design

EM 1110-2-2104 defines the design requirements for Reinforced Concrete Hydraulic Structures (RCHS) and is the governing criteria for the design of floodwalls, gatewells, and headwalls. The loading requirements in Chapter 3 will be applied per 3.8-Mandatory Requirements, the minimum loading cases in Appendix E shall be satisfied. EM 1110-2-2104 Table E-3 for Inland Floodwalls are analogous to EM 1110-2-2100 paragraph B-3 c and Table B-4, and the

controlling load case is determined similarly as was done for stability. Top of wall hydraulic loading controls for strength design, and temporary loads of seismic and construction loads do not control. EM 1110-2-2104 allows for the design of the T-walls using the strength design methodology, the serviceability stress design or the alternate serviceability stress design (single load case) methodology. Reference the attached load calculations for the derivation, it is shown that the single load case for the loading on the stem with the tallest exposed length of stem governs the design. The following designs for the T-walls does not include the strength design methodology nor the serviceability stress design. Loads applied using the single load case will be as described in Section 9.2 of this DDR, and applied as they are appropriate. Refer to the calculations package in the Appendix D of this report for the specific application. A load factor of 1.6 for the Unusual Case is the controlling case for concrete design for the T-wall structures on this project.

Table 9-6 provides typical reinforced concrete design parameters.

Table 9-6. Reinforced Concrete Design Parameters.

Design Parameter	Hydraulic Structures	Other Civil Works
Concrete design strength	f'_c=4,500 psi typical	
Reinforcement ASTM A615 Grade 60; ASTM A706 Grade 60 if welding is required	f_y=60,000 psi	
Strength reduction factor for shear ACI 318, Table 21.2.1	φ_v=0.75	
Strength reduction factor for flexure ACI 318, Table 21.2.2	φ_b=0.90	
Factor for equivalent rectangular compressive stress block ACI 318, 10.2.7.3	0.85-0.05 (f'c-4,000)/1,000=0.825	
Maximum flexural reinforcement without detailed analyses of the serviceability limit states EM 1110-2-2104, 3.5; EM 1110-2-2502 9-8b (3)	$\rho \leq 0.25\ \rho_b$ where ρ_b=balanced reinforcement	$\rho \leq 0.50\ \rho_b$
Minimum flexural reinforcement EM 1110-2-2104, 3.5.1.4 ACI 318 Sec. 9.6.1.2	$\rho > 3\sqrt{f'_c}/f_y$ and $\rho > 200/f_y$ or one-third greater than the reinforcement required by the structural analysis	
Minimum longitudinal temperature and shrinkage reinforcement EM 1110-2-2104, Table 2-3; ACI 318 Sec.24.4.3.2	$\rho \geq 0.0030$ (control joints<30 feet); $\rho \geq 0.0040$ (control joints 30–40 feet); $\rho \geq 0.0050$ (control joints >40 feet)	$\rho \geq 0.0018$
Minimum transverse temperature and shrinkage reinforcement EM 1110-2-2104, 2.9.1	Based on longitudinal requirements if the primary reinforcement is less than required for temperature and shrinkage.	
Maximum temperature and shrinkage reinforcement EM 1110-2-2104, 2.9.2	#9 bars at 12 inches in each face	

Design Parameter	Hydraulic Structures	Other Civil Works
Minimum concrete thickness EM 1110-2-2104 3.7 EM 1110-2-2502 6-3	12 inches	8 inches (H≤8 feet); 12 inches (H>8 feet)
Minimum concrete cover EM 1110-2-2104, Table 2-1; EM 1110-2-2502 9-4	ACI 318 §20.6.1.3 (T≤12 inches); 3 inches (12 inches<T<24 inches); 4 inches (T≥24 inches); 4 inches for unformed surfaces in contact with foundations	ACI 318 Sec. 20.6.1.3

9.4 Floodwalls

9.4.1 I-walls

I-walls were considered as a potential cost-effective alternative to T-walls in areas where lower floodwall heights were required and available right-of-way was limited between existing structures and the creek channel. The analysis focused on determining the required penetration depth of the sheet piling to account for rotational stability for various wall heights corresponding to the above-ground heights of the six different floodwall geometries.

The existing soil conditions indicate a maximum I-wall height (exposed stem height) of 7 feet per EM 1110-2-2502. In some areas, the required penetration depth needed for rotational stability was not available due to shallow bedrock. Reference Part 07 for expanded discussion on the computations for the I-walls. Prior to finalization of the locations for the I-walls, field verification and exploratory drilling will be conducted to ensure that the required penetration is available. If the required penetration is not available due to shallow bedrock or other issues, the I-walls in that area will be installed by trenching to the required depth, and backfilling around the sheet pile with flowable fill. It is recommended that field verification be performed during design to allow for adjustments as needed at specific locations. The analysis of underseepage and global slope stability for the I-walls is addressed in Part 07 of this report.

I-walls are also planned as part of the transition from floodwall to levee system as outlined in EM 1110-2-2502, Chapter 12, Figure 12.1, except that the floodwalls for the scope of this project do not include Z-pile sheet wall sections. Figure 12.2 shows the plan view of the sheet pile section with respect to the levee, I-wall and T-wall. Since the sheet pile is not intended to be carried into the heel of the T-wall, the variance from this plan view will include not having the stem wall stub to accommodate the slip joint between the I wall and the T-wall. This plan will be altered to include a portion of the heel of the T-wall that turns and comes back toward the center line of the wall to cut off the seepage path between the T-wall's foundation and the sheet pile cutoff of the I-wall. This is done to provide a continuous and uniform barrier that lengthens the seepage path under the foundation of the walls.

The connection of the sheet pile and the I-wall concrete section is outlined in Section 9.8 of EM 1110-2-2502. The intent of the design is that the I-wall sections that are exposed, above ground level, would have the same stem thickness as the T-wall sections. Because of the required Z-pile shape depth, the transition portion of the I-wall that contains the Z-pile will need to be a minimum of 2 and a half feet in thickness to accept the pile profile, the reinforcing steel, and the required 6 inches of concrete cover over the outside of the pile. The intent is to have the I-Wall taper down each face under the levee embankment slope so that when the wall is exposed it matches the stem thickness of the T-walls at one and a half feet in thickness.

9.4.2 T-wall Floodwall

Refer to Sections 9.3 and 9.3.2 for typical design loads and criteria governing the T-wall design. In addition, floodwalls and retaining walls are designed according to EM 1110-2-2502.

T-wall sections were selected based on required height from ground level and required cover to provide frost depth protection and stability. The USACE computer program CTWALL was utilized to analyze each of the wall locations. Locations for analysis were selected by locating the position on stretch of walls of similar height where the stem height above ground on the riverine side was the greatest. CTWALL files were created for each of these locations, and the output files loaded into an Excel workbook where the summarizing information could be read to the 'Wall Data' sheet. This sheet contains information on the soil geometry, wall dimensions, water surface elevations used for analysis, calculated factors of safety for stability and loads computed from each analysis. The table from the 'Wall Data' sheet is then imported into the MathCAD calculations for the stability calculations for the wall sections and the strength calculations for the stem, foundation and reinforcement detail for the wall sections. The wall sections that were evaluated were the tallest stem section, as measured above the ground surface, of the wall type considered.

Reference section 9.2 for discussion on loading criteria and methodology for the analysis conducted. The analysis of underseepage and global slope stability for the T-walls is addressed in Part 7 of this report.

9.4.3 Secant Pile Walls

During the design process the alignment on the right bank south of Johnson Drive bridge was moved away from the existing block walls and close to the existing structures along the right bank. Real Estate analysis showed that acquiring these properties would be costly. It was determined that we should investigate the use of secant piles that could be built without large amounts or lateral excavations along the alignment through this area. It was preliminarily determined that the piles would need to be approximately 3 ft in diameter to resist the loading applied and support the wall system placed on top of the secant pile. The preliminary design of the secant pile floodwall involved casing the secant pile wall to the existing ground surface elevation and using the secant wall as the foundation supporting the stem wall.

During the preliminary design of the secant pile walls it was determined that the cost of construction would be the main driver for limiting the use of secant pile walls throughout the project. The Project Team also held discussions with the Construction Division and even met on the site where the proposed secant walls would be located. Based on the discussed construction methods for both T walls and secant pile walls the team thought that construction of T walls would be challenging but not improbable enough to justify the high cost of installing the secant pile wall type. These conversations, and the resulting decisions were communicated up the vertical command.

Design considerations for the secant pile wall were taken from limited experience with constructing these wall types. Previous design examples from foundations that were used for pump houses on Argentine and the Interceptor design used for lower Turkey Creek were analyzed. It was discussed that the secant piles would form the foundation for the floodwalls, and would not extend to the full height of the required flood protection. This was determined based on the understanding of the construction method used to install secant pile walls, where the surrounding ground and subsurface are used as the forms for the drilled shafts, both primary and secondary. For these pile profiles to be extended to the top of protection circular false work would have to be provided, creating additional constructability constraints. Because the secant pile type foundation would be continuous, another limitation would be the length of consecutive

runs of this wall type. The secant pile foundation would need to be rigidly connected to the floodwall extending up to the level of protection, and thus would create lengths of floodwalls that would need to be critically analyzed for temperature and shrinkage stresses as they would not have construction or contraction joints at standard 20 foot lengths similar to other wall types.

9.5 Drainage System Structures

Gatewell and manhole designs will meet the strength and stability requirements in Section 9.3.2. The closure, or gate, configuration requirements for drainage structures are defined in EM 1110-2-2902 Table 9-1. For a fast-rising stream, Table 9-1 requires one passive closure for pipe diameters less than 36 inches and one passive and one active closure for pipe diameters 36 inches and larger. As discussed in Part 2, the PDT is pursuing a waiver to allow the use of one passive gate or flap instead of one passive and active gate (sluice) due to the flashy nature of Turkey Creek and the very limited time allowed for personnel to close a sluice gate. Refer to Part 2 for further discussion of the deviation from EM 1110-2-2902 Table 9-1.

9.5.1 Pipes Less Than 36 Inches

Riverside concrete outlet structures with flap gates will be constructed to meet the closure requirements of EM 1110-2-2902, Table 9-1.

9.5.2 Pipes Greater Than 36 Inches

With the waiver to remove the second closure structure on pipes larger than 36 inches, the outfall structures for the pipes that are larger than 36 inches in diameter will be similar to the outfall structures for the pipes with diameters less than 36 inches. Where a levee embankment is crossed, a flap gate and outfall structure will be provided on the riverine side of the levee. Reference structural detail sheets, and schedule for outfall structures for their size and location.

9.5.3 Existing Triple 7-foot-by-7-foot Reinforced Concrete Box at Station 38+70 (Quaker Creek)

Sheet SA-300 to SA-302 shows a concept design to construct a riverside gatewell/headwall structure with single flap gates at the existing triple 7-foot-by-7-foot RCB drainage structure. The headwall will be designed using the loading requirements described in Section 9.3 and the stability and strength requirements in Section 9.3.2. The location of the structure is limited by the adjacent street and sidewalk, limiting the extent of new conduit located behind the headwall of the structure. The wingwalls of the structure extends riverward relative to the upstream and downstream floodwalls and are located below the protection pool and upstream of the confluence of Quaker Creek and Turkey Creek. The structure will transition to T-walls on either side of the new box and the foundations of the T-wall will tie into the exterior walls of the gatewell structure. If it is determined that the bedrock in this location is deep enough that a I walls can be supported through this section then the T-walls juxtaposed to the box structure could be replaced with I walls. The advantage of using I-walls through this location would be the absence of the foundation for the wall, which would interfere with the wingwalls of the structure but would still be adequate.

9.5.4 Detention Basin Inlet and Outfall Structures

The inlet and outfall structure for the detention basin located near the Merriam Farmer's Market location are detailed on sheets SB-300 to SB-302. The intake structure is approximately four foot by twelve foot in cross section and five feet tall. The structure has a low flow opening on the back side that is 9 feet and 2 inches by one foot in height that allows the detention basin to drain during storm events that do not provide enough

rainfall to overtop the structure. The structure is open topped to allow for maximum drainage during larger events and is covered with a grating top that has the maximum standardized opening size. The grating will span in the short dimension across the top of the box so loading will not be an issue to the top of the structure, and the end panel can be removed should access to the inside of the box be required for maintenance. The front face of the intake structure is to be connected to two 48 inch diameter pipes. During design it was determined that the two 48 inch pipes would require a smaller intake structure and narrower outlet structure than the alternatives that were explored. The other pipe configurations that were considered for the detention basin were an arc section 54 inches by 88 inches and a box section that was 4 feet by 8 feet. Both of these sections were excluded for either their geometry or their cost. They would also have required that customized flap gates be fabricated for the outfall structures. The 2 side by side 48 inch pipes provide the required cross sectional area required to meet demand and would allow for standardized flap gates to be used on the outfall structure.

9.6 Bridges

9.6.1 Merriam Drive Bridge at River Mile 2.625 (Reach 3b-1)

During the feasibility study, a recommendation was made of adding 4.5 foot-to-6.0-foot-tall parapet walls on each side of the bridge as flood protection. The design of taller parapets on both sides of the bridge was analyzed, and results showed buoyancy forces exceeded the weight of the bridge. It was proposed that the bridge be raised 2.0 feet to an adequate elevation (909.4 feet) to prevent the buoyancy forces from lifting the bridge. This raise provides a FS against floatation of 1.51. A KDOT standard 180-foot slab bridge was proposed to replace the existing bridge. This proposed bridge will use shorter parapet walls as part of the flood protection system. As part of the new bridge design, new bridge piers are being proposed to overcome a current design issue associated with poor pier alignment in the channel, which currently impedes flow. Scour effects around bridge footings and side slopes will be considered in this design.

9.6.2 Johnson Drive Bridge at River Mile 3.058 (Reach 3b-1)

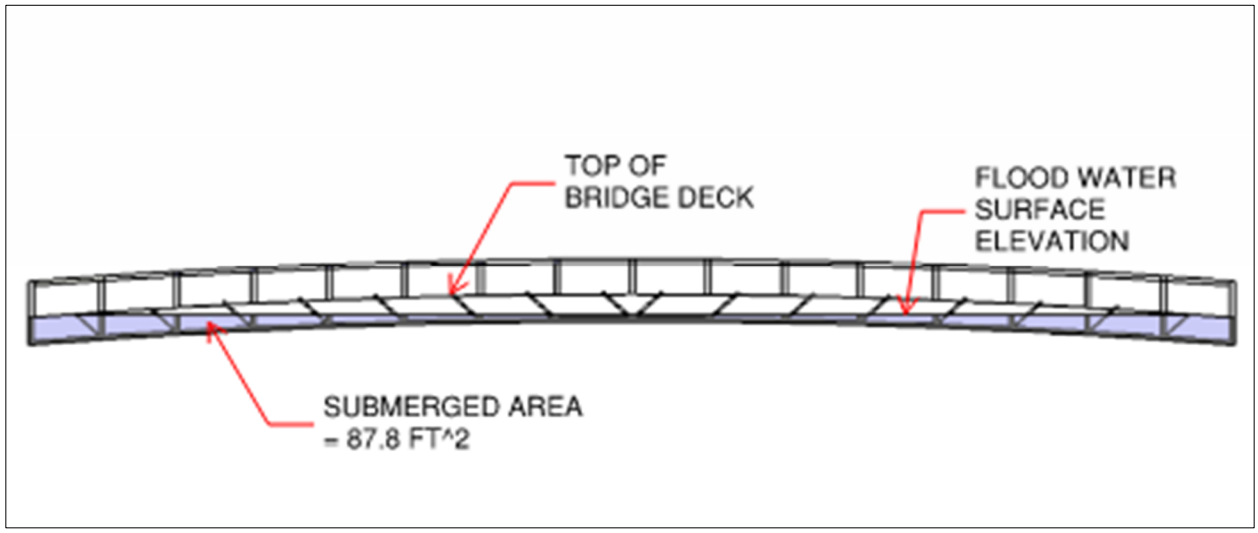
HNTB has provided a conceptual level plan to demolish and replace the existing bridge structure and substructure of Johnson Drive Bridge with a concrete fixed frame bridge as well as raising the roadway elevation of the bridge and widening the channel to the west of the creek centerline. Bridge realignment work will also require that the roadway of Johnson Drive to the west be elevated to a point where it ties into the existing profile elevation approximately 200 feet to the west of the abutment of the new bridge and elevation of the intersection of Merriam Drive and Johnson Drive would also raise approximately one and a half feet causing elevation increases approximately 300 feet east of the abutment of Johnson Drive as well as 200 ft north and south on Merriam Drive.

9.6.3 Farmer's Market Pedestrian Bridge at River Mile 2.875 (Crossing #5 As-Built Drawings) Station 85+10.2

The Farmers Market Pedestrian Bridge was analyzed for stability. The WSE reaches the bridge deck, and the bridge connection to the foundation was analyzed for capacity. Based on drawings of the bridge structure received from the City, the anchors were modeled and analyzed using Hilti's Profis software. Reference the calculations provided in Appendix D of this

report for additional detail, but for the Atlas 14 loading conditions the anchors and bridge connection are adequate to resist the flood loading.

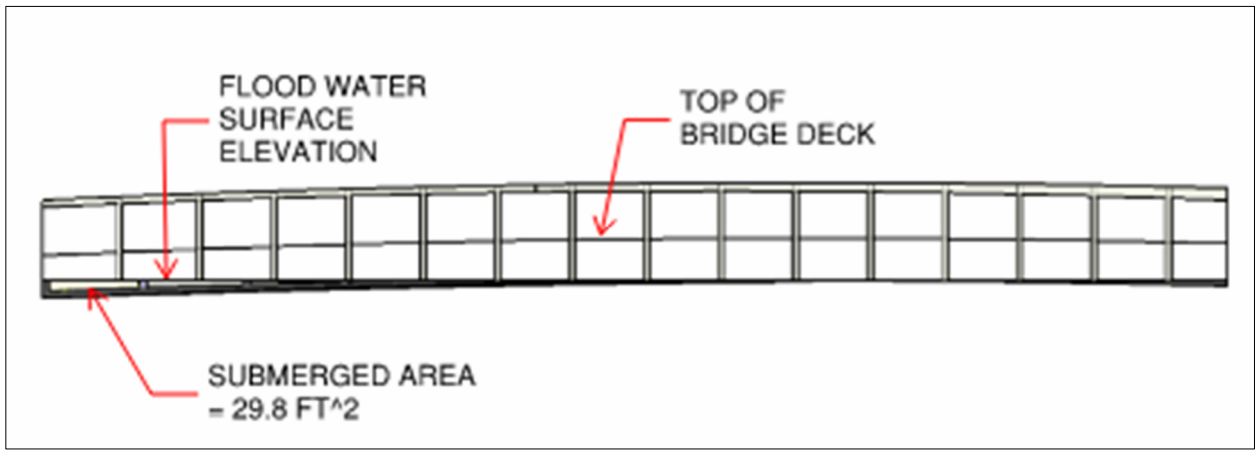
Figure 9.1. Cross Section of the Merriam Marketplace Bridge.



9.6.4 Pedestrian Bridge at River Mile 3.57 (Crossing #2 As-Built Drawings) Station 49+72

The Southern Pedestrian Bridge, Shawnee Mission Parkway Bridge (SMP) was analyzed for stability. The WSE reaches the bridge deck, and the overall shape of the bridge displaces water as would a bridge with closed parapets. Reference calculations provided in Appendix D of this report for additional information. Anchors for the bridge were assessed using Hilti's Profis software and it was determined to have adequate strength to resist the flood loading.

Figure 9.2. Cross Section of the Shawnee Mission Parkway Pedestrian Bridge.



10.0. CIVIL

10.1. Utilities

Existing pipelines crossing the levee that do not meet current USACE criteria in EM 1110-2-2902 will be replaced with pipelines that are compliant. If feasible, all utilities should be located outside of the influence zone. The influence zone is identified as the 15-foot distance from the toe of the levee or the face of the floodwall stem on both the riverside and landside. Existing pipe strengths, unless otherwise known, will be assumed to be that required by USACE criteria at the time of their installation. Pipe condition shall be determined by field assessment.

Gravity lines should be relocated under the crest of the levee if it cannot be located outside of the influence zone. Existing sanitary lines crossing the levee or floodwall will be evaluated by the utility owner (Johnson County Wastewater) to determine structural capacity and condition of the pipes. Any pipes requiring relocation due to levee or floodwall footprint or poor conditions of existing pipe will need to be replaced with a minimum 12-inch RCP. Johnson County Wastewater has requested USACE to design and construct all sanitary sewer relocations associated with this project. USACE requires an agreement between USACE, Sponsor, and utility owner to proceed. Pending approval of the agreement, design will continue after 65% submittal. All existing storm pipes crossing the levee or floodwall need to be replaced with a minimum 24-inch RCP as defined within Part 5, Interior Design. All storm pipes must have a flap gate on the riverside as a passive closure system.

Pressure and communication lines should be relocated over the crest of the levee if it cannot be located outside of the influence zone. Pipes crossing over the levee shall have enough cover for local frost protection, 42 inches minimum. The levee crest will have 10H:1V slopes on both sides of the raised utility to allow for vehicular traffic. Pressure lines need to be designed and constructed to prevent and control seepage and erosion along the pipe. Pressure lines will, at minimum, have shutoff valves on the landside of the levee or floodwall structure located outside of the influence zone.

Before consideration is given to allowing a pressure pipe to remain through or beneath the levee, the pipe owner will provide an engineering study to support their request. The owner, regardless of the type of pipe, must show adequate capability as to the condition of the existing pipe. An agreement will then be formed between the owner and the levee district to the effect that should repairs to a pipe in the levee become necessary, the pipe will be abandoned, grouted, and relocated over the levee.

Pipes that have been abandoned and do not meet criteria will be removed or grouted accordingly. Pipelines that are currently active but are to be abandoned as part of this project will be removed or grouted full in accordance with criteria.

Utility structures should be located outside of the influence zone if feasible. Structures that are placed inside of the influence zone must be evaluated to mitigate any potential breach. Gatewells as an active closure system are not used for this project due to the flashiness of the stream. A deviation memorandum will be drafted at 65% as defined in Part 2. Sanitary pipes should have a manhole located on the landside when feasible. Sanitary manholes located on the riverside of the levee and floodwall structures will have the rim elevation raised to the 25-year storm elevation with a bolted, waterproof cover. Existing manholes will be inspected and lined to prevent infiltration into the sanitary system.

See Appendix C for the detailed Subsurface Utility Engineering table and proposed actions for utilities impacted by construction of this project's levee and floodwall and associated features.

Utilities identified in the table were identified from physical survey and as-built maps provided by utility owners.

10.1.1. Utility Uplift

Will be completed at 65 percent.

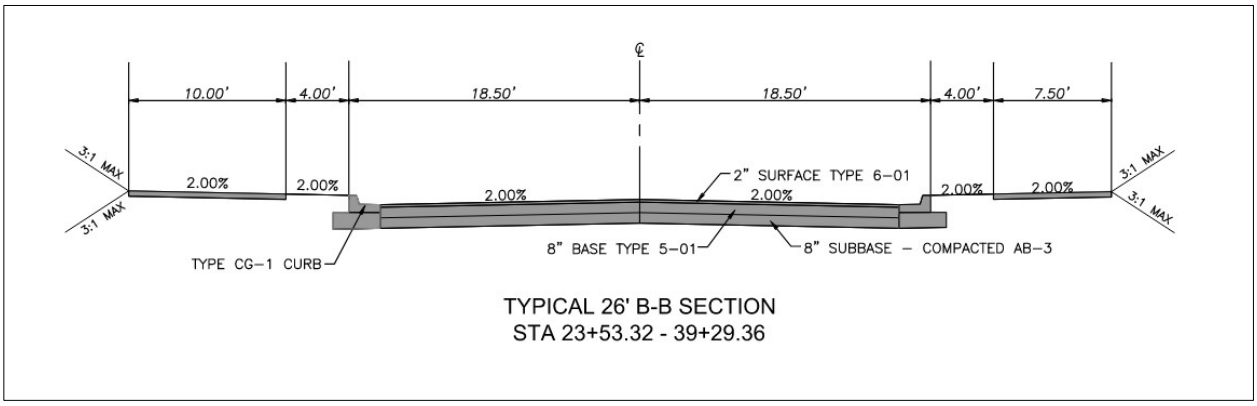
10.2. Bridges

Both vehicle bridges and both pedestrian bridges on this project are impacted by the water surface associated with the 0.01 AEP flood event. Both of the vehicle bridges need to be replaced due to stability concerns. Both pedestrian bridges on this project can remain in place. See Section 9 for further discussion regarding the structural stability of these structures during the design flood event.

In conjunction with the vehicle bridge replacements at Merriam Drive and Johnson Drive, road grading is required to accommodate the new bridge profiles. American Association of State Highway and Transportation Officials (AASHTO) and Kansas City Metropolitan American Public Works Association (APWA) standards were used to grade the road raise designs. HNTB was contracted to complete a conceptual level design for Johnson Drive bridge replacement. See Appendix I for HNTB's bridge report and proposed concept.

Merriam Drive bridge replacement and adjacent road grading was completed in-house. This work includes removing the existing bridge and replacing with a taller and longer KDOT haunch slab bridge, road grading to tie in the bridge raise, along with sidewalk and entrance grading work. Merriam Drive was designed to meet Industrial/Commercial Collector design standards per KCMO APWA 5200. Driveways were designed to meet KCMO APWA standard drawings. Prior to the start of our project, the City of Merriam was working on modifying Merriam Drive to accommodate additional parking and walkability through Merriam's downtown by modifying the road cross section. They have completed designs for their project but decided to pause while we design our FRM project. USACE's bridge and road raise of Merriam Drive will match the City's proposed typical section. Figure 10.1 shows the proposed typical section for Merriam Drive Draft plans provided by the City of Merriam.

Figure 10.1. Merriam Drive Bridge Typical Section.



10.3. Existing Roads

Table 10.1 summarizes impacts to existing roads.

Table 10.1. Roadway Impacts.

Bank	Roadway	Impacts
Left Bank	West 62nd Terrace	Roadway no longer be accessible off of Mastin Street. A floodwall approximately 50ft from the intersection of Mastin St. and West 62nd Terrace will be constructed across West 62nd Terrace.
Left Bank	West 62nd Street	Roadway will have interior ponding adjacent to floodwall during 100 year storm event.
Left Bank	West 61st Street	Roadway will have a floodwall along the curb from Frontage Road to West 62nd Terrace.
Left Bank	Campbell Lane	Roadway will no longer be accessible off of Johnson Drive. A floodwall approximately 20 ft and 175 ft south of Johnson Drive and Campbell Lane intersection will be constructed across Campbell Lane.
Right Bank	West 62nd Street	Cul-de-sac will be partially demolished to accommodate levee toe. Cul-de-sac will be reconstructed approximately 20 ft landward of existing location. Radius to remain the same.
Right Bank	West 61st Street	Roadway west of Merriam Lane will be reconstructed to an access ramp to provide access up and over the levee.
Right Bank	Merriam Drive	The western-most southbound lane south of Johnson Drive intersection will be removed. Three 12.5 ft wide lanes will be present until the existing lane drop approximately XX south of Johnson Drive intersection.
Right Bank	Hocker Drive	Roadway will be partially demolished and reconstructed to an access ramp to provide access to the levee crest.

10.4. Existing Access

The following Section summaries existing and proposed operations and maintenance access as well as existing and proposed pedestrian access.

10.4.1. Operations and Maintenance Access

Table 10.2 summarizes existing and proposed permanent O&M access points.

Table 10.2. Existing and Proposed Operations and maintenance Access.

Bank	Approximate Upper Turkey Creek Station	Existing O&M Access	Proposed O&M Access
Left	63+00	N/A	Propose landside and riverside access off of levee
Right	26+25	Access to creek off of West 61st Street	Propose landside and riverside access off of levee
Right	47+50	N/A	Propose landside and riverside access off of levee
Right	52+75	N/A	Propose landside and riverside access off of levee
Right	57+50	N/A	Propose landside access to detention basin off of levee
Right	61+75	Access to creek immediately south of Merriam Drive Bridge	Access along riverside of levee/floodwall via proposed ramps at approx. UTC Station 52+75

The PDT understands that MDD currently will not cross sanitary lines in the channel with vehicles due to concerns for pipe age and low clearance over the pipes.

10.4.2. Contractor Access

Sheet G-004 of the plan set shows proposed staging areas and contractor access during construction.

10.4.3. Trail Access

The Turkey Creek Streamway Trail is an existing paved trailway within the project limits that will be impacted by the construction of the proposed project. The PDT and Merriam Parks and Recreation are determining trail impacts. Considerations for public access to trails will remain important throughout design. If the trail is between the proposed levee/floodwall and Turkey Creek, the trail could be inundated quickly during flash flood events. During design, access to trails will be assessed and providing adequate alerts and/or evacuation routes during high water events will be evaluated.

Table 10.3 summarizes existing and proposed pedestrian trails.

Table 10.3. Existing and Proposed Pedestrian Trail Access.

Bank	Approximate Upper Turkey Creek Station	Existing Pedestrian Access	Proposed Pedestrian Access
Right	14+80	Access to pedestrian bridge near Merriam Drive and Shawnee Mission Parkway	Propose access around floodwall approximately 5 ft south of floodwall
Left	16+20	Access to private property along West 62nd Street	Propose access around floodwall approximately 5 ft north of floodwall
Left	26+80	Access to trail at Campbell Park	Current design does not impact this existing pedestrian access
Left	39+90	Access to trail at Merriam Lane south of Johnson Drive (bike commuters)	Current design eliminates access to trail south of Johnson Drive. Alternate access along Merriam Lane will be considered in future design.
Right	51+20	Access to pedestrian bridge adjacent to Merriam Marketplace and Werner Park	Propose access up and over proposed levee at existing location

This page was intentionally left blank for duplex printing.

11.0. ENVIRONMENTAL

11.1. Threatened and Endangered Species

During the project design for the Post Authorization Change Report (PACR), it was determined that 4.1 acres would be removed to implement project designs. The PACR concluded that 3.2 average annual habitats units would be required as mitigation based on analysis using the fox squirrel habitat suitability index. However, the project design has been altered, and the riparian and aquatic habitat evaluations will need to be reevaluated. Based on the design changes and a reevaluation of the affected habitat, this updated analysis may conclude that the impacts to riparian habitat would be negligible. If the updated analysis concludes that mitigation is necessary, The USACE determined that on-site mitigation is no longer feasible. Mitigation methods, such as the purchase of mitigation credits or the use of an in-lieu fee program, will continue to be evaluated to identify the most cost-effective mitigation strategy.

Other pertinent environmental considerations include threatened and endangered species due to the timespan of the study and EA and current phase of the project. A search of the U.S. Fish and Wildlife Service (USFWS) Information and Planning Consultation website indicates that activities within this project area have the potential to affect tricolored bat (proposed endangered), monarch butterfly (proposed threatened), and western regal fritillary (proposed threated). Coordination is ongoing with the US Fish and Wildlife Service (USFWS) Kansas Ecological Services Office. The USACE does not anticipate substantial adverse impacts to these species.

11.2. Cultural Resources

Section 106 of the National Historic Preservation Act requires federal agencies to take into account their actions on historic properties, that is, those cultural resources that are listed on or eligible for listing on the National Register of Historic Places (NRHP). To that end, archeologists from the USACE have completed several tasks to date, including a reconnaissance survey of the project area in 2012 and a historic architecture survey in 2025. USACE consulted with the Kansas State Historic Preservation Office (SHPO) regarding the archeological potential for the project area and the Kansas SHPO concurred with USACE's determination that the project should have no effect on historic properties below the surface.

Several standing structures have been selected for buy-out and demolition for the project. In a letter dated February 2023, the SHPO agreed that these structures (regardless of age or condition) must be recorded and evaluated for their eligibility for listing on the NRHP. USACE archeologists recorded all of the structures within the footprint and consulted with SHPO on their eligibility. USACE determined that at least three of the structures were eligible for the NRHP and SHPO concurred.

A Programmatic Agreement (PA) was developed to identify, evaluate, and resolve adverse effects to historic properties, including the eligible buildings, during a future phase of the project. Consulting parties include the Kansas SHPO, City of Merriam, Shawnee Tribe, Osage Nation, Pawnee Nation, Kaw Nation and Omaha Tribe of Nebraska.

11.3. Hazardous, Toxic, and Radioactive Waste

A records search for potential contaminated sites along the reach of Turkey Creek addressed by this DDR was performed in 2009, updated in 2015, and updated again in 2023. Based on the findings from the records search, there are no known superfund sites that will impact the proposed work. In addition, no solid waste facilities, such as former landfills, were identified at

locations that would impact the project. There were several spills reported near Turkey Creek along the reach addressed by this DDR, but they were quickly remediated and should not have any significant impact to the project. There are six properties with current or former USTs, with the status of the tanks unknown on two of those properties, 5560 Merriam Drive and 5745 Merriam Drive, which are no longer in use. There is no known existing contamination associated with any of the current or former tank locations, although some contamination was encountered and remediated when tanks were closed and removed.

A GPR study performed on properties along the channel found subsurface anomalies on 5568 Merriam Drive, 5726 Merriam Drive, 5639 Merriam Drive (DS Bus Lines), 5643 Merriam Drive, 9400 West 61st Street, 9420 West 61st Street, and 80 feet west of the intersection of Merriam Lane and West 61st Street. The nature of the subsurface anomalies is unknown but two of the properties, 5556 Merriam Drive and 5639 Merriam Drive, had historical USTs. Records show the USTs on 5639 Merriam Drive were removed and there was no contamination (but currently have two 10,000-gallon USTs in use) and the status of the UST on 5568 Merriam Drive was unknown but believed to no longer be in use. Appendix F contains additional information on the GPR survey.

Two areas of concern were discovered during a February 2023 soils investigation. Buried railroad ties, stained soil, and debris were discovered while advancing a boring on 5726 Merriam Drive. This may present foundation issues for the proposed levee along this reach as well as issues with construction of a retention basin proposed on this property. The other issue discovered during the February 2023 soil investigation was petroleum contamination located on city property just north of 5920 Merriam Drive at a depth of 10 to 11 feet below the ground surface. Sample results indicated total petroleum hydrocarbon (TPH) levels above residential values for mid-range hydrocarbons but below non-residential levels. Volatile organic compound (VOC) analysis results were all non-detect, which suggests the source of contamination is very old and the underlying constituents driving the health-based TPH risk levels have degraded over time.

Overall, the risk of encountering significant environmental/hazardous, toxic, and radioactive waste impacts during construction appear relatively low based on Federal and State environmental records. However, there remains uncertainty about activities that occurred prior to Federal/State regulations being enacted and records kept. An example of this is the debris encountered at 5726 Merriam Drive and the petroleum contamination just north of 5920 Merriam Drive, both discovered during subsurface investigations performed in February 2023. Neither of these sites were expected to have contamination issues based on environmental records. It is not possible nor cost effective to identify all potential environmental subsurface issues along a greater than one-mile portion of the channel addressed in this DDR. The density of borings and other investigative techniques would be both schedule and cost prohibitive due to the potentially isolated nature of contamination. Therefore, it is recommended that the contractor be required to develop contingency plans to address unexpected environmental conditions and contingency funding be set aside so unforeseen conditions can be addressed without significant project delays.

As for the two recently discovered sites with environmental concerns, 5726 Merriam Drive and just north of 5920 Merriam Drive, it is recommended additional investigations be performed to characterize the nature and extent of the environmental issues more fully on these properties. This can be performed either by USACE during design or by the construction contractor. For the debris at 5726 Merriam Drive, it is likely all debris below the proposed features will need to be removed and disposed of at a permitted facility. For the petroleum contamination encountered just north of 5920 Merriam Drive, based on the TPH levels, it does not appear any remedial activities would need to occur. However, it is good practice to segregate any contaminated soil

from clean soil excavated and dispose of it at a permitted landfill rather than reusing the soil on site or at other project locations. In this case it would likely be classified as a special waste that could be disposed of in a Resource Conservation and Recovery Act (RCRA) Subtitle D landfill, which is the type of landfill that accepts typical household waste.

There were no sites of hazardous, toxic, and radioactive waste concerns identified south of 61st Street within the limits of proposed DDR activities.

Refer to Appendix F for more detailed information on specific sites identified and figures showing the location of the properties of concern relative to project features.

This page was intentionally left blank for duplex printing.

12.0. REAL ESTATE

12.1. General

The project's purpose dictates which real estate must be acquired by the Non-Federal Sponsor (NFS). Along the levee footprint, the NFS owns or has easements for some of the properties required for the project. However, most land in the project area requires the NFS to obtain additional easements or fee acquisitions. Also, MDD operates on lands for which it does not hold easements to, which will also require the NFS to acquire these lands as part of the project.

12.2. Survey

The real estate limits and title survey, completed by Stockwell Engineers Inc. with AE Assistance, were finalized on 14 March 2024. The survey was conducted to help identify those boundaries for the required land needed for the project.

12.3. Site Specific Information

The City of Merriam, Kansas, owns 3.41 acres in fee along the project which will be made available for the project. There are 129 parcels identified within the project area.

12.4. Utility Relocations

The project design requires the relocation of numerous utilities. The NFS is responsible for coordinating with the utility owners and facilitating the relocation of each. Impacted utilities will be included in the 65 percent DDR.

12.5. Site Access

Temporary access for sites that are not part of public roadways will be acquired by the sponsor and will be shown on the real estate exhibits.

This page was intentionally left blank for duplex printing.

13.0. FUTURE CONSIDERATIONS

This project is pausing at 50% design. Below is a summary of future considerations for the PDT when this project continues.

13.1. PM Considerations

- Project Deliverables: 50% design, PACR, and AE (HNTB) submitted deliverables.
    - 50% design (plans and DDR) can be found at – "K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\6.0 Engineering-Design\6.5 Product for Release\6.5.2 65% deliverables"
    - PACR documents (Executive Summary, Body of Report, and Appendices) can be found at – "K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\14.0 PACR\Report". The Decision Document Checklist and Study Issue Checklist are found at the same K: Drive location seen in this bullet. All of the PACR documents will need to be updated when the project comes off the shelf and re-starts.
    - HNTB submitted deliverables can be found at – "K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\9.0 Contracting\9.4 AETaskOrder\9.4.5 Deliverables"
- Decision Log location – "K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\1.0 Project Administration"
- All PCRs can be found at – "K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\1.0 Project Administration\1.1 Project Management Plan\11.0 Change Management".
- While the team was revising the PACR from NWD/ATR reviews and working on the 65% design, USACE met with the City of Merriam. In these discussions with the Sponsor, it was relayed that the City Council will need to review the levee/floodwall design and decide if the City would like to move forward with the levee/floodwall project or if they would like to proceed in a different direction. With the need for the Sponsor to be in full support of the project during the PACR process and to not spend the Sponsor's or USACE's funds unnecessarily, USACE and the City of Merriam decided that we would pause the PACR and shelve the design at 50%. If the City does decide to move forward with the levee/floodwall project, USACE will pick up the design at 50% and resume the PACR for WRDA28.
- The 50% design includes the plans and DDR (no specs). Levee/Floodwall is at 65% design; bridges were closer to 35% design, which is why USACE is calling this the 50% design.
- The 50% design plans to obtain an updated Certified Cost Estimate from the MCX (Walla Walla).
- The PACR was stopped after the Draft Report was sent through ATR/NWD and HQ Policy Review. The Final Report was to go back through ATR and Policy Review, as the Environmental Section and Induced Impacts needed revisions, and the Certified Cost Estimate needed to be revised with 2026 CWICCS numbers. When and if this project is picked back up, the PM should verify with NWD to see if the PACR Final Report needs to go back through ATR. Driving need for PACR was the cost (almost doubling the authorized cost).

- One of the comments from NWD & HQ Policy Review was completion of a Cultural Resources Programmatic Agreement (PA). We had started the PA before it was determined that the PACR would be paused. The City of Merriam would like to pause the PA and it will be picked back up if the project resumes. All information regarding this is saved at – "K:\projects\pogis\admin\Archaeology\CR Project Folder\Kansas River\4. Reports\2026-NWK-002-Upper-Turkey-Creek\7-PA".
- Brief Timeline
    - Authorized under WRDA16 – December 2015
    - Funding Received/Design Initiation – June 2022
    - 35% design complete – Winter 2025
    - PACR started – Fall 2024
    - PACR stopped – Winter 2026
    - 50% design complete – Spring 2026
    - Project shelved – Spring 2026
- Regarding Induced Impacts – please reference the PCR at - "K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\1.0 Project Administration\1.1 Project Management Plan\11.0 Change Management\2026\PCR #2". As of February 2026, USACE has not yet determined how to account for Induced Impacts. For Upper Turkey Creek, flowage easements required for upstream and downstream of the project could range from $4M to $107M.
- Budget Summary – will update when project is closer to being shelved.
- We know of petroleum being discharged in some places along the project ROW. Superfund contaminants would be a non-project cost and the responsibility of the Sponsor. Petroleum is normally a project cost to remove/remediate. Verify when and if this project is taken off the shelf for further design.

13.2. Design Considerations

- H&H Levee Profile
    - It is recommended that a full update to the levee profile be computed with the 85% assurance target in HEC-FDA utilizing the most up to date hydraulic results from design
    - The left bank flood protection alignment was refined prior to the 50% submittal. A shortened analysis was performed in HEC-FDA at only the locations where the water surface profile increased compared to the post-35% design iteration. Due to time constraints, these locations of needed increase to levee/floodwall heights were not implemented in the plan. Detailed information can be found in the Section 4.6.7.
    - A full re-run of HEC-FDA is recommended because the left bank refinements dropped the water surface profile on average relative to the post-35% profile. But due to schedule, locations where the levee heights could be reduced were kept the same to minimize rework for Civil Design.

- Consideration should be given to the possibility of a designated overtopping location if this project resumes in the future. The City of Merriam may have a master redevelopment plan by that point in time that could present an ideal location to design for initial overtopping.

- H&H Channel Scour/Erosion documentation
  - It is recommended that an analysis on the scour potential for the channel is completed in a future submittal. The proposed design is expected to constrain more flow to the channel compared to FWOP conditions. Though it is unlikely for channel scour to be a large concern given the almost entirely bedrock channel bottom, analysis and documentation stating as much is needed to confirm.
  - Channel scour and erosion at the pedestrian bridges should also be future investigated. Based on the analysis that has been conducted, there isn't a concern for scour, but if the water surface profiles are updated to meet new standards, this should be re-evaluated to confirm that there is no additional concern for scour at the pedestrian bridges.

- FEMA Water Surface Comparison
  - It is recommended that once the levee design is near finalization to conduct a comparison to FEMA water surface elevations, profiles, and inundation maps. A rise in water surface is expected with the current design, but a comparison should be documented for future reference.
  - FEMA floodplain mapping for Turkey Creek may be updated between the date of this 50% design submittal and when the project resumes. The fully 2D FEMA map was in draft stage at this point in time and the subsequent mapping from that recent study may supersede the effective products as of Spring 2026..

- T Wall and I Wall Design
  - Preliminary calculations and considerations for the connections of the I walls to the top of the sheet pile section were made such that the wall thickness for the I walls tapers to match the thickness of the T Wall sections. Reference 9.0 STRUCTURES for additional information on the I Wall and T Wall design. Section sizes may need to be further updated to limit deflections and meet serviceability requirements.
  - Additional design considerations should take into account the land usage behind the floodwall. In particular, areas along the right bank, where heavy industrial or automotive types of land use are located directly behind the floodwalls, additional surcharge loadings and additional dead loads from stored materials and heavy wheel traffic should be analyzed to confirm that there are not additional shear demands for the toe of the floodwall foundation that would require additional design considerations.
  - Discussions with Construction and previous experience with T Wall construction on projects like Swope informed that the contractor constructing the T Wall's foundation may want to substitute in a significantly larger section for the T Wall's foundation key than the design drawings show. A preliminary set of CT Wall runs were made for a 10 ft wall section and found that the wall had an insufficient factor of safety for stability. Future designers should consider creating alternate profiles for the T Wall foundations before the project goes to construction so that

initial conversations with the contractor can be had as to what the potential new geometry for the foundation key could be.

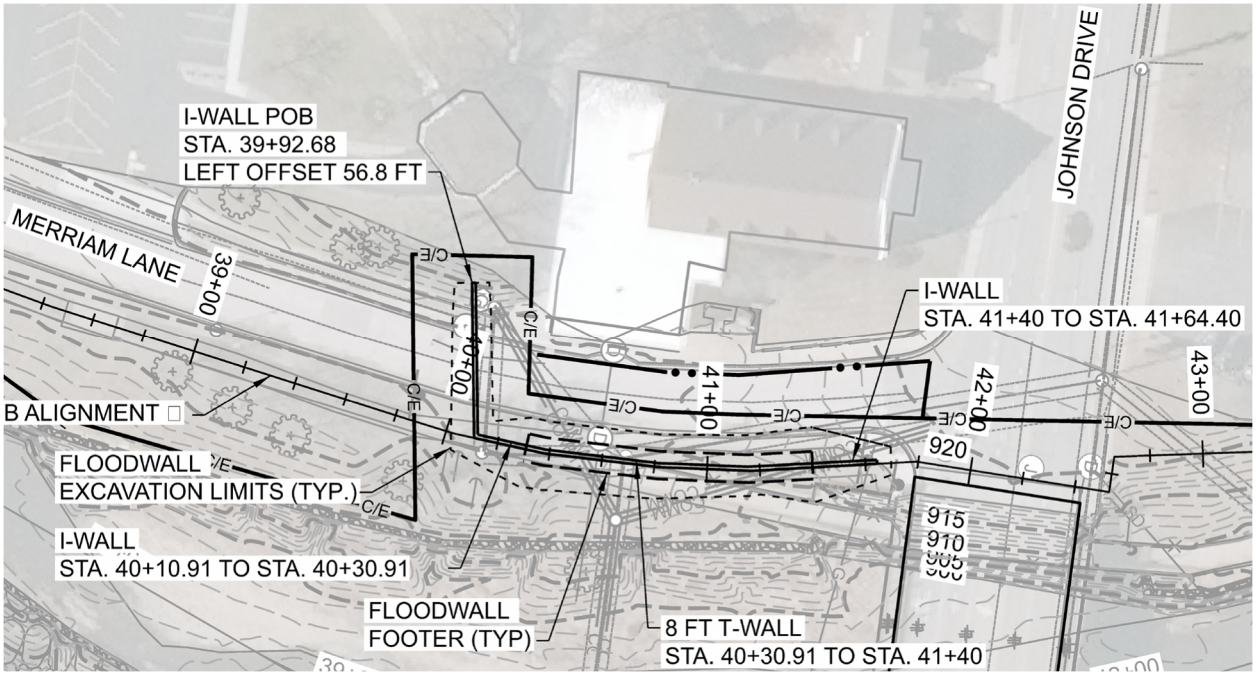
- Outfall Storm Structures, gatewell structures
    - Future considerations for the stability and serviceability of the outfall structures for the storm sewer system should be made. Outfall structures were preliminarily sized to match roughly the land profile of the embankment approximately where the structure is to be located. The limestone wall was not considered in the initial layout of the outfall structure. Future designers for the structures should consider the rotational stability if the outfall structure is shortened to match to the land profile created by the limestone block wall. This new layout would make the outfall structure tall with a narrow base and considerations should be made to move the toe of the structure further behind the wall of the outfall. Structural plans should be updated to include the revised geometry and could possibly include drawings showing the block wall interaction with the new structure and where grouting around the blocks could be used to prevent differential movement between the limestone and the new outfall structures.
    - Outfall structures as they are currently detailed do not include any hoisting mechanism for opening the flap gates attached to the front faces. Past projects have employed the use of a steel tube attached to the top face of the wing walls of the structures such that a chain hoist or come-along winch could be used to safety open the flap gates for inspection.
    - Additional research should be done to find suppliers of flapgates and considerations should be made for the types of material based on producer lead times and long term maintenance issues. Cast Iron gates could be used for the smaller or round pipe openings. Considerations should be made for the gates that will be required at the larger openings such as the detention basin outfall and the Quaker Creek Box Extension.
    - The Quaker Creek Box Extension and the juxtaposed T Wall foundation sections should be analyzed for potential differential settlement issues. Considerations should be made to attach the T Wall sections and their foundations to the side walls and wing walls of the box extension. This joint could potentially have a post-installed waterstop and anchors similar in nature to the post-installed waterstop used at the junction between the existing structure and the new extension of the structure. This would allow for some differential movement between the side walls of the box and the foundations of the T Walls. Another potential consideration would be a smooth dowel connection to the side walls and foundation that would prevent differential settlement between the structure and the T Wall foundation, and would allow for expansion and contraction movements between the foundations and the box structure.
- Bridge refinements
    - Floodwall tie-ins to the bridge will need more refinement and coordination to make sure the bridge and the floodwall line up and include proper detailing. Currently the bridges are planned to be replaced before the construction of the floodwall sections. This will require that the floodwalls tie in to the abutments of the bridges either side of the abutment. Considerations should be made so that the contract drawings have flexibility in the location of these abutments, and allow for ease of construction by the contractor.

- Detours have been provided by the City of Merriam from the last time Johnson Drive was replaced. These detours will need refinement as the design progresses.
- Consider reducing the length of Merriam Drive bridge from 180ft long to 170ft long to reduce the impacts to the bus barn property. H&H would have to re-evaluate the 1D model at the bridge.
- Grading adjacent to and under Merriam Drive bridge will need to be coordinated and shown on the plans. An existing access ramp immediately upstream of Merriam Drive bridge on the right bank will be removed and the area will be regraded so that the area landside of the proposed floodwall grades to drain toward the storm inlet along the roadway.
- The storm outfall immediately downstream of Johnson Drive bridge will need to be coordinated with the bridge abutment. Scour protection measures will also need to be coordinated.

- I-wall location refinements – There is a potential to utilize I-walls in more locations if we add fill around the floodwall an additional one to three feet above the existing ground. Consider taking one last look at potential I-wall locations.
- Settlement of subgrade soils under pipes should be evaluated at utility pipe crossings.
- Alignment refinements – after I-wall location refinements have been identified, consider reevaluating floodwall alignments, especially where riprap riverside of I-wall is planned. A smaller riverside riprap footprint is needed for I-walls, leading to potentially shifting small ranges of floodwall alignments riverside several feet. H&H would need to verify any riverside shift does not negatively impact the minimum level of protection needed.
- Levee Betterments – The Sponsor will decide whether or not to include a betterment to all or some ranges of levee within the project area. Betterment options include (1) widening the levee by 4 feet, (2) adding an asphalt trail atop the levee, (3) adding several levee “bump-outs” along the landside of the levee to accommodate benches, or a combination of these betterment options.
- Refinements adjacent to 9401 Johnson Drive (Merriam Christian Church)
  - Consider modifying the alignment of the proposed floodwall adjacent to 9401 Johnson Drive. Current configuration cuts off an existing access point to Turkey Creek Trail immediately sound of Johnson Drive Bridge along the left bank. Potentially shifting the alignment north could allow the existing access point to remain in-place or be modified to continue to allow access to the trail at this location.
- Interior drainage along Johnson Dr west of bridge – The current left bank alignment at Merriam Christian Church shown below in Figure 13.1 allows for drainage flowing east from Johnson Dr (top right portion of the figure) to flow south along Merriam Lane behind the floodwall. The interior drainage ponds behind the wall and inundates the church basement. A new curb inlet was examined to be placed along the south edge of Johnson Dr before the intersection with Merriam Lane to intersect this drainage. Rational Method analysis of the watershed and FHWA HEC-22 inlet design determined a curb inlet with 5’ wide opening was only 16% efficient at capturing drainage along Johnson Dr. Due to this, the floodwall alignment should be shifted to tie into high ground on the Merriam Christian Church property instead of Johnson Dr bridge and close the entrance to Merriam Lane. This would allow the drainage along Johnson Dr to flow directly into the

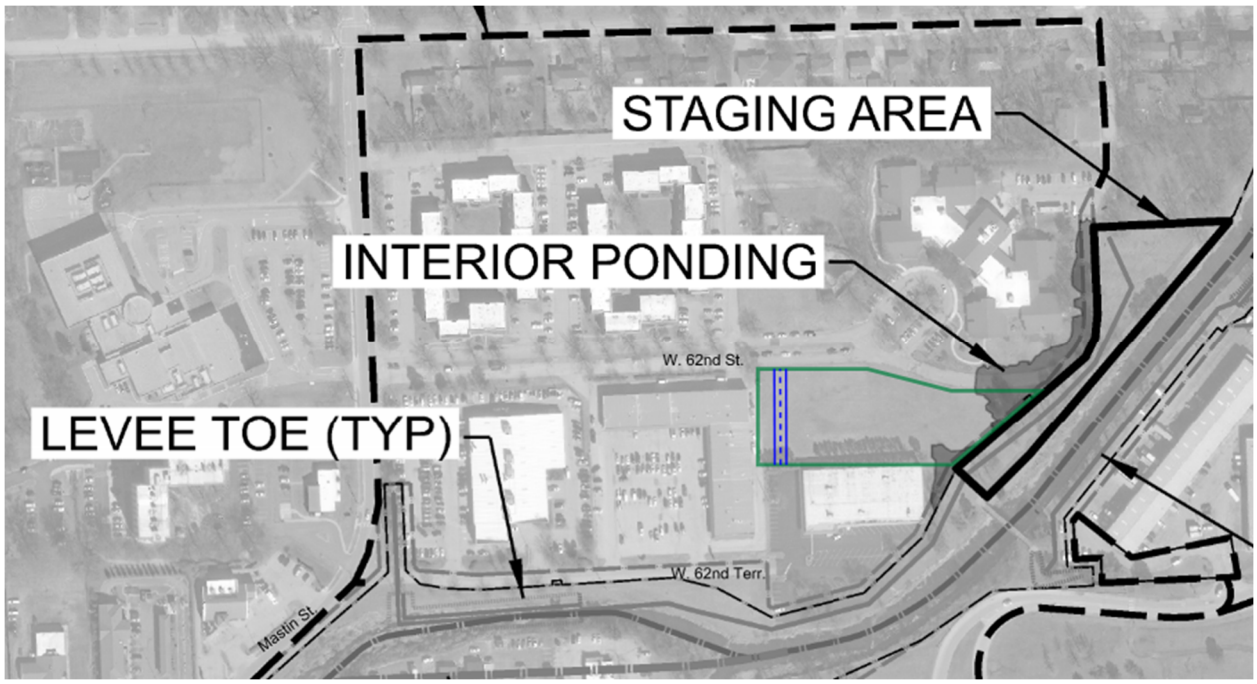
channel. This change would cut off access to the church basement and dumpster area from Merriam Lane and would require the property to move this access.

Figure 13.1. Current left bank alignment of floodwall at Merriam Christian Church.



- Sanitary Sewer Design – Johnson County Wastewater has expressed interest in USACE designing and constructing sanitary sewer relocations as a part of this construction contract. This requires an agreement with USACE, Sponsor, and Johnson County Wastewater to proceed. In order to minimize impacts to the project, only sanitary sewer directly impacted by the alignment of the project was deemed to be relocated. All other sanitary sewer work will be handled on a case-by-case basis on the conditions of the infrastructure. Johnson County Wastewater has shared data within the Johnson County AIMS online system that contains reports and videos on all of their manholes and sanitary lines within the project area. Design of the sanitary system must conform to UFC guidelines and Johnson County Wastewater typical specifications. Construction of the sanitary relocations may include phasing and pumping plans as some of the relocations will be in the same location as existing pipes.
- Staging Areas – Consider expanding the staging area on the left bank off of W. 62nd St, to the area immediately north of 9800 W. 62nd Terrace (Skate City). With the addition of the tieback levee/floodwall crossing W. 62nd Terrace east of the Mastin St. and W. 62nd St. intersection, the business at 9800 W. 62nd Terr. will need new access. A new access entrance off of W. 62nd St. is a logic location. New access will be discussed with the property owner during Real Estate Acquisition.

Figure 13.2: Left Bank Potential Staging Area Expansion.



13.3. Communication

- Getting the constructed project FEMA accredited is the Sponsor's top priority, especially in the downtown reach. If this project continues, suggest setting up meetings with FEMA to get a better level of assurance that this is possible with the proposed profile at 1% AEP with 85% assurance.

13.4. Environmental Considerations

- The impacts were modeled, then the design was modified at 50%. Since we were awaiting the Sponsor's concurrence on the PACR and the overall project, and USACEs decisions regarding Induced Impacts and the overall project, we did not remodel the impacts to riparian and aquatic habitat. These items will need to be reevaluated. The updated analysis may conclude that the impacts to the riparian habitat would be negligible.

14.0. COST ESTIMATE

14.1. General

The Upper Turkey Creek Flood Risk Management (FRM) project is a new project located in the City of Merriam, in Johnson County, Kansas. Turkey Creek is a right bank tributary of the Lower Kansas River located in Johnson and Wyandotte counties, Kansas between approximate river miles (RMs) 2.7 to 3.7. This project includes placing new levees or floodwalls, utility relocations, and replacing Merriam and Johnson Drive bridges to increase the current line of protection. The 65 percent current working estimate (CWE), including Cost and Schedule Risk Analysis (CSRA), will be developed and will be certified by the Mandatory Center of Expertise (MCX) and will be complete around May 2026. The DDR will be updated after the 65 percent CWE is completed. The current working estimate is not considered Controlled Unclassified Information.

14.2. Contract Acquisition

The acquisition method is assumed to be full and open invitation for bid (IFB).

14.3. Estimated Project Duration

Construction Schedule will be updated after 65 percent CWE.

14.4. Basis of Estimate

The basis of the estimate is the 65 percent drawings. 65 percent specs will not be provided. The estimate is based on three contracts and the assumed midpoints of construction for each were provided by the project manager.

14.5. Estimate Preparation

This cost estimate was prepared using the Micro Computer Aided Cost Engineering System (MCACES) Second Generation (MII), version 4.4.3. Subsections 15.5.1 through 15.5.4 detail the supporting databases used in the preparation of the cost estimate.

14.5.1. Cost Book

CB24EN (2024 MII English Cost Book); each unit cost was analyzed and adjusted to reflect current costs.

14.5.2. Labor Rates

This estimate uses a prevailing wage labor rates for Jackson County, Missouri. The latest Davis Bacon were used for preparing the estimate.

14.5.3. Equipment Rates

This estimate is based on the latest available/supported MCACES MII equipment rate database (EP22R05), which has been updated for the estimate to more accurately represent current prices.

14.5.4. Contractors/Subcontractors

The estimate assumes the majority of the work will be performed by the prime contractor, but some subcontracting will be needed by the prime to execute the project.

A surety bond rate was assumed for the prime contractor. The bond rate is calculated by the MCACES MII software.

The following markups were applied to the prime contractor's work:

- Job Office Overhead
- Home Office Overhead
- Profit
- Bond
- Contingency.

Currently a contingency of XX percent will be applied until further design detail is provided. Other specific percentages are retained in MII.

The following markups were applied to any subcontracting work:

- General Overhead
- Profit.

An equipment adjustment factor has been applied to the 2022 equipment prices to update the equipment to current pricing. An equipment adjustment factor has been applied to all equipment items. The adjustment factor has been derived by assuming and escalation factor of 5.0 percent per year.

A material adjustment factor of has been applied to the 2024 Cost Book Material prices to update the materials to current pricing. The adjustment factor has been derived from the Engineering News Record index for construction material.

A sub bid adjustment factor has been applied to the 2024 Cost Book Sub Bid prices to update the sub bids to current pricing. The adjustment factor has been derived from the Engineering News Record index for construction cost.

14.6. Project Owner Markups

The owner also has markups on the project level that are applied after contractor markups. These markups include:

- Escalation
- Supervisory and administration
- Construction contingency is included in design contingency.

14.7. Cost Drivers and Risks

Updating the CSRA after each design submittal will help better define cost and schedule risks and provide confidence levels in being able to complete the project at various funding amounts. High level risks include:

- Will be updated with 65 percent CSRA risks

14.8. Market Conditions and Competition

A large factor in the total project cost will be the number of bids received as open competition has a significant impact on receiving a fair and reasonable price. Historical data shows a small labor pool and the lack of labor could affect cost and schedule.

14.9. Scope Uncertainty

There are many unknowns in the current scope that could impact the total project cost. The estimate will change throughout design as the scope becomes better defined. Below are a few of the unknowns.

- Fill material sourcing is not final.
- Unknown condition of limestone retaining wall.
- Possible new bridge.
- Long term city plans.

This page was intentionally left blank for duplex printing.

15.0 PROJECT FEATURE CHANGES DUE TO ENGINEERING DURING CONSTRUCTION

The DDR should be updated at the completion of construction to document engineering changes that resulted from construction activities. This chapter should be organized in a “by feature” discussion of changes to previously described designs. For example, if anticipated inspection trenches were extended to provide proper cut off, unexpected utilities, unexpected foundation conditions, etc.

This page was intentionally left blank for duplex printing.

16.0. REFERENCES

16.1. USACE Criteria

16.1.1. Engineering Manuals

Reference #	Title	Date
EM 1110-2-1413	Engineering and Design, Hydrologic Analysis of Interior Areas	August 2018
EM 1110-2-1416	River Hydraulics	15 Oct 1993
EM 1110-2-1417	Flood-Runoff Analysis	31 Aug 1994
EM 1110-2-1601	Hydraulic Design of Flood Control Channels	30 Jun 1994
EM 1110-2-1619	Risk-Based Analysis for Flood Damage Reduction Studies	1 Aug 1996
EM 1110-1-1804	Geotechnical Investigations	1 Jan 2001
EM-1110-2-1902	Slope Stability	31 Oct 2003
EM-1110-2-1906	Laboratory Soils Testing	20 Aug 1986
EM 1110-2-1913	Design and Construction of Levees	30 Apr 2000
EM 1110-2-2100	Stability Analysis of Concrete Structures	1 Dec 2005
EM 1110-2-2104	Strength Design for Reinforced Concrete Hydraulic Structures	30 Nov 2016
EM 1110-2-2107	Design of Hydraulic Steel Structures	1 Aug 2022
EM 1110-2-2502	Floodwalls and Other Hydraulic Retaining Walls	1 Aug 2022
EM 1110-2-2504	Design of Sheet Pile Walls	31 Mar 1994
EM 1110-2-2902	Conduits, Pipes, and Culverts associated with Dams and Levee Systems	31 Dec 2020
EM 1110-2-2906	Design of Pile Foundations	15 Jan 1991

16.1.2. Engineering Regulations

Reference #	Title	Date
ER 1110-2-1150	Engineering and Design for Civil Works Projects	August 1999
ER 1110-2-1405	Hydraulic Design for Local Flood Protection Projects	30 Sep 1982
ER 1110-1-1807	Drilling and Invasive Activities at Dams and Levees	1 Jun 2023

16.1.3. Engineering Technical Letter

Reference #	Title	Date
ETL 1110-2-299	Overtopping of Flood Control Levees and Floodwalls	22 Aug 86
ETL 1110-2-575	Evaluation of I-walls	01 Sep 11
ETL 1110-2-584	Design of Hydraulic Steel Structures	30 Jun 14

16.1.4. Engineering Circular

Reference #	Title	Date
EC 1110-2-6066	Engineering and Design: Design of I-walls for design of I-walls	01 Apr 11
EC 1165-2-218	Engineering and Design USACE Levee Safety Program	26 Nov 24

16.1.5. Engineering and Construction Bulletin

Reference #	Title	Date
ECB 2017-2	Revision and Clarification of EM 1110-2-2100 and EM 1110-2-2502	27 Jan 17
ECB 2017-3	Design and Evaluation of I-Walls Including Sheet Pile Walls	27 Jan 17
ECB 2019-8	Managed Overtopping of Levee Systems	24 Apr 19
ECB 2018-14	Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects	10 Sep 18
ECB 2022-7	Interim Approach for Risk-Informed Designs for Dam and Levee Projects	20 Oct 22
ECB 2023-9	Civil Works Design Milestone Checklist	20 Jul 23
ECB 2024-3	Technical Lead for Engineering and Construction Deliverables	9 May 24

16.1.6. Other

	Title	Date
	Upper Turkey Creek Johnson County and Wyandotte County, Kansas, Flood Risk Management Project, Feasibility report with Integrated Environmental Assessment	March 2015
	Upper Turkey Creek Flood Risk Management Project, Preliminary Design Document	June 2023
	Rainfall for Flood Studies: Indian and Tomahawk Creeks	November 2023
	Topographic Survey, GPR and Drafting of Upper Turkey Creek, Merriam, Kansas	March 2024
	Turkey Creek and Brush Creek 2D Watershed, Hydrologic and Hydraulics Report	July 2024

16.2. Federal and State Criteria

16.2.1. State of Kansas

	Title	Date
	Kansas Department of Transportation Design Manual	May 2014

16.2.2. City of Merriam, Kansas

	Title	Date
	City of Merriam Manual of Infrastructure Standards	2019
	City of Merriam Code of Ordinances, Chapter 26-Environment	March 2023

16.3. Associations and Institutes

16.3.1. American Concrete Institute

Reference #	Title	Date
ACI 318-19	Building Code Requirements for Structural Concrete	2022

16.3.2. American Institute of Steel Construction

Reference #	Title	Date
ANSI/AISC 360-16	Specification for Structural Steel Buildings	2016

16.3.3. American Society of Civil Engineers

Reference #	Title	Date
ASCE/SEI 7-16	Minimum Design Loads for Buildings and Other Structures	2017

16.3.4. American Public Works Association

Reference #	Title	Date
APWA 5200	Streets	1996
APWA 5600	Storm Drainage Systems & Facilities	2011

16.4. Computer Programs

Title	Version
Pile Group Analysis Computer Program	2016
STAAD.Pro	
ENERCALC	
HEC-RAS	6.5
HEC-HMS	4.12
HY-8	7.50
CHANLPRO	Win7 64-bit
SEEP/W	2018
SLOPE/W	2018
Bentley Open Roads Designer	2020

17.0. LIST OF ACRONYMS AND ABBREVIATIONS

.tiff tagged image file format

1501 Kansas State Plane North

2D two-dimensional

AASHTO American Association of State Highway and Transportation Officials

AEP annual exceedance probability

AGP Auger grout pile

AIMS Automated Information Mapping System

AMS annual maximum series

APWA 5600 APWA Specification Section 5600, *Storm Drainage Systems & Facilities*

APWA American Public Works Administration

ASCE American Society of Civil Engineers

Atlas 14 NOAA Atlas 14 – Precipitation Frequency Atlas of the United States – Volume 8: Midwestern States

cfs cubic feet per second

CH High plasticity clay

CL lean clay

CSRA Cost and Schedule Risk Analysis

CTP Cooperating Technical Partner

CU consolidated-undrained

CWE current working estimate

DASC Data Access and Support Center

DDR Design Documentation Report

EA Environmental Assessment

EDG-C Engineering Division, Geotechnical Branch, Civil Design Section

EM engineering manual

EP20R05 equipment rate database

EPA Environmental Protection Agency

FEMA Federal Emergency Management Agency

FIRM Flood Insurance Rate Map

FRM flood risk management

FS Factor of Safety

FY fiscal year

GPR ground penetrating radar

GPS Global Positioning System

HEC-1 Hydrologic Engineering Center 1

HEC-HMS Hydrologic Engineering Center Hydrologic Modeling System

HEC-RAS Hydrologic Engineering Center River Analysis System

HEC-SSP Statistical Software Package (HEC-SSP)

I-35 Interstate Highway 35

I-435 Interstate Highway 435

IF infrequent flood

IFB invitation for bid

KDHE Kansas Department of Health and Environment

KDOT Kansas Department of Transportation

LIDAR light detection and ranging

LL life loss

LUST leaking underground storage tank

MARC Mid-America Regional Council

MCACES Micro Computer Aided Cost Engineering System

MCX Mandatory Center for Expertise

MDD Merriam Drainage District

MDF Maximum Design Flood

MetVUE Meteorological Visualization Utility Engine

MH high plasticity silt

MII MCACES second generation

ML silt

MRLS Missouri River Levee System

NAD 83 North American Datum of 1983

NAVD 88 North American Vertical Datum of 1988

NFS	Non-Federal Sponsor
NGVD 29	National Geodetic Vertical Datum of 1929
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRHP	National Register of Historic Places
NWS	National Weather Service
OCR	Overconsolidation Ratio
OPUS	Online Positioning User Service
PDT	project delivery team
PED	Pre-construction Engineering and Design
PL	Public Law
PLF	pounds per linear foot
PMP	Project Management Plan
Psf	pounds per square foot
Psi	pounds per square inch
RCB	reinforced concrete box
RCHS	Reinforced Concrete Hydraulic Structures
RCP	reinforced concrete pipe
RCRA	Resource Conservation and Recovery Act
RM	river mile
SC	clayey sand
SCS	Soil Conservation Service
SDS	design acceleration
SHPO	State Historic Preservation Office
SM	silty sand
SP	poorly-graded sand
SPT	Standard Penetration Testing
STA	station
SQRA	Semi-Quantitative Risk Assessment
SW	well-graded sand

SWE-ELM Shallow Water Equations, Eulerian-Lagrangian Method

TBD to be determined

TP-40 Technical Paper 40

TPH total petroleum hydrocarbon

USACE U.S. Army Corps of Engineers

USCS Unified Soil Classification System

USFWS U.S. Fish and Wildlife Service

USGS U.S. Geological Survey

UTS underground storage tank

UU unconsolidated undrained

VOC Volatile organic compound

WIIN Act Water Infrastructure Improvements for the Nation Act

WRDA Water Resources Development Act

WSE water surface elevation

APPENDIX A – BORING AND LAB DATA

A.1 USACE UTC Alignment Boring Logs
A.2 CFS Borrow Boring Logs
A.3 USACE Borrow Boring Logs
A.4 Lab Data

(See Appendix B Underseepage for Boring Profile with annotations)

This page was intentionally left blank for duplex printing.

APPENDIX A – BORING AND LAB DATA

A.1 USACE UTC Alignment Boring Logs
A.2 CFS Borrow Boring Logs
A.3 USACE Borrow Boring Logs
A.4 Lab Data

(See Appendix B Underseepage for Boring Profile with annotations)

APPENDIX A.1 – USACE UTC ALIGNMENT BORING LOGS

Map of Boring Locations



LOG OF BORING ADUC-196

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-196
COORDINATES: N269673.443, E2251327.337; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.5 (ft) DATE(S) DRILLED: 2/1/23 - 2/2/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: S.Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 21.0(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
0				4			FILL Silty Lean Clay SOFT DRY BROWN TO LIGHT BROWN WITH RUST MOTTLING frozen, trace root and coarse gravel					
1				3								
				1			more gravel and no roots at 1.5'					
2												
3												
4												
5							5.0					
			W-1		0.9		GRAVELLY FAT CLAY With Sand MEDIUM TO STIFF DRY					
6					0.6	11s 1100psi	BROWN TO LIGHT BROWN MOTTLING Few coarse gravel, trace asphalt, trace brick, trace concrete 5'-7'					
					0.4		Possible Fill 5'-7'					
7				4			With Sand STIFF MOIST					
			J-3	7			BROWN TO LIGHT BROWN MOTTLING WITH IRON STAINING	CH	58	36	7.7	53.8
8				3			Few coarse gravel 7'-10'					
9												
10					NS		DRY TO MOIST DARK BROWN some coarse gravel 10'-10.8'					
11			W-2	8		4s 1500psi						
				8			little coarse gravel 10.8'-15'					
12												

LEGEND

- Fill (made ground)
- gCH - Gravelly Fat Clay
- CL - USCS Low Plasticity (Lean) Clay
- SH (WTH) - Shale, Weathered
- SH - Shale

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-1 (secondary nomenclature given by engineer)
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADUC-196

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-196
COORDINATES: N269673.443, E2251327.337; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.5 (ft) DATE(S) DRILLED: 2/1/23 - 2/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: S.Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 21.0(ft): Water Level during drilling.

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
13				6			GRAVELLY FAT CLAY With Sand *(continued)*					
14												
15							15.0					
16				3 3 3			LEAN CLAY MOIST MEDIUM DARK BROWN high silt content					
17												
18												
19												
20				2			SOFT DARK BROWN TO BLACK Silt is blue copper color 20'-25'					
21				2 2			▽					
22												
23												
24												

LEGEND

- Fill (made ground)
- gCH - Gravelly Fat Clay
- CL - USCS Low Plasticity (Lean) Clay
- SH (WTH) - Shale, Weathered
- SH - Shale

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-1 (secondary nomenclature given by engineer)
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADUC-196

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-196
COORDINATES: N269673.443, E2251327.337; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.5 (ft) DATE(S) DRILLED: 2/1/23 - 2/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: S.Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 21.0(ft): Water Level during drilling.

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
25				50			LEAN CLAY MOIST *(continued)* 25.0 HIGHLY WEATHERED SHALE VERY SOFT PARTING Sand at top of sample, consistency of glue						
26													
27							27.8						
28							SHALE Sandy SOFT BANDED GREY Fne sand with occasional coarse grain bands						
29													
30			P-1			RC:100 RQD:54		SH			12.8		U:40
31													
32													
33													
34													
35			P-2			RC:100 RQD:0							
36													

LEGEND

- Fill (made ground)
- gCH - Gravelly Fat Clay
- CL - USCS Low Plasticity (Lean) Clay
- SH (WTH) - Shale, Weathered
- SH - Shale

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-1 (secondary nomenclature given by engineer)
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADUC-196

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-196
COORDINATES: N269673.443, E2251327.337; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.5 (ft) DATE(S) DRILLED: 2/1/23 - 2/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: S.Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 21.0(ft): Water Level during drilling.

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
37							SHALE Sandy SOFT BANDED GREY Fne sand with occasional coarse grain bands *(continued)*	Fill (made ground) gCH - Gravelly Fat Clay CL - USCS Low Plasticity (Lean) Clay SH (WTH) - Shale, Weathered SH - Shale					
38							37.9 BOH No Refusal BACKFILL: Grouted boring with 9 (92.6 lb) bags of portland cement; tremied						
39													
40													
41													
42													
43													
44													
45													
46													
47													
48													

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-1 (secondary nomenclature given by engineer)
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-197

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-197
COORDINATES: N2690007.82, E2250981.87; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +909.0 (ft) DATE(S) DRILLED: 1/25/23 - 1/25/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S.Keeran

GROUNDWATER INFORMATION:

▽ 18.5(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
1				13			GRAVELLY LEAN CLAY SOFT WET DARK BROWN Silty; fine to coarse gravel, mostly coarse; possible asphalt at 1.7						
							1.7						
2				7			FILL Limestone Gravel In Silty, Clay Matrix MEDIUM MOIST BROWN						
3				5			Rubble fill, gravels, asphalt, coarse gravel						
							2.4						
4							FILL Highly Weathered Shale SOFT PARTING VERY FINE GRAINED GREY						
5							3.5						
					0.45		LEAN CLAY STIFF MOIST MOTTLED DARK TO LIGHT BROWN Silty, some coarse gravel						
6			W-1		0.4	8s 1000psi		CL	43	22	14.6	69.3	O:DD:113.1
					0.65		5.0						
7				3			GRAVELLY LEAN CLAY MEDIUM TO STIFF MOIST MOTTLED DARK TO LIGHT BROWN						
8				5									
9				4									
10							10.0						
					0.85		LEAN CLAY MOIST MOTTLED DARK TO LIGHT BROWN AND GREY STIFF						
11			W-2		0.8	6s 800psi		CL	47	28	26	92.3	O:DD:95.8
					0.65								
12													

LEGEND

- gCL - Gravelly Lean Clay
- Fill (made ground)
- CL - USCS Low Plasticity (Lean) Clay
- SH (V WTH) - Shale, Highly Weathered

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-2 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-197

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-197
COORDINATES: N2690007.82, E2250981.87; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +909.0 (ft) DATE(S) DRILLED: 1/25/23 - 1/25/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S.Keeran

GROUNDWATER INFORMATION:

▽ 18.5(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
13			J-7	2			LEAN CLAY MOIST MOTTLED DARK TO LIGHT BROWN AND GREY *(continued)* MEDIUM little black coal nodules at 12'	CL	48	28	30.4	92.0
				2								
				3								
14												
15							15.0					
				2			GRAVELLY LEAN CLAY SOFT WET MOTTLED DARK TO LIGHT BROWN AND GREY Some coarse gravel					
16				1								
				1								
17												
18							18.5 ▽					
				5			HIGHLY WEATHERED SHALE SOFT PARTING VERY FINE GRAINED GREY WITH YELLOW STAINING					
19				15								
				15								
20							20.0					
							BOH No Refusal BACKFILL: Grouted boring with 5 (92.6 lb) bags of portland cement					
21												
22												
23												
24												

LEGEND

- gCL - Gravelly Lean Clay
- Fill (made ground)
- CL - USCS Low Plasticity (Lean) Clay
- SH (V WTH) - Shale, Highly Weathered

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-2 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADU-198

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-198
COORDINATES: N268690.793, E2250985.239; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +909.0 (ft) DATE(S) DRILLED: 1/24/23 - 1/24/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

FIELD DATA								LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
0				2			LEAN CLAY MOIST silty STIFF DARK BROWN trace roots 0'-0.65'						
1				10									
				5			DARK BROWN WITH LIGHT BROWN MOTTLING few coarse gravel 0.65'-1.5' Logged By Cuttings Starting At 1.5' BROWN						
2													
3													
4													
5				2			MEDIUM						
			J-3	3				CL	46	25	24.5	99.2	
6				2									
7													
8													
9													
10					0.2		trace coal fragments, little grey mottling/staining 10'-12'						
11			W-1		0.25	6s 400psi		CL	38	18	28.9	95.1	O:DD:124.2
					0.35								
12													

LEGEND

- CL - USCS Low Plasticity (Lean) Clay
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- SH (V WTH) - Shale, Highly Weathered

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-3 (secondary nomenclature given by engineer); DD=Dry Density; Auger refusal was target depth |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-198

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-198
COORDINATES: N268690.793, E2250985.239; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +909.0 (ft) DATE(S) DRILLED: 1/24/23 - 1/24/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
13				0			LEAN CLAY MOIST silty *(continued)* SOFT Silty, black staining throughout, fine to coarse gravel at bottom 12'-15'	CL - USCS Low Plasticity (Lean) Clay CLg - USCS Low Plasticity (Lean) Clay with Gravel SH (V WTH) - Shale, Highly Weathered						
14				2										
15				2			15.0							
16				2			LEAN CLAY WITH GRAVEL SOFT MOIST TO WET BROWN MOTTLING Silty, black streaking, fine to coarse gravel throughout, trace sand							
17				1										
18				1			17.5							
19							WEATHERED SHALE SOFT PARTING TO BANDED DENSE GREY	**DEFINITIONS:** R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel **ABBREVIATIONS:** J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface					
20				19										
21				41										
22				50			21.5							
23							BOH Sampler Refusal BACKFILL: Grouted boring with 5 (92.6 lb) bags of portland cement							
24														

REMARKS: B-3 (secondary nomenclature given by engineer); DD=Dry Density; Auger refusal was target depth |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADUC-199

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-199
COORDINATES: N268316.968, E2251091.345; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.0 (ft) DATE(S) DRILLED: 3/2/23 - 3/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; NQ core barrel

Driller: Matt Light Geologist: S. Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 17.9(ft): Water Level during drilling.

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
0				2			LEAN CLAY STIFF MOIST BROWN Silty, Few roots					
1				4								
				5			0.8					
2							GRAVELLY LEAN CLAY STIFF MOIST BROWN Fine to coarse gravel					
							2.0					
3							LEAN CLAY MOIST BROWN Silty, Flocculated					
4												
5							5.0					
				1			FAT CLAY SOFT MOIST BROWN Silty					
6				1								
				1								
7												
8												
9												
10							10.0					
				2			LEAN CLAY SOFT MOIST MOTTLED BROWN W/ IRON & BLACK STAINING Silty, Trace wood 10'-15'					
11				2								
				2								
12												

LEGEND

- CL - USCS Low Plasticity (Lean) Clay
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- CH - USCS High Plasticity (Fat) Clay
- SH (WTH) - Shale, Weathered
- SH - Shale

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-4 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADUC-199

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-199
COORDINATES: N268316.968, E2251091.345; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.0 (ft) DATE(S) DRILLED: 3/2/23 - 3/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; NQ core barrel

Driller: Matt Light Geologist: S. Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 17.9(ft): Water Level during drilling.

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
13							LEAN CLAY *(continued)*	CL - USCS Low Plasticity (Lean) Clay CLg - USCS Low Plasticity (Lean) Clay with Gravel CH - USCS High Plasticity (Fat) Clay SH (WTH) - Shale, Weathered SH - Shale					
14													
15				0			VERY SOFT MOIST TO WET DARK BROWN W/ FEW GREY STAINING 15'-20'						
16				0									
17				0									
18							▽ WET no grey at 17.5'						
19													
20							20.0 WEATHERED SHALE VERY SOFT PARTING DENSE						
21							21.2 SHALE SOFT PARTING DENSE						
22													
23			P-1										
24													

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-4 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADUC-199

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-199
COORDINATES: N268316.968, E2251091.345; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.0 (ft) DATE(S) DRILLED: 3/2/23 - 3/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; NQ core barrel

Driller: Matt Light Geologist: S. Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 17.9(ft): Water Level during drilling.

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
25							SHALE SOFT PARTING DENSE *(continued)*					
26												
27												
28			P-2									
29												
30												
31							31.0 BOH No Refusal BACKFILL: Grouted boring with 7 (92.6 lb) bags of portland cement; tremied					
32												
33												
34												
35												
36												

LEGEND

- CL - USCS Low Plasticity (Lean) Clay
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- CH - USCS High Plasticity (Fat) Clay
- SH (WTH) - Shale, Weathered
- SH - Shale

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-4 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-200

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-200
COORDINATES: N267768.548, E2251290.481; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.6 (ft) DATE(S) DRILLED: 1/23/23 - 1/23/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION:

▽ 10.0(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
1							FAT CLAY SOFT MOIST BROWN Silty, Few roots					
							1.0					
					0.55		LEAN CLAY STIFF					
2			W-1		0.9	6s 800psi	MOIST BROWN Silty, Little iron staining, little grey mottling, few coarse gravel					59.0
3							3.0					
				4			GRAVELLY LEAN CLAY MEDIUM MOIST DARK BROWN with sand; Little iron staining, chips of weathered shale, little coarse gravel					
4				3								
				3								
5							STIFF					
					0.7							
6			W-2		0.5	6s 1000psi		CL	40	19	13.4	59.7
					0.85							
7							7.0					
				3			CLAYEY GRAVEL SOFT MOIST DARK BROWN WITH IRON MOTTLING Silty; coarse gravel					
8			J-4	1				GC	45	22	25.6	40.9
				2								
9												
10							▽					
				3			MEDIUM WET DARK & LIGHT BROWN MOTTLING Fine and coarse gravel					
11				2								
				4								
12												

LEGEND

- CH - USCS High Plasticity (Fat) Clay
- CL - USCS Low Plasticity (Lean) Clay
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- GC - USCS Clayey Gravel
- SH (V WTH) - Shale, Highly Weathered

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-5 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-200

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-200
COORDINATES: N267768.548, E2251290.481; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +910.6 (ft) DATE(S) DRILLED: 1/23/23 - 1/23/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION:

▽ 10.0(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND
13							CLAYEY GRAVEL *(continued)*	CH - USCS High Plasticity (Fat) Clay CL - USCS Low Plasticity (Lean) Clay CLg - USCS Low Plasticity (Lean) Clay with Gravel GC - USCS Clayey Gravel SH (V WTH) - Shale, Highly Weathered
14								
15				3				
				6			15.8	
16				11			HIGHLY WEATHERED SHALE SOFT PARTING VERY FINE GRAINED GREY	
17								
18				10				
				50			18.8	
19							BOH Sampler Refusal BACKFILL: Grouted boring with 6 (92.6 lb) bags of portland cement tremied	**DEFINITIONS:** R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel **ABBREVIATIONS:** J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface
20								
21								
22								
23								
24								

LABORATORY DATA

USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
					U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates (mg/kg) P: Soil pH O: Other lab data defined in Remarks

REMARKS: B-5 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING AD-201

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-201
COORDINATES: N267560.282, E267560.3; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.1 (ft) DATE(S) DRILLED: 2/6/23 - 2/6/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: C. Broksieck Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

FIELD DATA								LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
0				4			SILTY CLAY STIFF MOIST DARK BROWN trace roots 0'-0.5'	CL-ML border line lean clay-silt						U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates (mg/kg) P: Soil pH O: Other lab data defined in Remarks
1				5										
				8			DARK BROWN WITH IRON MOTTLING 0.5'-1.5' some coarse gravel, trace coarse sand DARK BROWN AT 1.5'						
2														
3														
4														
5														
6														
7				2			SOFT DARK BROWN WITH IRON MOTTLING few coarse gravel							
				1			sand lens at 7.8'							
8				3			8.5							
9							BOH No Refusal BACKFILL: Backfilled and tamped with cuttings							
10														
11														
12														

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-6 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING AD-201A

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-201A
COORDINATES: N267560.35, E267560.36; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.0 (ft) DATE(S) DRILLED: 2/6/23 - 2/6/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer

Driller: C. Broksieck Geologist: S. Keeran

GROUNDWATER INFORMATION:

▽ 14.7(ft): Water Level during drilling.

FIELD DATA							LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
1							SILTY CLAY STIFF MOIST DARK BROWN trace roots 0'-0.5'	CL-ML border line lean clay-silt GP-GC - USCS Poorly-graded Gravel with Clay SP - USCS Poorly-graded Sand SP-SM - USCS Poorly-graded Sand with Silt CL - USCS Low Plasticity (Lean) Clay						U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates (mg/kg) P: Soil pH O: Other lab data defined in Remarks
2							DARK BROWN WITH IRON MOTTLING 0.5'-1.5' some coarse gravel, trace coarse sand DARK BROWN AT 1.5'							
3														
4														
5							Auger Chatter At 5'`							
6														
7							SOFT DARK BROWN WITH IRON MOTTLING few coarse gravel							
8							8.0 POORLY GRADED GRAVEL WITH CLAY MOIST DARK BROWN coarse gravel with silty lean clay							
9							9.0 POORLY GRADED SAND MOIST DARK TO LIGHT BROWN fine sand							
10				3			10.0 POORLY GRADED SAND WITH SILT LOOSE WET BROWN TO BLACK Strong petroleum odor and staining; fine to coarse sand		SP-SM			18.6	11.1	
11			J-1	2										
12				2										

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: |Final Review by AML
COORDINATE SOURCE: GPS & Google Earth

LOG OF BORING AD-201A

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-201A
COORDINATES: N267560.35, E267560.36; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.0 (ft) DATE(S) DRILLED: 2/6/23 - 2/6/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer

Driller: C. Broksieck Geologist: S. Keeran

GROUNDWATER INFORMATION:

▽ 14.7(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM
13							POORLY GRADED SAND WITH SILT LOOSE WET BROWN TO BLACK Strong petroleum odor and staining; fine to coarse sand *(continued)*
14							
15							15.0
16			J-2	14 29 43			LEAN CLAY Hightly Weathered Shale HARD GREY
							16.5
17							BOH No Refusal BACKFILL: Grouted boring with 5 (92.6 lb) bags of portland cement; tremied
18							
19							
20							
21							
22							
23							
24							

LEGEND

- CL-ML border line lean clay-silt
- GP-GC - USCS Poorly-graded Gravel with Clay
- SP - USCS Poorly-graded Sand
- SP-SM - USCS Poorly-graded Sand with Silt
- CL - USCS Low Plasticity (Lean) Clay

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

LABORATORY DATA

USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
CL	44	22	12.7	91.3

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: |Final Review by AML
COORDINATE SOURCE: GPS & Google Earth

LOG OF BORING AD-201B

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-201B
COORDINATES: N267560.36, E267560.37; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.0 (ft) DATE(S) DRILLED: 2/6/23 - 2/6/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: C. Broksieck Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

FIELD DATA								LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
0				4			SILTY CLAY STIFF MOIST DARK BROWN trace roots 0'-0.5'	CL-ML border line lean clay-silt						U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates (mg/kg) P: Soil pH O: Other lab data defined in Remarks
1				5										
				8			DARK BROWN WITH IRON MOTTLING 0.5'-1.5' some coarse gravel, trace coarse sand							
2							DARK BROWN AT 1.5'							
3														
4														
5							Auger Chatter At 5'							
6														
7				2			SOFT DARK BROWN WITH RUST MOTTLING few coarse gravels; medium grained sand lens at 7.8'							
8				1										
				3			8.5							
9							BOH No Refusal BACKFILL: Backfilled and tamped with cuttings							
10														
11														
12														

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: |Final Review by AML
COORDINATE SOURCE: GPS & Google Earth

LOG OF BORING ADUC-202

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-202
COORDINATES: N267053.92, E2251115.475; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.0 (ft) DATE(S) DRILLED: 2/7/23 - 2/7/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: S. Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 10.3(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
				5			ASPHALT Concrete underneath 0.7						
1				9			LEAN CLAY STIFF DRY BROWN Silty, likely fill (few brick and gravel pieces)						
2				6			2.5						
3							SANDY LEAN CLAY DRY TO MOIST BROWN silty						
4													
5					0.3		MEDIUM TO STIFF MOIST MOTTLED GREY & BROWN Silty, Some coarse gravel and trace roots						
6			W-1		0.65	5s 600psi		CL	42	21	23.2	67.2	O:DD:96.6
7					NS								
				2			SOFT no roots at 7'						
8				1									
9				2									
10			NS			3s 1500psi	10.0 SANDY SILT WET LIGHT BROWN	ML			54.1	67.6	
11			J-5	50+			SHALE AND LIMESTONE FRAGMENTS 10.3 WEATHERED SHALE SOFT PARTING						
12													

LEGEND

- Asphalt
- CL - USCS Low Plasticity (Lean) Clay
- sCL - Sandy Lean Clay
- ML - USCS Silt
- SH (WTH) - Shale, Weathered
- LS - Limestone

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-7 (secondary nomenclature given by engineer)
DD=Dry Density
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADUC-202

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-202
COORDINATES: N267053.92, E2251115.475; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.0 (ft) DATE(S) DRILLED: 2/7/23 - 2/7/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: S. Keeran & T. Fox

GROUNDWATER INFORMATION:

▽ 10.3(ft): Water Level during drilling.

FIELD DATA								LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
13			P-1			RC:82.5 RQD:58	VERY FINE GRAINED GREY	SH			15.4		U:60
							10.7						
14							LIMESTONE HARD MEDIUM BEDDED VERY FINE GRAINED LIGHT GREY						
							12.0						
15							WEATHERED SHALE SOFT PARTING VERY FINE GRAINED GREY						
16													
17			P-2			RC:100 RQD:54							
18													
19													
20													
21			P-3			RC:100+ RQD:41							
22													
							22.7						
23							BOH No Refusal BACKFILL: Grouted boring with 4 (92.6 lb) bags of portland cement; tremied						
24													

LEGEND

- Asphalt
- CL - USCS Low Plasticity (Lean) Clay
- sCL - Sandy Lean Clay
- ML - USCS Silt
- SH (WTH) - Shale, Weathered
- LS - Limestone

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-7 (secondary nomenclature given by engineer)
DD=Dry Density
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING AD-203

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-203
COORDINATES: N266661.508, E2250806.114; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +916.0 (ft) DATE(S) DRILLED: 1/19/23 - 1/19/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer.

Driller: Matt Light Geologist: S.Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
0				6			LEAN CLAY STIFF MOIST DARK BROWN WITH RUST STAINING						
1			J-2	14			Silty, trace roots 0'-0.5'		GM	NP	NP	7.6	15.1
				9			0.5 SILTY GRAVEL Fill STIFF DRY TO MOIST BROWN TO LIGHT BROWN with sand; fine to coarse white gravel	CL - USCS Low Plasticity (Lean) Clay GM - USCS Silty Gravel GP-GC - USCS Poorly-graded Gravel with Clay					
5				4			MOIST DARK BROWN with lean clay; coarse gravel; glass shards						
				17			5.7 POORLY GRADED GRAVEL WITH CLAY STIFF DRY LIGHT BROWN TO WHITE Silty, fine to coarse gravel						
				(50)			6.5 BOH Sampler Refusal BACKFILL: Grouted boring with 2 (92.6 lb) bags of portland cement						

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-8 (secondary nomenclature given by engineer) |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-204

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-204
COORDINATES: N266258.574, E2250611.583; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +921.4 (ft) DATE(S) DRILLED: 1/18/23 - 1/18/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
1							ASPHALT Concrete underneath						
2							2.0						
3			W-1		0.55 0.65 0.45	6s 800psi	CLAYEY GRAVEL WITH SAND MEDIUM TO STIFF MOIST BROWN W/ GREY AND IRON MOTTLING Silty, coarse gravel, few roots, fine sand	GC	38	18	22.2	47.4	O:DD:99.7
4							4.0						
5				2 3 2			LEAN CLAY MEDIUM MOIST DARK BROWN Silty						
6							6.0						
7			W-2		0.5 0.45	7s 800psi	LEAN CLAY WITH GRAVEL Coarse Gravel, Small Cobbles MEDIUM DRY TO WET DARK BROWN Silty						
8							8.0						
9			J-4	6 7 50			SILTY GRAVEL HARD DRY TO MOIST BROWN TO LIGHT BROWN Silty clay, Little iron staining	GM			8.7	17.7	
							9.7						
10							BOH Auger Refusal BACKFILL: Grouted boring with 3 (92.6 lb) bags of portland cement						
11													
12													

LEGEND

- Asphalt
- GC - USCS Clayey Gravel
- CL - USCS Low Plasticity (Lean) Clay
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- GM - USCS Silty Gravel

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-9 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADUC-205

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-205
COORDINATES: N265638.893, E2250109.571; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +928.8 (ft) DATE(S) DRILLED: 1/30/23 - 1/30/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: T. Fox

GROUNDWATER INFORMATION:

▽ 10.5(ft): Water Level during drilling.

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
0				2			GRAVELLY LEAN CLAY VERY STIFF DRY TO MOIST BROWN Trace organic material; some (approx 30%) fine to coarse gravel, mostly fine					
1				8								
				9								
2												
3												
4												
5							5.0					
				4			LEAN CLAY MEDIUM DRY TO MOIST REDDISH BROWN Trace fine gravel, Trace fine to medium sand grains					
6			NS	3								
				5								
7												
8												
9							GREY					
10							10.0					
			W-1		0.1		SANDY LEAN CLAY VERY SOFT MOIST GREY	CL	38	16	33	67.2
11				8			Fine to medium grain sand, trace fine gravel, subtle organic odor VERY STIFF consistency change at 11'					
12				11								

3s 600psi ▽

LEGEND

- gCL - Gravelly Lean Clay
- CL - USCS Low Plasticity (Lean) Clay
- CLs - USCS Low Plasticity (Lean) Clay with Sand
- LS - Limestone

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-10 (secondary nomenclature given by engineer)
DD=Dry Density
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADUC-205

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADUC-205
COORDINATES: N265638.893, E2250109.571; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +928.8 (ft) DATE(S) DRILLED: 1/30/23 - 1/30/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler; NQ core barrel

Driller: Matt Light Geologist: T. Fox

GROUNDWATER INFORMATION:

▽ 10.5(ft): Water Level during drilling.

FIELD DATA								LABORATORY DATA				
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
13				8			few fine gravel at 12'						
				50			13.0						
14							LIMESTONE MODERATELY HARD MEDIUM BEDDED LIGHT GREY Fossiliferous, trace calcite pockets						
15			P-1			RC:65 RQD:13		LS					U:12560
16													
17													
18													
19			P-2			RC:100 RQD:51	Dark grey clayey material in bedding breaks at 18.5'	LS					U:8540
20													
21													
22			P-3			RC:97 RQD:42							
23													
							23.7						
24							BOH No Refusal BACKFILL: Grouted boring with 4 (92.6 lb) bags of portland cement; tremied						

LEGEND

- gCL - Gravelly Lean Clay
- CL - USCS Low Plasticity (Lean) Clay
- CLs - USCS Low Plasticity (Lean) Clay with Sand
- LS - Limestone

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-10 (secondary nomenclature given by engineer)
DD=Dry Density
UC in PSI |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-206

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-206
COORDINATES: N265377.464, E2249485.597; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +931.4 (ft) DATE(S) DRILLED: 1/17/23 - 1/17/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; Sampler includes 5" waxed shelby tube.

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
1					0.4		SILTY CLAY MOIST BROWN MEDIUM few grass roots Some rust staining, few trace roots, trace fine gravel, White film on bottom of shelby sample (degraded plastic?)						
2			W-1		0.45	7s 700psi							
3					0.5		3.0						
4			J-2	2 3 3			LEAN CLAY MEDIUM MOIST BROWN few trace roots, trace fine gravel	CL	41	19	21.5	93.2	
5							5.0						
6			W-2		0.3 0.35 0.35	8s 1300psi	LEAN CLAY WITH SAND MEDIUM DRY TO MOIST BROWN trace roots	CL	40	19	18.1	80.0	O:DD:103.1
7				5			STIFF Trace iron staining 7'-10'						
8				6 9									
9													
10				8			10.0 POORLY GRADED GRAVEL WITH CLAY HARD DRY TO MOIST LIGHT TO DARK BROWN fine to coarse gravel in a sandy, lean clay matrix						
11				8 50			11.2 BOH Sampler Refusal BACKFILL: Grouted boring with 4 (92.6 lb) bags of portland cement						
12													

LEGEND

- CL-ML border line lean clay-silt
- CL - USCS Low Plasticity (Lean) Clay
- CLs - USCS Low Plasticity (Lean) Clay with Sand
- GP-GC - USCS Poorly-graded Gravel with Clay

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-11 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-207

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-207
COORDINATES: N266661.508, E2250806.114; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +914.5 (ft) DATE(S) DRILLED: 1/18/23 - 1/18/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
1							FAT CLAY MOIST BROWN Silty, few grass roots	CH - USCS High Plasticity (Fat) Clay CL - USCS Low Plasticity (Lean) Clay GP-GC - USCS Poorly-graded Gravel with Clay						
							1.0							
2			W-1		0.5 0.55 0.6	5s 700psi	LEAN CLAY MEDIUM TO STIFF MOIST BROWN							
3				2			MEDIUM							
4				3 2										
5														
6			W-2		0.3 0.45 0.45	6s 650psi			CL	44	23	25	98.1	O:DD:92.5
7							7.0							
8			J-4	3 3 3			SANDY LEAN CLAY MEDIUM MOIST BROWN WITH IRON MOTTLING Silty, trace well graded gravel, Coal/shale nodules		CL	46	26	22.1	62.4	
							8.5							
9				3 4 11			POORLY GRADED GRAVEL WITH CLAY Fine To Coarse Gravel In A Sandy Clay Matrix STIFF MOIST BROWN TO LIGHT BROWN							
10							9.8 BOH SPT Refusal BACKFILL: Grouted boring with 3 (92.6 lb) bags of portland cement							
11														
12														

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-12 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING ADU-208

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-208
COORDINATES: N266226.66, E2250316.424; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +921.8 (ft) DATE(S) DRILLED: 1/31/23 - 1/31/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: C. Broksieck Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
1					0.3		LEAN CLAY SOFT MOIST BROWN Silty 0'-1' SOFT TO MEDIUM BROWN WITH LIGHT BROWN MOTTLING Some coarse gravel at 1'-3'						
2			W-1		0.1	4s, 600psi		CL	45	23	23	97.7	
3			J-1	3	0.2			CL	45	23	25.4	96.7	O:DD: 94.3
4				4			MEDIUM MOIST no gravel						
5				3			5.0						
6			W-2		0.3	5s, 700psi	SILTY CLAY Few Cobbles Near 4' SOFT TO MEDIUM MOIST BROWN WITH LIGHT BROWN & GREY MOTTLING some coarse gravel, trace roots						
7					0.2								
8				4	0.1		7.0 LEAN CLAY WITH GRAVEL HARD MOIST BROWN WITH IRON STAINING Silty, fine to coarse gravel						
9				5									
10				48			8.7 LIMESTONE MODERATELY HARD VERY FINE GRAINED GREY						
11							9.5 BOH Auger Refusal BACKFILL: Grouted boring with 3 (92.6 lb) bags of portland cement						
12													

LEGEND

- CL - USCS Low Plasticity (Lean) Clay
- CL-ML border line lean clay-silt
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- LS - Limestone

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-13 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING AD-209

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-209
COORDINATES: N267070.799, E2250989.498; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.5 (ft) DATE(S) DRILLED: 1/24/23 - 1/24/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer.

Driller: Matt Light Geologist: S.Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
0				2			LEAN CLAY MOIST MEDIUM DARK BROWN Silty, few roots, weathered light brown rock at bottom of 1.5' SPT 0'-5'					
1				3								
2				3								
3												
4												
5				2			SOFT DARK BROWN WITH DARK IRON MOTTLING trace roots 5'-10'					
6			J-2	2				CL	47	22	31.4	93.2
7				1								
8												
9												
10				16			10.0 LIMESTONE HARD GREY, WHITE, BROWN SPT drive fragmented limestone					
11				31			10.5 HIGHLY WEATHERED SHALE SOFT GREY					
				50								
12												

LEGEND
- CL - USCS Low Plasticity (Lean) Clay
- LS (CHT) - Limestone - Cherty
- SH (V WTH) - Shale, Highly Weathered

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-14 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING AD-209

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-209
COORDINATES: N267070.799, E2250989.498; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +913.5 (ft) DATE(S) DRILLED: 1/24/23 - 1/24/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer.

Driller: Matt Light Geologist: S.Keeran

GROUNDWATER INFORMATION: No water encountered

FIELD DATA								LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
13							11.3 BOH Sampler Refusal BACKFILL: Grouted boring with 4 (92.6 lb) bags of portland cement	CL - USCS Low Plasticity (Lean) Clay LS (CHT) - Limestone - Cherty SH (V WTH) - Shale, Highly Weathered						U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates (mg/kg) P: Soil pH O: Other lab data defined in Remarks
14														
15														
16														
17														
18														
19								**DEFINITIONS:** R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel **ABBREVIATIONS:** J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface						
20														
21														
22														
23														
24														

REMARKS: B-14 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADU-210

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-210
COORDINATES: N267592.539, E2251092.162; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +914.7 (ft) DATE(S) DRILLED: 1/19/23 - 1/19/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: S. Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
0							LEAN CLAY SOFT MOIST BROWN Silty, Trace roots MEDIUM TO STIFF Trace fine gravel at 1'	CL - USCS Low Plasticity (Lean) Clay GC - USCS Clayey Gravel						
1					0.35									
2			W-1		0.55	5s 700psi								
3					0.35		no gravel at 3'							
4			J-2	2 2 2					CL	43	23	27.4	94.2	
5							SLIGHT IRON STAINING							
6			W-2		0.75 0.75	4s 1000psi								
7					NS		Auger chatter at 7'							
8														
9														
10							10.0 CLAYEY GRAVEL HARD DRY LIGHT BROWN small to coarse gravel up to 0.75" diam; gravel is chipped bedrock							
11				10 34 32			11.1 BOH SPT Refusal BACKFILL: Grouted boring with 3 (92.6 lb) bags of portland cement							
12														

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-15 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING AD-211

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-211
COORDINATES: N269868.472, E2251623.507; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +907.7 (ft) DATE(S) DRILLED: 2/2/23 - 2/2/23

FIELD DATA

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: Tyler Fox

GROUNDWATER INFORMATION: No water encountered

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
0							ASPHALT					
1.0							POORLY GRADED GRAVEL DRY TO MOIST LIGHT GREY sandy- fine to coarse, mostly medium; fine to coarse gravel					
2.0							SANDY LEAN CLAY VERY SOFT MOIST ORANGE BROWN few fine sand, trace fine gravel					
3				5								
4				1								
5				0								
5.0							CLAYEY GRAVEL WITH SAND DRY TO MOIST BLACK Slight Diesel Odor LOOSE possible fill; asphalt, coarse gravel, fine gravel					
6			J-1	2 3 4				GC	41	21	15.4	24.3
7												
8												
9												
10				4			MEDIUM					
11				2 4								
12												
12.0												

LEGEND

- Asphalt
- GP - USCS Poorly-graded Gravel
- CLs - USCS Low Plasticity (Lean) Clay with Sand
- GC - USCS Clayey Gravel
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- LS - Limestone

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-16 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG_A_2018_UPDATE UPPER-TURKEY-CREEK.GPJ 1/9/24

LOG OF BORING AD-211

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: AD-211
COORDINATES: N269868.472, E2251623.507; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +907.7 (ft) DATE(S) DRILLED: 2/2/23 - 2/2/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: Matt Light Geologist: Tyler Fox

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
13				2			SANDY LEAN CLAY SOFT DRY TO MOIST ORANGE BROWN fine sand					
				1								
14				2								
15							15.0					
16				4			GRAVELLY LEAN CLAY HARD DRY TO MOIST BROWN ~20% fine to coarse gravel, mostly fine					
				21								
17				17								
18				4			18.5					
19				50			LIMESTONE SOFT LIGHT GREY crushed					
							18.7					
20							BOH SPT Refusal BACKFILL: Grouted boring with 4 (92.6 lb) bags of portland cement, topped with 0.9' of asphalt patch					
21												
22												
23												
24												

LEGEND

- Asphalt
- GP - USCS Poorly-graded Gravel
- CLs - USCS Low Plasticity (Lean) Clay with Sand
- GC - USCS Clayey Gravel
- CLg - USCS Low Plasticity (Lean) Clay with Gravel
- LS - Limestone

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel

ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

LABORATORY DATA

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-16 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADU-212

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-212
COORDINATES: N269143.073, E2251164.469; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +906.0 (ft) DATE(S) DRILLED: 2/6/23 - 2/6/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: C. Broksieck Geologist: S.Keeran

GROUNDWATER INFORMATION: No water encountered

DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)
1				29 50			POORLY GRADED GRAVEL Gravel Parking Lot HARD DRY GREY limestone					
							1.5					
2							FILL Clayey Sand DRY TO MOIST DARK GREY fine to medium sand; glass shards 1.5'-5'					
3												
4												
5												
6							Sandy Silty Clay With Gravel VERY SOFT TO SOFT DARK BROWN fiberous material present; large wood piece-possible railroad tie wood; rotting odor 5'-7'					
7							7.0					
8			J-3	1 1 1			LEAN CLAY WITH SAND SOFT MOIST DARK GREY TO BLACK Silty, slight sand lens at bottom, Rotting odor 7'-7.6' BLACK Silty, RR ties pieces (some wood chips), Rotting odor SAA with metal pieces (fill)	CL	39	17	23.4	71.3
9												
10							10.0					
11			W-1		0.15 0.2 0.25	5s 500psi	GRAVELLY LEAN CLAY SOFT MOIST DARK GREY Silty, little coarse gravel, few wood chips, Rotting odor	CL	49	24	38.8	56.0
12							12.0					

LEGEND

- GP - USCS Poorly-graded Gravel
- Fill (made ground)
- CLs - USCS Low Plasticity (Lean) Clay with Sand
- CLg - USCS Low Plasticity (Lean) Clay with Gravel

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

OTHER LAB DATA
U: Unconfined Compressive Strength (tsf)
C: Confining Pressure (psi)
F: Failure Strain (%)
T: Total Sulfates (mg/kg)
P: Soil pH
O: Other lab data defined in Remarks

REMARKS: B-17 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

LOG OF BORING ADU-212

US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Upper Turkey Creek LOCATION: Kansas City, MO
PROJECT: UTC Floodwall & Levee
BORING NUMBER: ADU-212
COORDINATES: N269143.073, E2251164.469; UTM NAD 83 Meters 15N
STATION/RANGE:
ELEVATION: +906.0 (ft) DATE(S) DRILLED: 2/6/23 - 2/6/23

DRILLING METHOD(S): Diedrich-90 truck mount; 4.25 ID Hollow Stem Auger; 1 3/8" standard split spoon with plastic catcher; 140 lb autohammer; 5" shelby sampler

Driller: C. Broksieck Geologist: S.Keeran

GROUNDWATER INFORMATION: No water encountered

FIELD DATA								LABORATORY DATA					
DEPTH (ft)	SOIL SYMBOL	Other	SAMPLE/DRILL METHOD	BLOWS	TORVANE TSF (equally spaced along sample)	RC: % RQD: % Additional Field Data (See Remarks)	DESCRIPTION OF STRATUM	LEGEND	USCS SYMBOL (ASTM 2487)	ATTERBERG LIMITS LIQUID LIMIT LL	ATTERBERG LIMITS PLASTIC INDEX PI	MOISTURE CONTENT (%)	Minus 200 Sieve (%)	OTHER LAB DATA
13				3 6 50			POORLY GRADED GRAVEL VERY DENSE WET DARK GREY silty lean clay lense at 13'	GP - USCS Poorly-graded Gravel Fill (made ground) CLs - USCS Low Plasticity (Lean) Clay with Sand CLg - USCS Low Plasticity (Lean) Clay with Gravel						U: Unconfined Compressive Strength (tsf) C: Confining Pressure (psi) F: Failure Strain (%) T: Total Sulfates (mg/kg) P: Soil pH O: Other lab data defined in Remarks
14							14.0 BOH No Refusal BACKFILL: Grouted boring with 7 (92.6 lb) bags of portland cement							
15														
16														
17														
18														
19														
20														
21														
22														
23														
24														

DEFINITIONS:
R: Blow Count Refusal = >50 blows/1/2 foot for SPT, > 100 blows for drive barrel
ABBREVIATIONS:
J-# = Jar, W-# = Extruded & Waxed Shelby sample, T-# = Unextruded Shelby Tube, B-# = 1 gallon bag, S-# = 50 pound sack, M-# = Moisture Tin, P-# = Pull; r = sample recovery; RC - Rock Core Recovery; RQD - Rock Quality Designation; NS = No Sample; Occ = occasional(ly); Approx = approximately; Diam = diameter, NP=non-plastic, BGS = below ground surface

REMARKS: B-17 (secondary nomenclature given by engineer)
DD=Dry Density |Final Review by AML
COORDINATE SOURCE: Survey

APPENDIX A.2 – CFS BORROW AREA BORING LOGS

2025 CFS Subsurface Investigation

In 2025, Cook, Flatt & Strobel Engineers (CFS) conducted a subsurface investigation at the planned Merriam Grand Station North project site, a portion of which the City of Merriam has identified as a potential borrow source for Upper Turkey Creek levee construction. A total of five borings (Borings B1 through B5) were conducted in the area identified as a potential borrow source. The potential borrow site is in the north central portion of the lot bounded by West 62nd Street to the north, Slater Street to the east, Shawnee Mission Parkway to the south, and Ikea Way to the west in Merriam, Kansas.

The planned construction within the potential borrow area consists of a multistory apartment building with a below grade parking garage. Per the geotechnical report by CFS dated 12 February 2025, the finished floor elevation of the lowest parking level is expected to be at 1008 feet above sea level. Based on Google Earth topography, the site slopes down from north to south from approximate elevation 1015 feet to 1005 feet, with about half the site below 1008 feet. The potential borrow soils consist of excess material from this excavation.

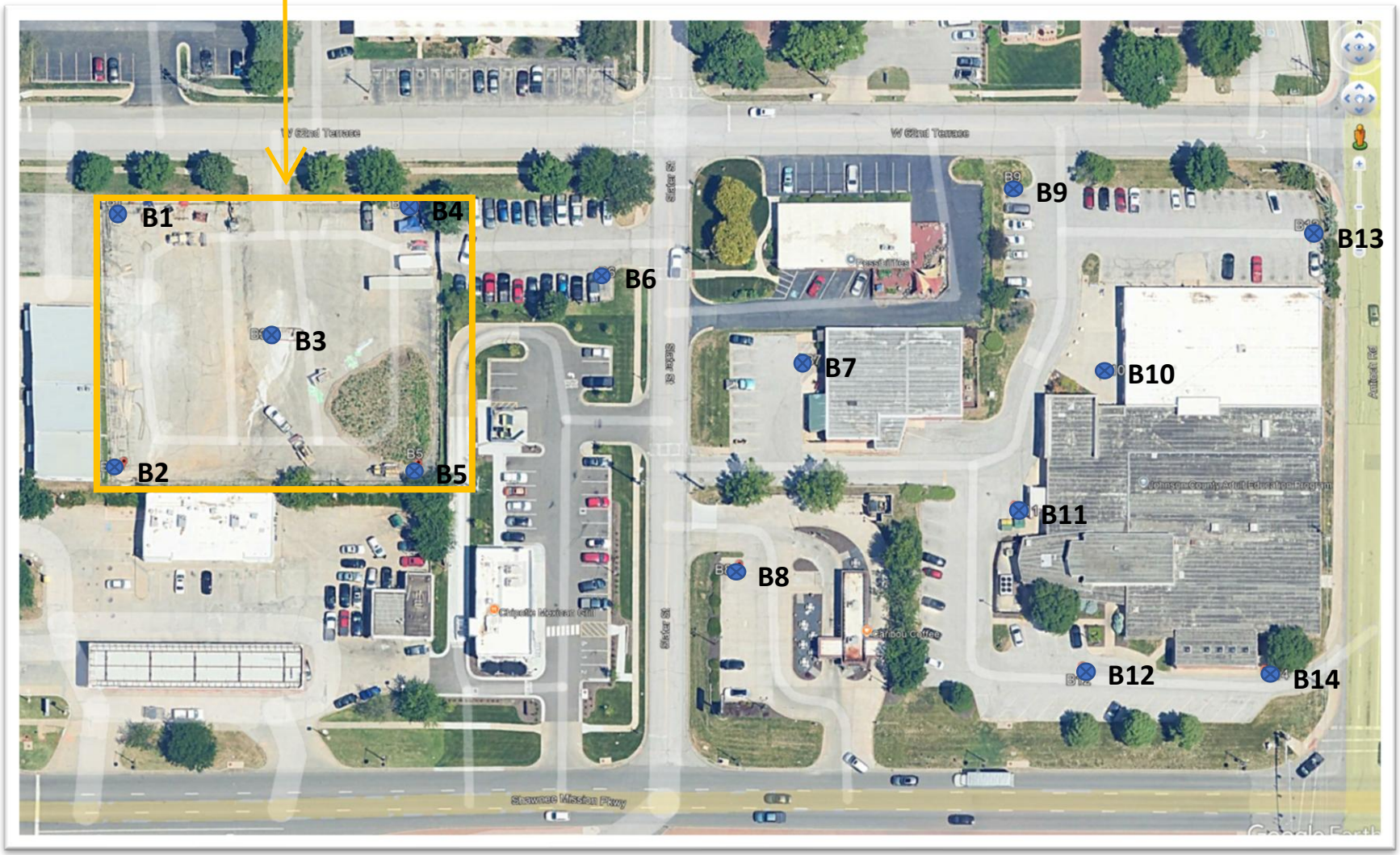
The site is planned to be excavated in December 2025 with any excess material stockpiled. The City of Merriam will inform USACE of how much material is stockpiled for use as borrow.

Split spoon samples and rock cores were taken at the boring locations. The boring logs show an average of three feet of lean clay fill below the asphalt parking surface and aggregate base. The fill is underlain by natural lean clay, with weathered limestone encountered at 4 to 11 feet below grade. In two of the five borings, the fill material was described as with gravel. Atterberg limits were tested on one sample, with a Liquid Limit of 38 and Plasticity Index of 24.

Based on the visual classifications given in the boring logs and on the limited laboratory testing, the material is assumed to be suitable for use as impervious fill. It is unclear if there will be a significant amount of borrow available from this site. If the project is delayed for a significant period, the City of Merriam may use this borrow elsewhere.

Boring Location Plan and pertinent Boring Logs are included in this Appendix.

UTC Proposed Borrow Area



CFS ENGINEERS

1100 W. Cambridge Circle Dr, Ste 700
Kansas City, Kansas 66103

Project: MERRIAM GRAND STATION NORTH		Project #: 24-5392	Figure 2: BORING LOCATION PLAN
Project Location:	Merriam, Kansas	Comments:	
Client:	Merriam Grand Station North		
Date:	2/12/2025		

N
W E
S

CFS Engineers

BORING NUMBER B1

CLIENT Merriam Grand Station North
PROJECT NAME MERRIAM GRAND STATION NORTH
PROJECT NUMBER 24-5392
PROJECT LOCATION Merriam, Kansas
DATE STARTED 1/30/25 **COMPLETED** 1/30/25
GROUND ELEVATION 1020 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR CFS Engineers
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING --- No Free Water Encountered
LOGGED BY CM **CHECKED BY** JE
AT END OF DRILLING --- No Free Water Encountered
NOTES
AFTER DRILLING --- No Free Water Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	UNCONFINED COMP (psf)
0		2-inches of ASPHALT										
		(GW) LEAN CLAY, (CL) dark grayish brown, moist, with roots and gravel (FILL)	SPT 1	61	4-4-4 (8)	2		20				
		LEAN CLAY, (CL) brown and grayish brown, moist, stiff	SPT 2	94	4-5-5 (10)	2.5		26				
5		(CL) reddish brown, with iron striations below 6'	SPT 3	100	3-5-5 (10)	2		23	38	14	24	
			SPT 4	100	3-4-6 (10)	2.5		23				
10		LIMESTONE, moderately weathered										

Refusal at 10.8 feet.
Bottom of borehole at 10.8 feet.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - 2/12/25 10:16 - G:\SHARED DRIVES\245392\GEOTECH\EXPLORATION REPORTS\24-5392.GPJ

CFS Engineers

BORING NUMBER B2

CLIENT Merriam Grand Station North
PROJECT NAME MERRIAM GRAND STATION NORTH
PROJECT NUMBER 24-5392
PROJECT LOCATION Merriam, Kansas
DATE STARTED 2/3/25 **COMPLETED** 2/3/25
GROUND ELEVATION 1008 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR CFS Engineers
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight Augers/NQ-Size Rock Core
AT TIME OF DRILLING --- No Free Water Encountered
LOGGED BY CM **CHECKED BY** JE
AT END OF DRILLING --- No Free Water Encountered
NOTES
AFTER DRILLING --- No Free Water Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	UNCONFINED COMP (psf)
0		2-inches of ASPHALT										
		(GW) 5.5-inches of BASE ROCK										
		LEAN CLAY, (CL) dark brown and reddish brown, dry to moist, stiff	SPT 1	61	3-4-4 (8)	2		23				
		LIMESTONE, highly weathered, with clay seams										
		Moderately weathered to slightly weathered										
5		SHALE, slightly weathered, gray and olive	RC 2	81 (60)		3.5						
		LIMESTONE, moderately weathered to slightly weathered										
		Unweathered										
10		SHALE, unweathered, gray	RC 3	100 (92)								
		With LIMESTONE stringers	RC 4	121 (121)								

Refusal at 5.7 feet.
Bottom of borehole at 14.0 feet.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - 2/12/25 10:16 - G:\SHARED DRIVES\245392\GEOTECH\EXPLORATION REPORTS\24-5392.GPJ

CFS Engineers

BORING NUMBER B3

CLIENT Merriam Grand Station North
PROJECT NAME MERRIAM GRAND STATION NORTH
PROJECT NUMBER 24-5392
PROJECT LOCATION Merriam, Kansas
DATE STARTED 1/30/25 **COMPLETED** 1/30/25
GROUND ELEVATION 1013 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR CFS Engineers
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING --- No Free Water Encountered
LOGGED BY CM **CHECKED BY** JE
AT END OF DRILLING --- No Free Water Encountered
NOTES
AFTER DRILLING --- No Free Water Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	UNCONFINED COMP (psf)
0		6-inches of ASPHALT										
		(GW) 4-inches of BASE ROCK										
		LEAN CLAY, (CL) brown and blueish gray, moist, with gravel (FILL)	SPT 1	50	3-4-5 (9)	2.5		24				
		(CL) brown, with fine sand below 3'	SPT 2	78	4-6-9 (15)	2.5		24				
5		LEAN CLAY, (CL) reddish brown and brown, moist, stiff	SPT 3	100	5-7-6 (13)	1.5		43				
		(CL) highly weathered LIMESTONE fragments below 8.5'	SPT 4	56	6-8-4 (12)	1		42				
10		LIMESTONE, moderately weathered										

Refusal at 11.2 feet.
Bottom of borehole at 11.2 feet.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - 2/12/25 10:16 - G:\SHARED DRIVES\245392\GEOTECH\EXPLORATION REPORTS\24-5392.GPJ

CFS Engineers

BORING NUMBER B4

CLIENT Merriam Grand Station North
PROJECT NAME MERRIAM GRAND STATION NORTH
PROJECT NUMBER 24-5392
PROJECT LOCATION Merriam, Kansas
DATE STARTED 2/3/25 **COMPLETED** 2/3/25
GROUND ELEVATION 1020 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR CFS Engineers
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight Augers/NQ-Size Rock Core
AT TIME OF DRILLING --- No Free Water Encountered
LOGGED BY CM **CHECKED BY** JE
AT END OF DRILLING --- No Free Water Encountered
NOTES
AFTER DRILLING --- No Free Water Encountered

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	UNCONFINED COMP (psf)
0		4-inches of ASPHALT										
		(GW) 4-inches of BASE ROCK										
		LEAN CLAY, (CL) dark brown, dry, (FILL)	SPT 1	44	3-5-9 (14)	4.5		20				
		LEAN CLAY, (CL) brown, dry, stiff, with trace of fine sand	SPT 2	50	5-6-10 (16)	4.5+		17				
5												
		(CL) reddish brown below 6'	SPT 3	83	6-8-8 (16)	4.5+		17				
		LIMESTONE, highly weathered, with clay seams	SPT 4	56	11-37-31 (68)	4		30				
10												
		Moderately weathered	RC 5	88 (59)								
15		SHALE, unweathered, gray										
		LIMESTONE, unweathered										
20		SHALE, unweathered, gray, with LIMESTONE stringers	RC 6	98 (72)								
			RC 7	100 (63)								

Refusal at 14.0 feet.
Bottom of borehole at 24.0 feet.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - 2/12/25 10:16 - G:\SHARED DRIVES\245392\GEOTECH\EXPLORATION REPORTS\24-5392.GPJ

CFS ENGINEERS

CFS Engineers

BORING NUMBER B5

CLIENT Merriam Grand Station North
PROJECT NAME MERRIAM GRAND STATION NORTH
PROJECT NUMBER 24-5392
PROJECT LOCATION Merriam, Kansas
DATE STARTED 1/30/25 **COMPLETED** 1/30/25
GROUND ELEVATION 1013 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR CFS Engineers
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight Augers
AT TIME OF DRILLING --- No Free Water Encountered
LOGGED BY CM **CHECKED BY** JE
AT END OF DRILLING --- No Free Water Encountered
NOTES
AFTER DRILLING --- No Free Water Encountered

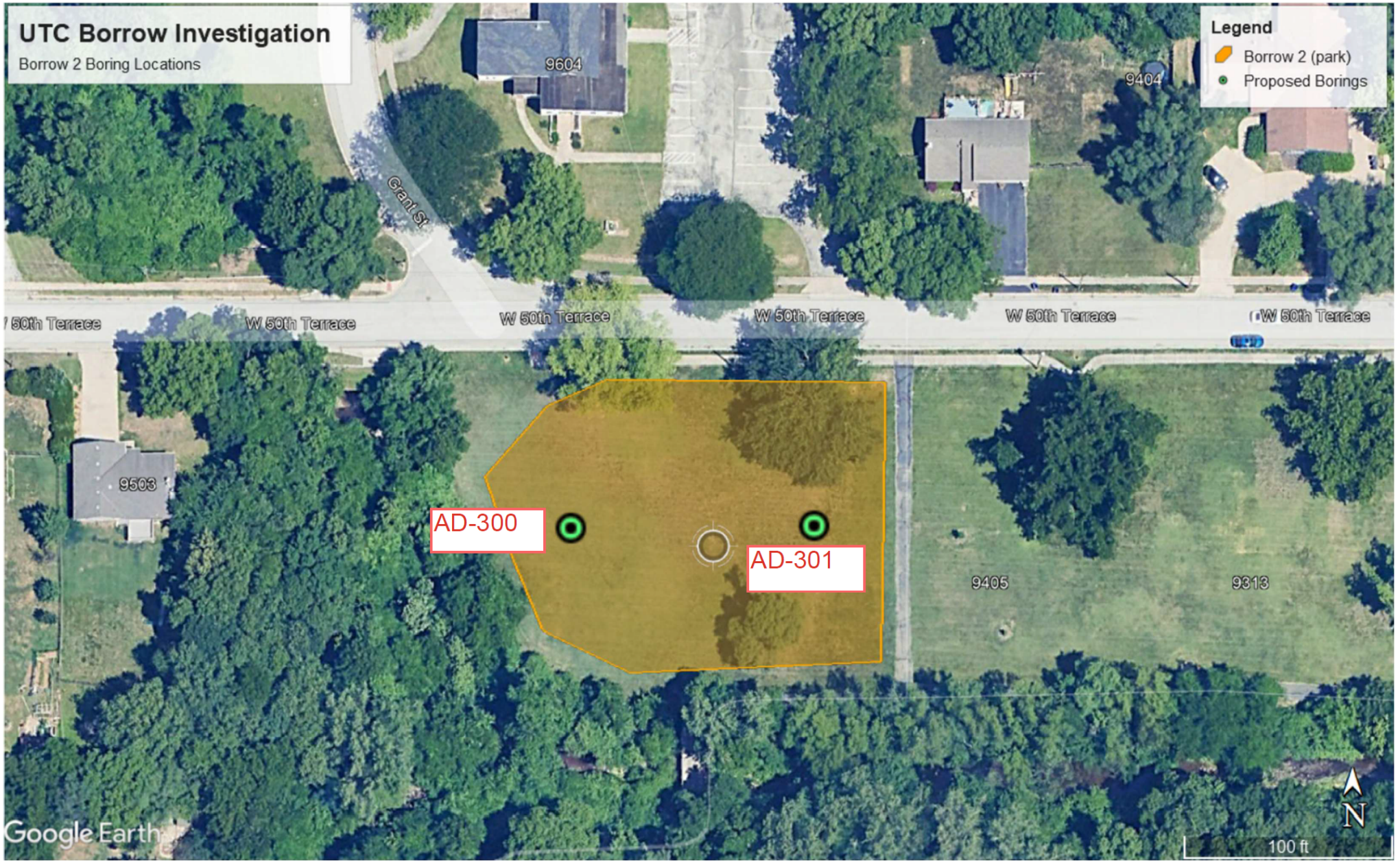
DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	UNCONFINED COMP (psf)
0		4-inches of ASPHALT										
		(GW) 4-inches of BASE ROCK										
		LEAN CLAY, (CL) brown and gray, moist, stiff	SPT 1	83	3-5-7 (12)	2.75		30				
		(CL) reddish brown below 3'	SPT 2	100	3-4-6 (10)	1.5		25				
5		(CL) highly weathered LIMESTONE fragments below 6.5'	SPT 3	33	9-12-21 (33)	2.5		25				
		LIMESTONE, highly weathered to moderately weathered, with clay seams	SPT 4	100	50/1"							
10												

Refusal at 10.6 feet.
Bottom of borehole at 10.6 feet.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - 2/12/25 10:16 - G:\SHARED DRIVES\245392\GEOTECH\EXPLORATION REPORTS\24-5392.GPJ

APPENDIX A.3 – USACE BORROW AREA BORING LOGS

Map of Boring Locations



DRILLING LOG

DIVISION: Northwestern	INSTALLATION: Turkey Creek		Sheet 1 of 1
1. PROJECT: Upper Turkey Creek, Kansas City, MO	10. COORDINATE SYSTEM: NAD83 / UTM zone 15N		VERTICAL: NAVD88
	LOCATION COORDINATES: N: 4322329.58 E: 353257.61		LOCATION METHOD: Garmin GPS
2. HOLE NUMBER: AD-300	3. DRILLING AGENCY: USACE-NWK	11. DATE BORING: STARTED Dec 29 2025	COMPLETED Dec 29 2025
4. NAME OF DRILLER: Lane Wisdom		12. HAMMER TYPE: Automatic, 30" drop, 140 lb	EFFICIENCY (%)
5. EQUIPMENT: Diedrich D-90 ATV		13. SIZE AND TYPE OF BIT: 3.75" HSA	
6. DIRECTION OF BORING: Vertical	DEG FROM VERTICAL: 0	BEARING:	14. ELEVATION SURFACE: 910.0'
			15. ELEVATION GROUND WATER: 896.5'
7. THICKNESS OF OVERBURDEN: 15.0'			16. TOTAL NUMBER CORE BOXES
8. DEPTH DRILLED INTO ROCK			17. SIGNATURE AND TITLE OF INSPECTOR: S. Orndoff, Geologist; APPROVED BY:
9. TOTAL DEPTH OF BORING: 15.0'			

DEPTH	BLOWS/0.5ft	N	LEGEND	LAYER DEPTH ELEV.	CLASSIFICATION OF MATERIALS (Description)	% REC	SAMP NO.	LABORATORY: Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
	2, 3, 4	7			LEAN CLAY (CL) brown, dry, trace roots	80									
				2.20											
	28, 10, 7	17		907.80	White, dry, Fill: highly weathered limestone, FILL	60									
5	16, 20, 10	30				20	Bulk 1	4	20	76	46	25	21	CL	
	4, 2, 3	5				0									
				9.50											
10	1, 1, 2	3		900.50	SANDY FAT CLAY (CH) brown, dry, soft	100	Jar 5	13	21	66	55	34	26	CH	
	2, 2, 2	4				40									
				13.50											
	9, 21, 25	46		896.50	LEAN CLAY (CL) gray, wet, hard	100	Jar 7		2	98	47	23	17	CL	
15				15.00											
				895.00	Borehole finished at 15.0 ft. No Refusal. Backfilled with bentonite grout.										
20															
25															
30															

905 900 895 890 885 880

DRILLING LOG

DIVISION: Northwestern	INSTALLATION: Turkey Creek		Sheet 1 of 1
1. PROJECT: Upper Turkey Creek, Kansas City, MO	10. COORDINATE SYSTEM: NAD83 / UTM zone 15N		VERTICAL
	LOCATION COORDINATES: N: 4322329.25 E: 353293.19		LOCATION METHOD: Garmin GPS
2. HOLE NUMBER: AD-301	3. DRILLING AGENCY: USACE-NWK	11. DATE BORING: STARTED Dec 29 2025	COMPLETED Dec 29 2025
4. NAME OF DRILLER: Lane Wisdom		12. HAMMER TYPE: Automatic, 30" drop, 140 lb	EFFICIENCY (%)
5. EQUIPMENT: Diedrich D-90 ATV		13. SIZE AND TYPE OF BIT: 3.75" HSA	
6. DIRECTION OF BORING: Vertical	DEG FROM VERTICAL: 0	BEARING:	14. ELEVATION SURFACE: 908.0'
			15. ELEVATION GROUND WATER: 898.0'
7. THICKNESS OF OVERBURDEN: 15.0'			16. TOTAL NUMBER CORE BOXES
8. DEPTH DRILLED INTO ROCK			17. SIGNATURE AND TITLE OF INSPECTOR: S. Orndoff, Geologist — APPROVED BY:
9. TOTAL DEPTH OF BORING: 15.0'			

DEPTH	BLOWS/0.5ft	N	LEGEND	LAYER DEPTH ELEV.	CLASSIFICATION OF MATERIALS (Description)	% REC	SAMP NO.	LABORATORY: Gravel	Sand	Fines	LL	PI	MC	ASTM Class	REMARKS
	3 3 11	14			LEAN CLAY WITH SAND (CL) brown, dry, very stiff, iron oxide staining	33	Bulk 1	3	18	80	45	25	22	CL	
	4 3 2	5			*2 ft: changes to STIFF* *2.3 - 2.5 ft: gravel layer*	60	Jar 2	4	10	87	47	25	20	CL	
5	7 6 5	11			*4 - 4.7 ft: Gravel layer, coarse, white, dry*	67	Jar 5	0	5	94	47	24	20	CL	903
	4 3 3	6			*6 ft: 1 piece of 2" gravel, changes to MEDIUM*	67	Bulk 1								
	2 2 2	4			*8 ft: changes to SOFT*	100									
10	6 8 3	11		10.00 898.00	LEAN CLAY (CL) light brown, wet, stiff	100									898
	11 20 38	58			*13.5 ft: changes to HARD*	0									
15				15.00 893.00	Borehole finished at 15.0 ft. Backfilled with bentonite chips.										893
20															888
25															883
30															878

APPENDIX A.4 – LABORATORY DATA

UTC Alignment
Laboratory Testing Results

LETTER OF TRANSMITTAL

15620 West 113th Street
Lenexa, Kansas 66219
Phone: (913) 492-7777
FAX: (913) 492-7443

Attention:	**Derek Jensen**	Date:	**April 28, 2023**
Company:	**U.S. Army Corps of Engineers**		
	Kansas City District Office, EDG-D, Room 526		
	601 East 12th St.		
	Kansas City, Missouri 64106-2896		
Project Name:	**Upper Turkey Creek Project Geotechnical Testing**		
Project Location:	**Merriam, Kansas**		

Terracon Project No: **02221476** Client Project No: **W912DQ23F1003**

We are transmitting
- ☒ Herewith
- ☐ Under separate cover

☐ Field Data ☒ Laboratory Data ☐ Report

Regarding:

Geotechnical laboratory testing for the Upper Turkey Creek Project including USCS Classification, Water Content, Consolidation, Unconfined Compression, UU Triaxial, CU Triaxial and Uniaxial Compressive Strength Core testing.

Comments:

As previously communicated in emails, the following issues were found with the samples:

- Boring No. ADU-197; Sample W-1; 5 to 7 feet: Due to the substantial presence of rock in the sample, we were unable to conduct the CU Triaxial testing.
- Boring No. ADU-200; Sample W-1; 5 to 7 feet: Due to the substantial presence of rock in the sample, we were unable to conduct the CU Triaxial testing.
- Boring No. ADU-204; Sample W-1 2 to 4 feet: The limited quantity of sample recovery allowed us to obtain only two samples for CU Triaxial testing.

We have not been asked to interpret the data or make design and/or construction recommendations based on the data and cannot assume responsibility or liability for interpretation of this data by others.

Troy B. Bandy, PE

SUMMARY OF LABORATORY RESULTS

BORING ID	Depth (Ft.)	Soil Classification USCS	Liquid Limit	Plastic Limit	Plasticity Index	Water Content (%)	Unit Weight Dry Density (pcf)	Triaxial Testing Attached (UU and CU)	Consolidation Test Attached	Rock Core Compression Attached	% Fines
202	5	SANDY LEAN CLAY(CL)	42	21	21	23.2	96.6	X CU			67.2
202	10	SANDY SILT(ML)	NP	NP	NP	54.1					67.6
202	12.1									X	
203	0.5	SILTY GRAVEL with SAND(GM)	NP	NP	NP	7.6					15.1
204	2	CLAYEY GRAVEL with SAND(GC)	38	20	18	22.2	99.7	X UU	X		47.4
204	8	SILTY GRAVEL with SAND(GM)	NP	NP	NP	8.7					17.7
205	10	SANDY LEAN CLAY(CL)	38	22	16	33.0					67.2
205	15.1									X	
205	18.4									X	
206	3	LEAN CLAY(CL)	41	22	19	21.5					93.2
206	5	LEAN CLAY with SAND(CL)	40	21	19	18.1	103.1	X CU	X		80.0
207	5	LEAN CLAY(CL)	44	21	23	25.0	92.5	X UU			98.1
207	7	SANDY LEAN CLAY(CL)	46	20	26	22.1					62.4
208	1	LEAN CLAY(CL)	45	22	23	23.0					97.7
208	1.1	LEAN CLAY(CL)	45	22	23	25.4	94.3	X CU	X		96.7
209	5	LEAN CLAY(CL)	47	25	22	31.4					93.2
210	3	LEAN CLAY(CL)	43	20	23	27.4					94.2
211	5	CLAYEY GRAVEL with SAND(GC)	41	20	21	15.4					24.3
212	7	LEAN CLAY with SAND(CL)	39	22	17	23.4					71.3
212	10	GRAVELLY LEAN CLAY with SAND(CL)	49	25	24	38.8					56.0
201A	10	POORLY GRADED SAND with SILT(SP-SM)	NP	NP	NP	18.6					11.1
201A	15	LEAN CLAY(CL)	44	22	22	12.7					91.3
AD-198	5	LEAN CLAY(CL)	46	21	25	24.5					99.2
AD-198	10	LEAN CLAY(CL)	38	20	18	28.9	124.2	X CU	X		95.1
ADU-197	5	GRAVELLY LEAN CLAY(CL)	43	21	22	14.6	113.1		X		69.3
ADU-197	10	LEAN CLAY(CL)	47	19	28	26.0	95.8	X UU			92.3
ADU-197	12	LEAN CLAY(CL)	48	20	28	30.4					92.0
ADU-200	1										59.0

PROJECT: USACE Upper Turkey Creek Laboratory Testing

SITE:

Terracon
15620 W 113th St
Lenexa, KS
PH. 913-492-7777 FAX. 913-492-7443

PROJECT NUMBER: 02221476

CLIENT: U.S. Army Corps of Engineers
Kansas City, Missouri

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. SMART LAB SUMMARY-LANDSCAPE_A_02221476.GPJ TERRACON_DATATEMPLATE.GDT 10/5/23

SUMMARY OF LABORATORY RESULTS

BORING ID	Depth (Ft.)	Soil Classification USCS	Liquid Limit	Plastic Limit	Plasticity Index	Water Content (%)	Unit Weight Dry Density (pcf)	Triaxial Testing Attached (UU and CU)	Consolidation Test Attached	Rock Core Compression Attached	% Fines
ADU-200	5	GRAVELLY LEAN CLAY with SAND(CL)	40	21	19	13.4					59.7
ADU-200	7	CLAYEY GRAVEL(GC)	45	23	22	25.6					40.9
ADUC-196	7	GRAVELLY FAT CLAY with SAND(CH)	58	22	36	7.7					53.8
ADUC-196	28.3									X	

PROJECT: USACE Upper Turkey Creek Laboratory Testing

SITE:

Terracon
15620 W 113th St
Lenexa, KS
PH. 913-492-7777 FAX. 913-492-7443

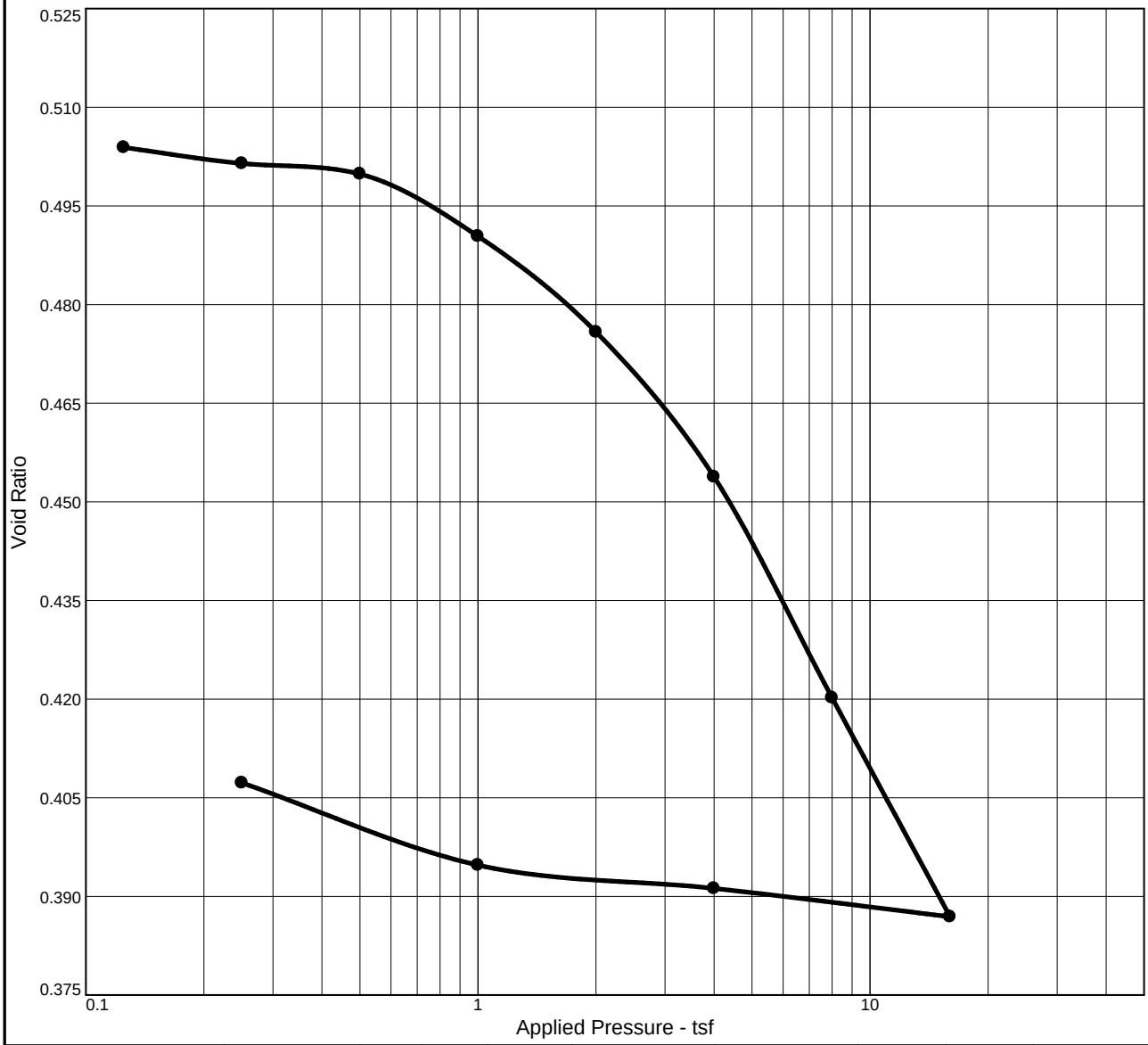
PROJECT NUMBER: 02221476

CLIENT: U.S. Army Corps of Engineers
Kansas City, Missouri

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. SMART LAB SUMMARY-LANDSCAPE_A 02221476.GPJ TERRACON_DATATEMPLATE.GDT 10/5/23

ASTM D2435
Incremental Loading Consolidation Test

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
78.2 %	14.6 %	113.0	43	22	2.7	0.39	3.5	0.11	0.01	0.503

MATERIAL DESCRIPTION	USCS	AASHTO
GRAVELLY LEAN CLAY	CL	NA

Project No. 02221476 **Client:** USACE

Project: Upper Turkey Creek

Source of Sample: 197 **Depth:** 5-7 feet **Sample Number:** W-1

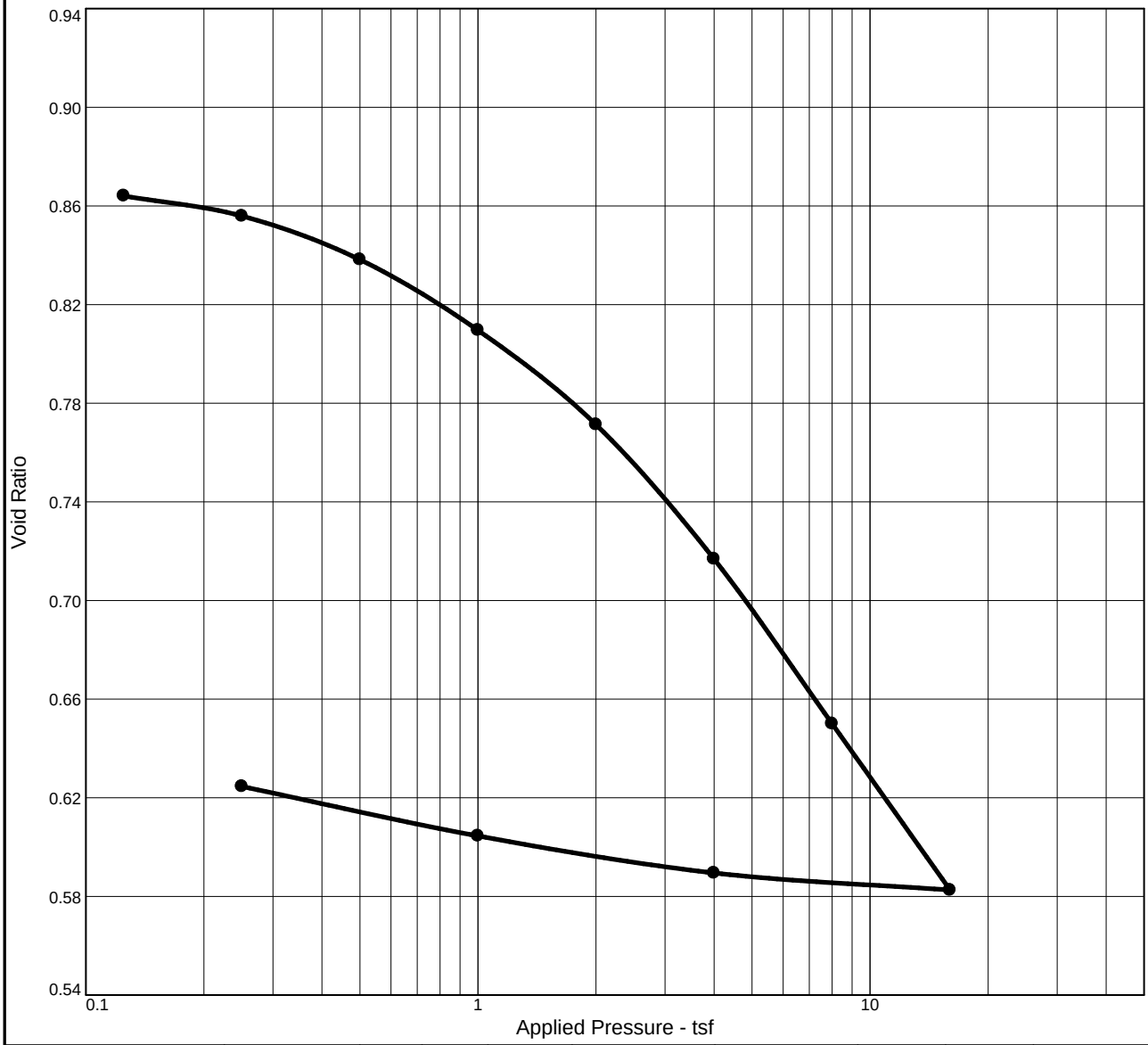
Terracon Consultants, Inc.

Lenexa, KS

Remarks:
The sample had a significant amount of limestone in it.

Tested By: SM

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
89.2 %	28.9 %	124.2	38	18	2.7	0.88	3.2	0.23	0.03	0.876

MATERIAL DESCRIPTION	USCS	AASHTO
LEAN CLAY	CL	NA

Project No. 02221476 **Client:** USACE

Project: Upper Turkey Creek

Source of Sample: 198 **Depth:** 10-12 feet **Sample Number:** W-1

Terracon Consultants, Inc.

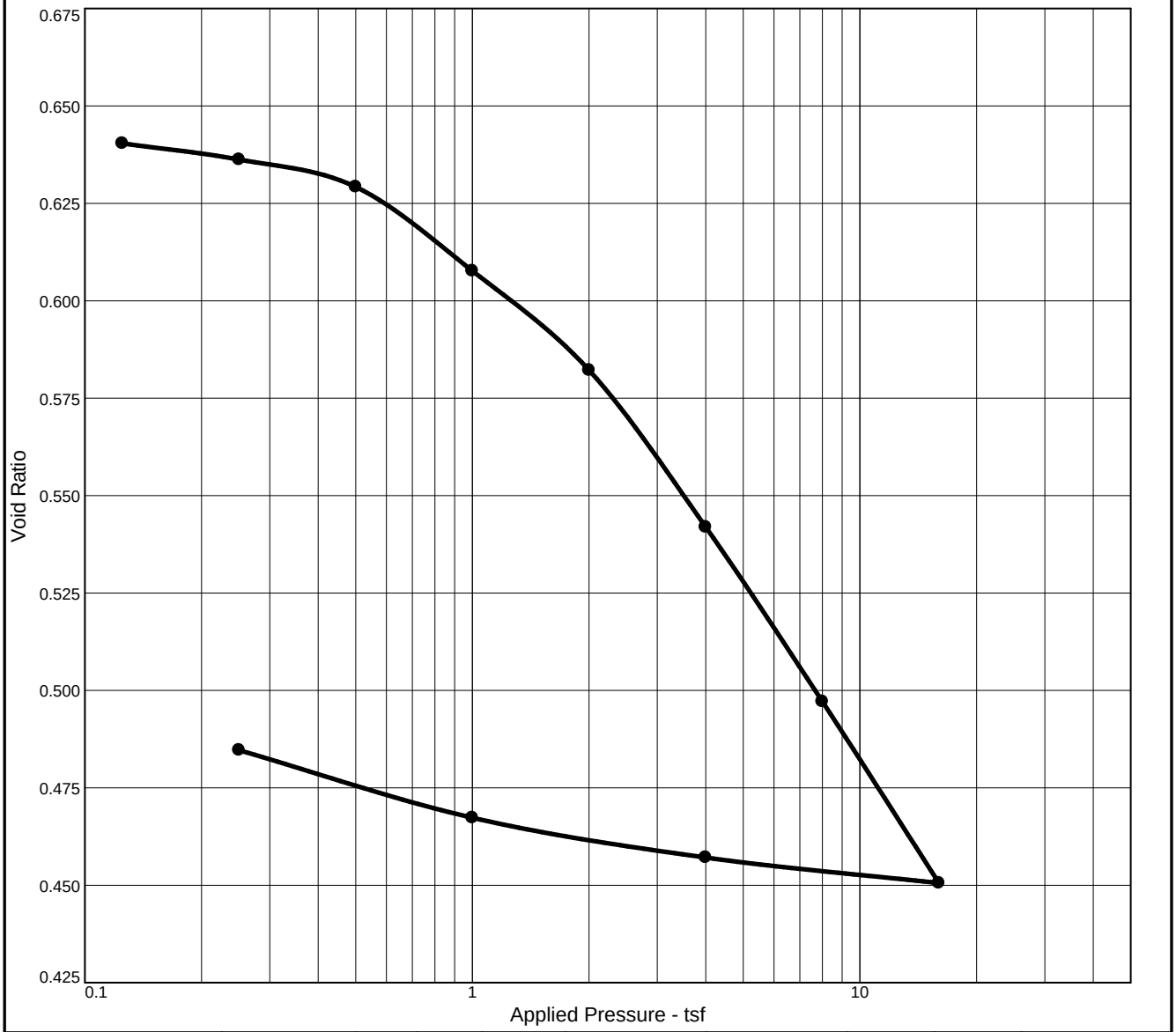
Lenexa, KS

Remarks:

None

Tested By: SM/SR

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
94.0 %	22.2 %	99.5	38	18	2.7	0.18	2.9	0.15	0.02	0.639

MATERIAL DESCRIPTION	USCS	AASHTO
CLAYEY GRAVEL with SAND	GC	NA

Project No. 02221476 **Client:** USACE

Project: Upper Turkey Creek

Source of Sample: 204 **Depth:** 2-4 feet **Sample Number:** W-1 2.8 psi

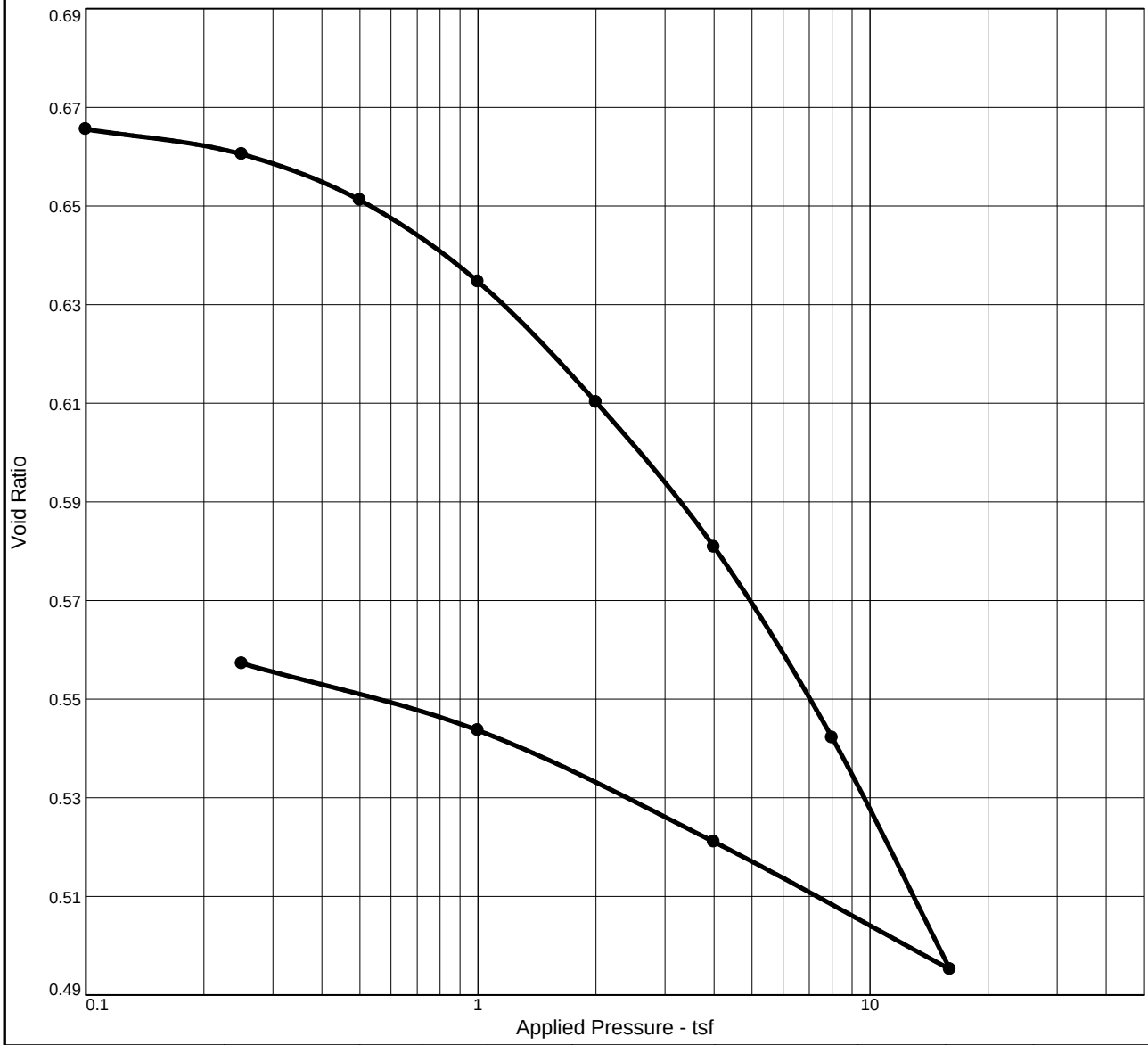
Terracon Consultants, Inc.

Lenexa, KS

Remarks:
None

Tested By: SM/SR

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
73.3 %	18.1 %	103.1	40	19	2.7	0.37	4.1	0.16	0.03	0.667

MATERIAL DESCRIPTION	USCS	AASHTO
LEAN CLAY with SAND	CL	NA

Project No. 02221476 **Client:** USACE

Project: Upper Turkey Creek

Source of Sample: 206 **Depth:** 5-7 feet **Sample Number:** W-2

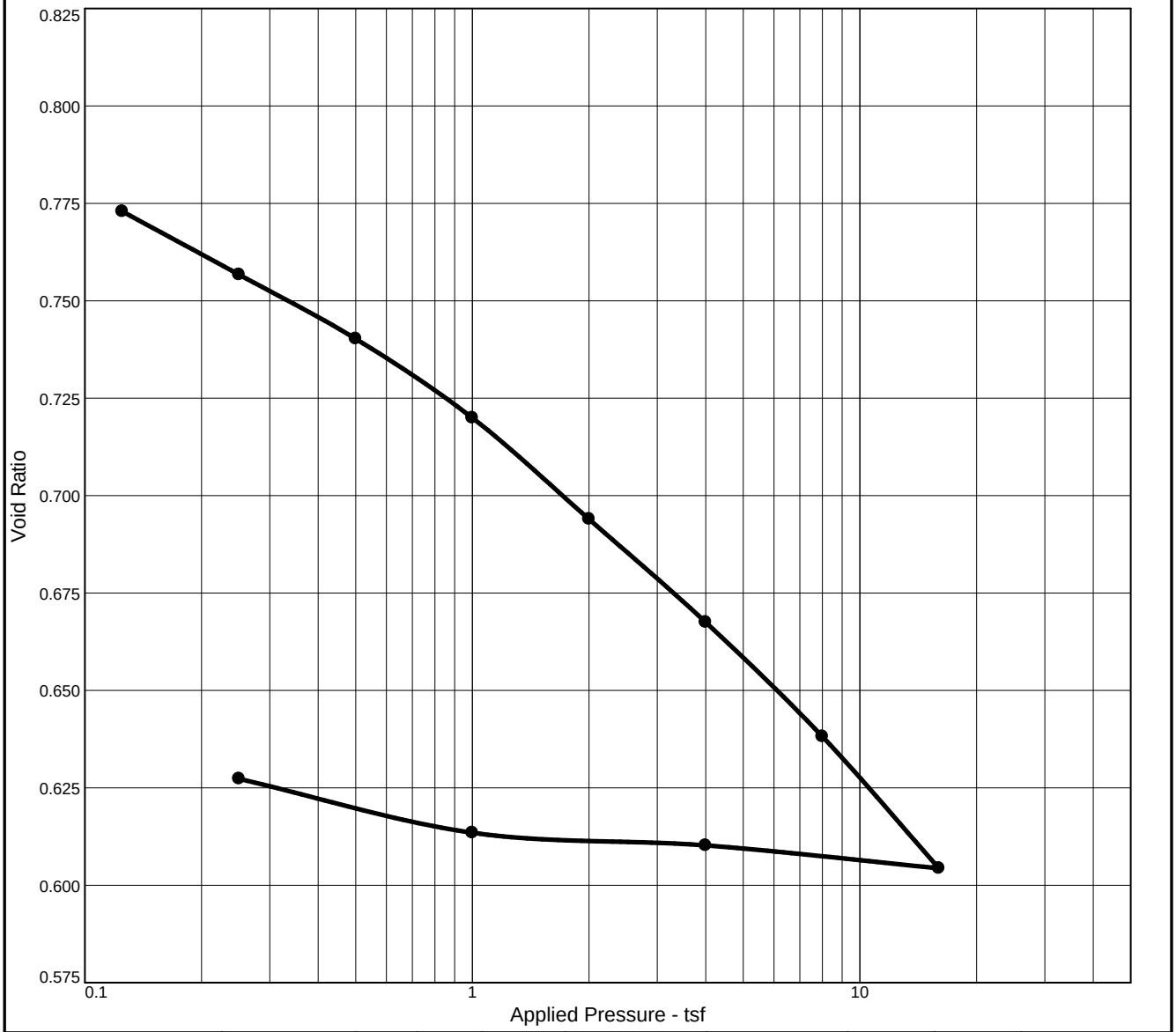
Terracon Consultants, Inc.

Lenexa, KS

Remarks:
Sample had trace gravel.

Tested By: SM

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
86.9 %	25.4 %	94.3	45	23	2.7	0.12	2.3	0.11	0.01	0.791

MATERIAL DESCRIPTION	USCS	AASHTO
LEAN CLAY	CL	NA

Project No. 02221476 **Client:** USACE

Project: Upper Turkey Creek

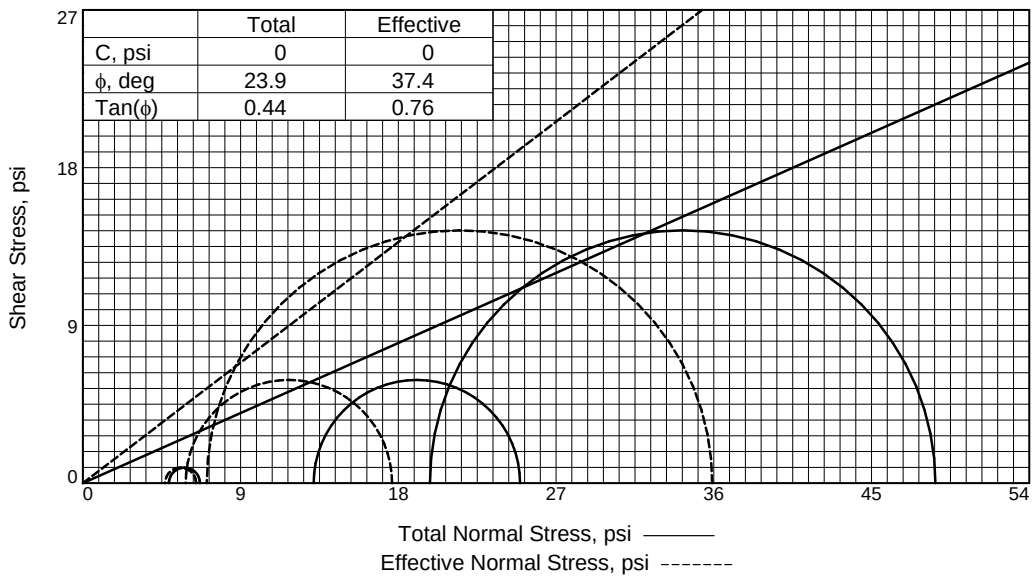
Source of Sample: 208 **Depth:** 1-3 **Sample Number:** W-1

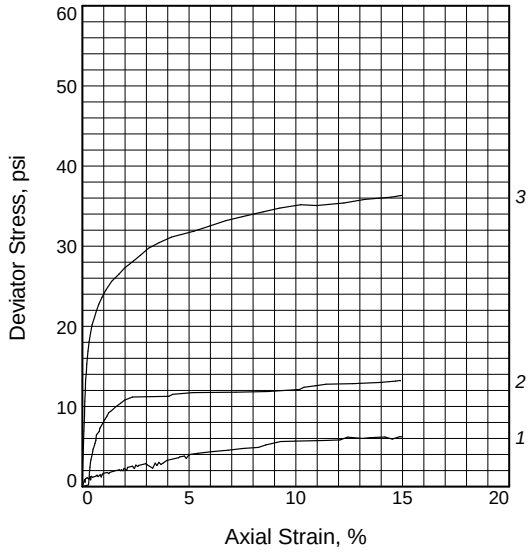
Terracon Consultants, Inc.

Lenexa, KS

Remarks:
Sample had trace organics.

Tested By: SM





Sample No.		1	2	3
Initial	Water Content, %	28.1	27.3	28.1
	Dry Density, pcf	96.5	93.4	93.6
	Saturation, %	101.5	91.7	94.5
	Void Ratio	0.7464	0.8040	0.8011
	Diameter, in.	1.33	1.36	1.35
	Height, in.	2.92	3.05	2.81
At Test	Water Content, %	27.6	29.8	29.7
	Dry Density, pcf	96.5	93.4	93.6
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.7464	0.8040	0.8011
	Diameter, in.	1.33	1.36	1.35
	Height, in.	2.92	3.05	2.81
Strain rate, in./min.		0.001	0.001	0.001
Back Pressure, psi		52.1	49.4	58.2
Cell Pressure, psi		57.0	62.6	78.0
Fail. Stress, psi		1.8	11.8	28.8
Excess Pore Pr., psi		0.2	7.3	12.7
Ult. Stress, psi				
Excess Pore Pr., psi				
$\bar{\sigma}_1$ Failure, psi		6.5	17.6	35.9
$\bar{\sigma}_3$ Failure, psi		4.7	5.9	7.1

Type of Test:
CU with Pore Pressures
Sample Type: 5" diameter wax tube
Description: LEAN CLAY

LL= 38 **PL=** 20 **PI=** 18
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to 1.4" diameter specimen.

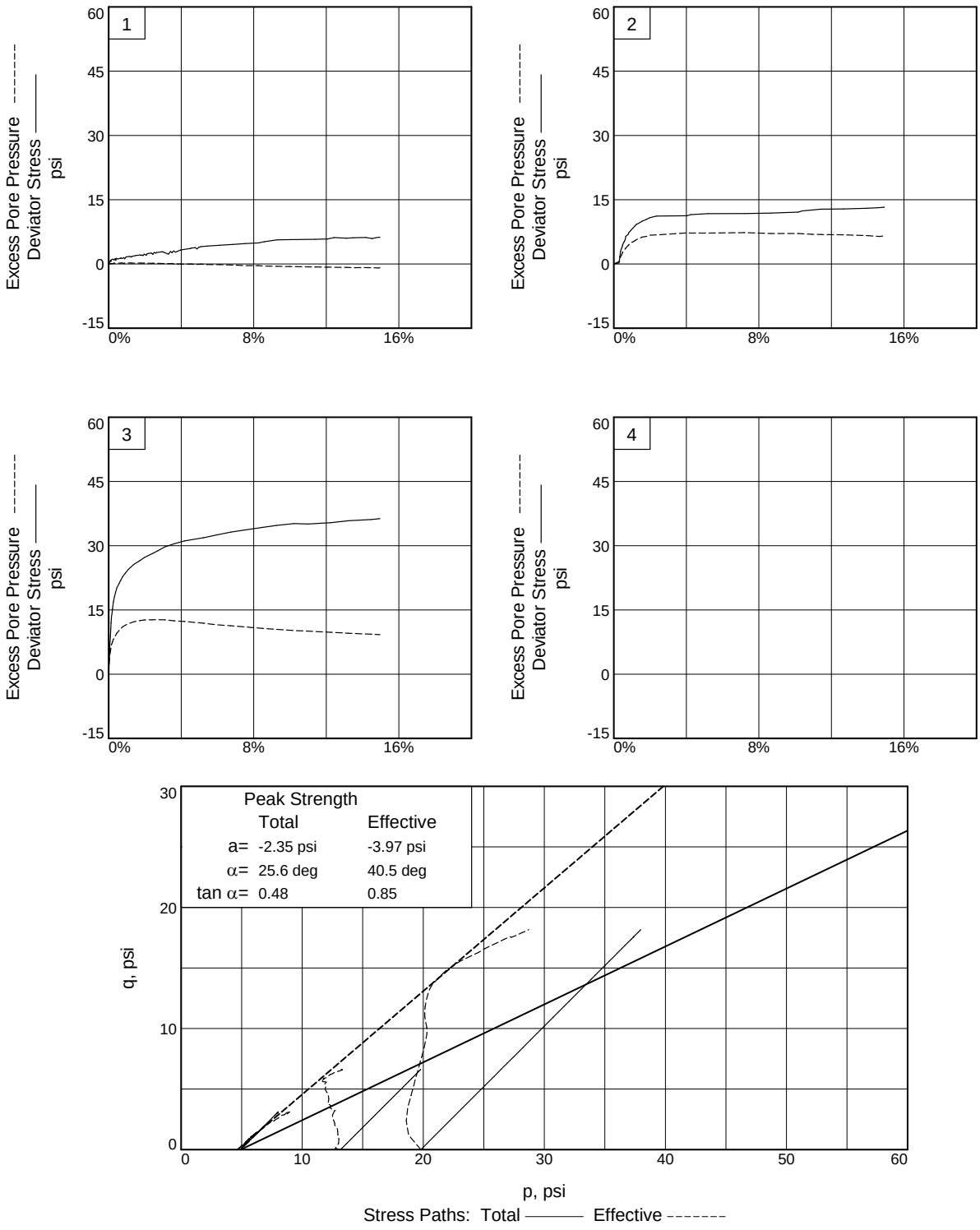
Client: USACE

Project: Upper Turkey Creek

Source of Sample: 198 **Depth:** 10-12 feet
Sample Number: W-1
Proj. No.: 02221476 **Date Sampled:** 3/21/2023

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

Tested By: SR



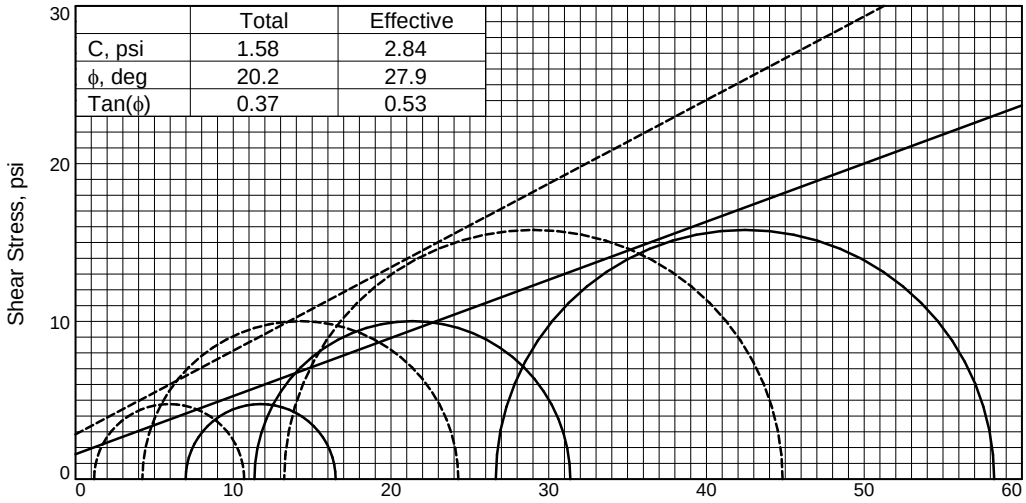
Client: USACE
Project: Upper Turkey Creek
Source of Sample: 198 **Depth:** 10-12 feet **Sample Number:** W-1
Project No.: 02221476

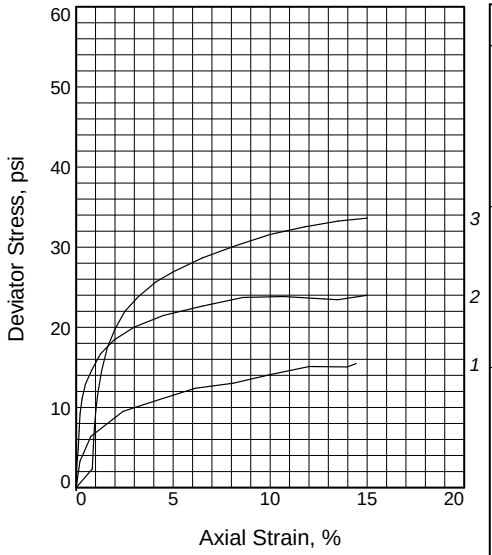
Terracon Consultants, Inc.

Tested By: SR

Triaxial CU
ASTM D4767

	Total	Effective
C, psi	1.58	2.84
ϕ, deg	20.2	27.9
Tan(ϕ)	0.37	0.53





Sample No.		1	2	3
Initial	Water Content, %	23.2	23.2	23.2
	Dry Density, pcf	96.8	98.4	98.8
	Saturation, %	84.4	87.6	88.5
	Void Ratio	0.7406	0.7137	0.7064
	Diameter, in.	1.34	1.34	1.33
	Height, in.	2.80	2.79	2.98
At Test	Water Content, %	27.4	26.4	26.2
	Dry Density, pcf	96.8	98.4	98.8
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.7406	0.7137	0.7064
	Diameter, in.	1.34	1.34	1.33
	Height, in.	2.80	2.79	2.98
Strain rate, in./min.		0.002	0.000	0.001
Back Pressure, psi		55.0	55.0	55.0
Cell Pressure, psi		62.0	66.3	81.6
Fail. Stress, psi		9.5	20.0	31.6
Excess Pore Pr., psi		5.8	7.1	13.4
Ult. Stress, psi				
Excess Pore Pr., psi				
$\bar{\sigma}_1$ Failure, psi		10.7	24.3	44.8
$\bar{\sigma}_3$ Failure, psi		1.2	4.2	13.2

Type of Test:
CU with Pore Pressures
Sample Type: 5" diameter wax tube
Description: SANDY LEAN CLAY(CL)

LL= 42 **PL=** 21 **PI=** 21
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen. Sample had limestone fragments in it.

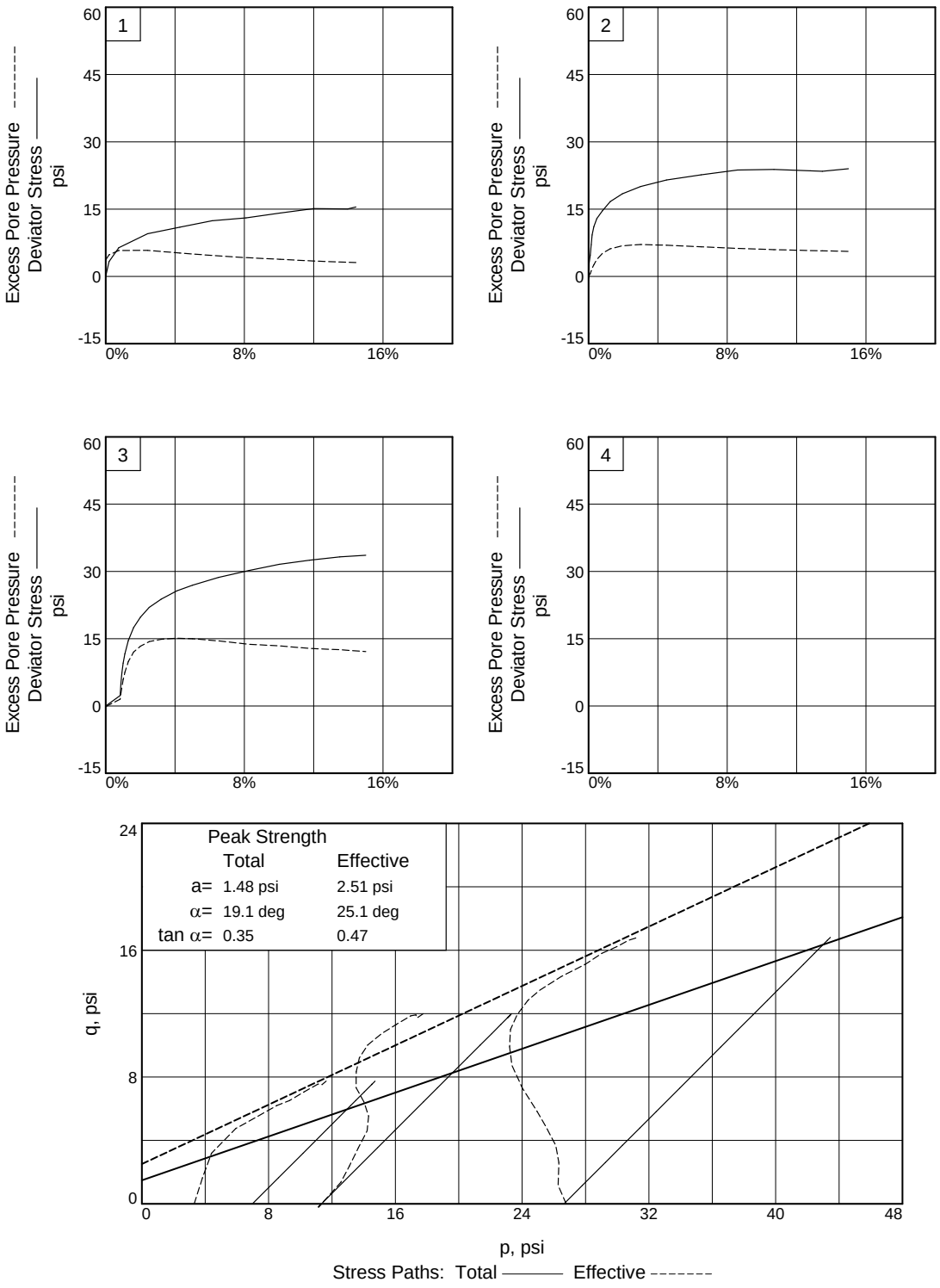
Client: USACE

Project: Upper Turkey Creek

Source of Sample: 202 **Depth:** 5-7 feet
Sample Number: W-1
Proj. No.: 02221476 **Date Sampled:** 3/6/23

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

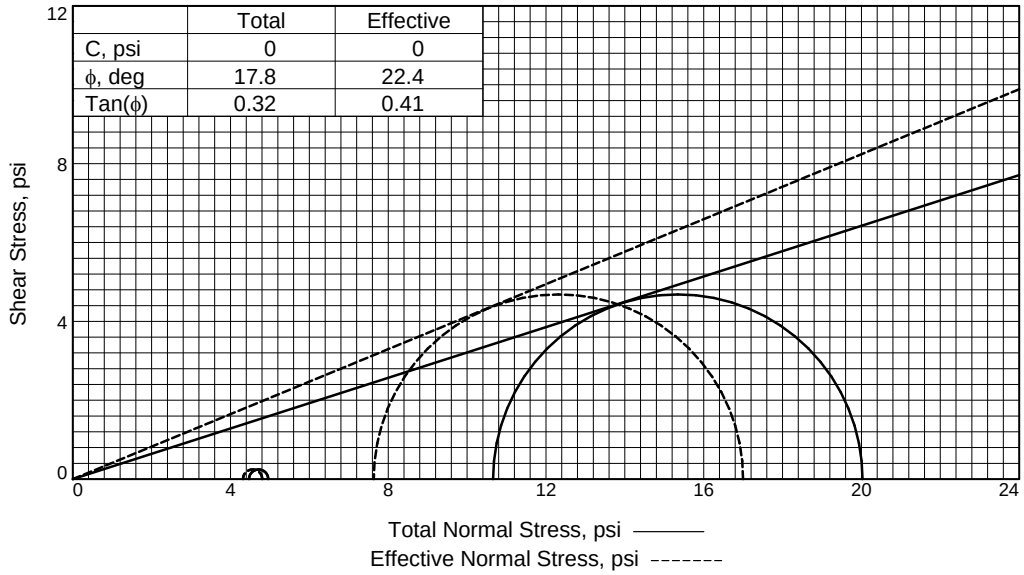
Tested By: SM/SR

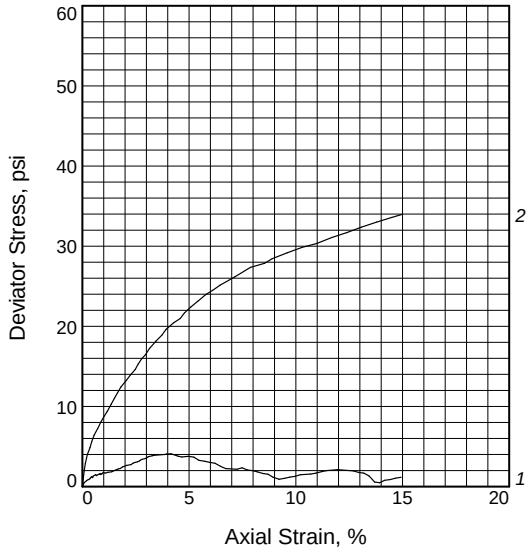


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 202 **Depth:** 5-7 feet **Sample Number:** W-1
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SM/SR





Sample No.		1	2
Initial	Water Content, %	21.6	21.6
	Dry Density, pcf	96.3	100.2
	Saturation, %	77.7	85.5
	Void Ratio	0.7499	0.6816
	Diameter, in.	1.34	1.35
	Height, in.	3.10	2.50
At Test	Water Content, %	27.8	25.2
	Dry Density, pcf	96.3	100.2
	Saturation, %	100.0	100.0
	Void Ratio	0.7499	0.6816
	Diameter, in.	1.34	1.35
	Height, in.	3.10	2.50
Strain rate, in./min.		0.001	0.001
Back Pressure, psi		48.5	54.0
Cell Pressure, psi		53.0	64.7
Fail. Stress, psi		0.5	9.4
Excess Pore Pr., psi		0.2	3.0
Ult. Stress, psi			
Excess Pore Pr., psi			
$\bar{\sigma}_1$ Failure, psi		4.8	17.0
$\bar{\sigma}_3$ Failure, psi		4.3	7.6

Type of Test:
CU with Pore Pressures

Sample Type: 5" diameter wax tube

Description: LEAN CLAY with SAND

LL= 40 **PL=** 21 **PI=** 19

Assumed Specific Gravity= 2.7

Remarks: Due to the presence of gravel, only two samples were able to be tested from the recovery. The first sample was affected by the gravel and yielded irregular testing results.

Client: USACE

Project: Upper Turkey Creek

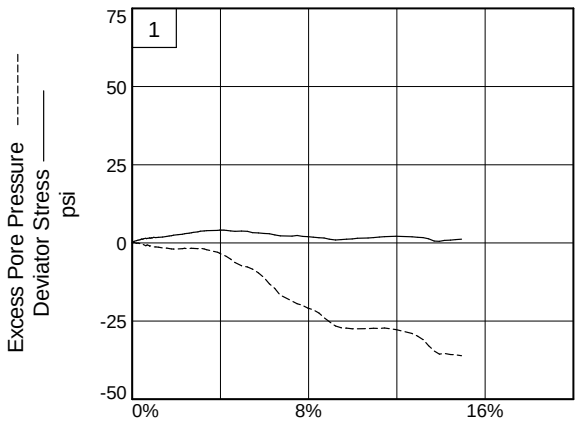
Source of Sample: 206 **Depth:** 5-7 feet

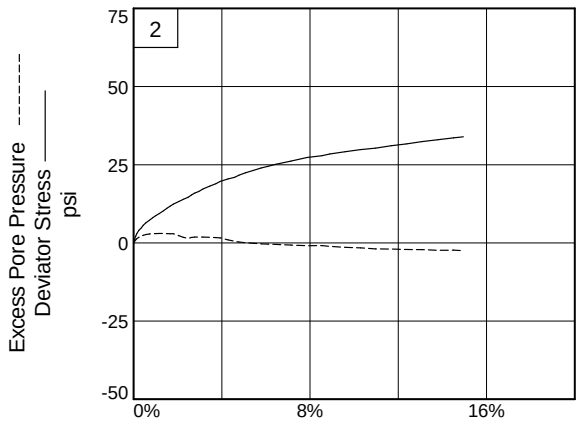
Sample Number: W-2

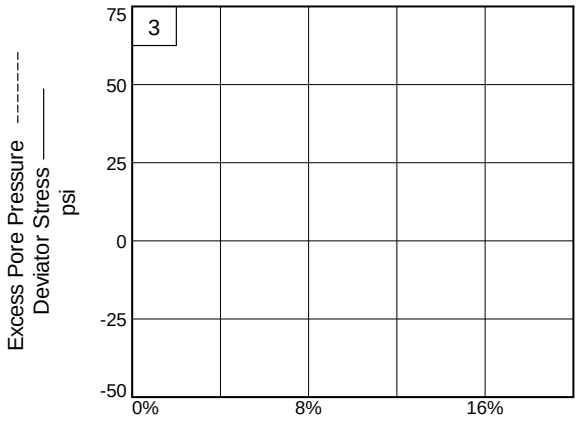
Proj. No.: 02221476 **Date Sampled:** 3/8/23

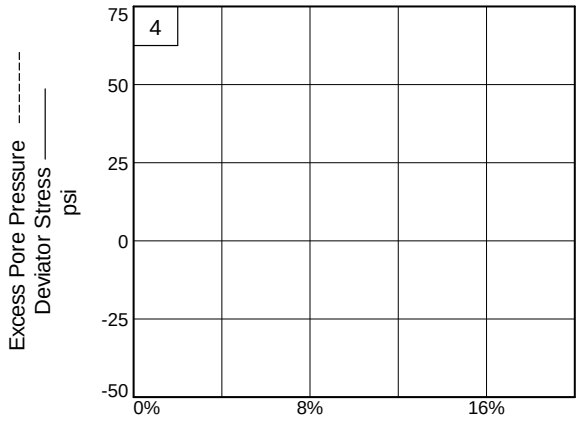
TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

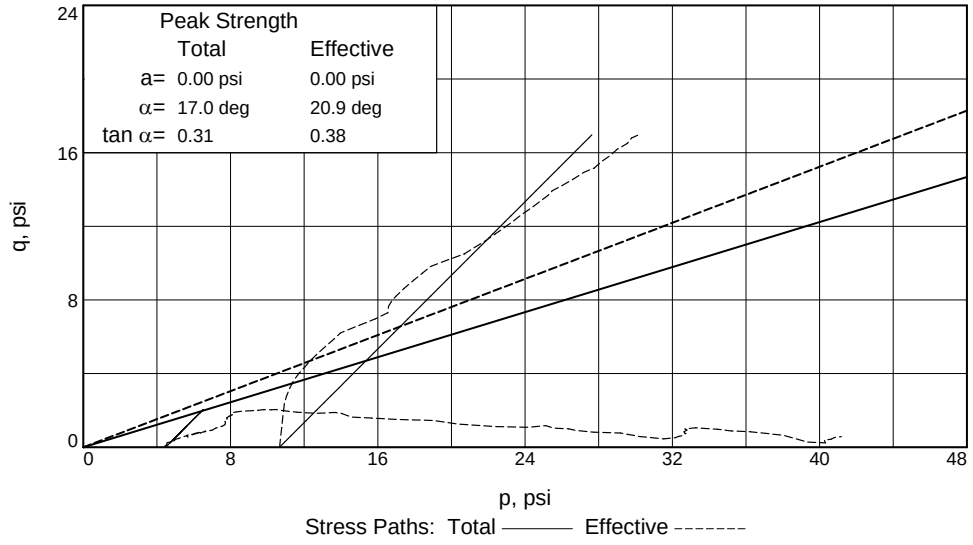
Tested By: SM/SR







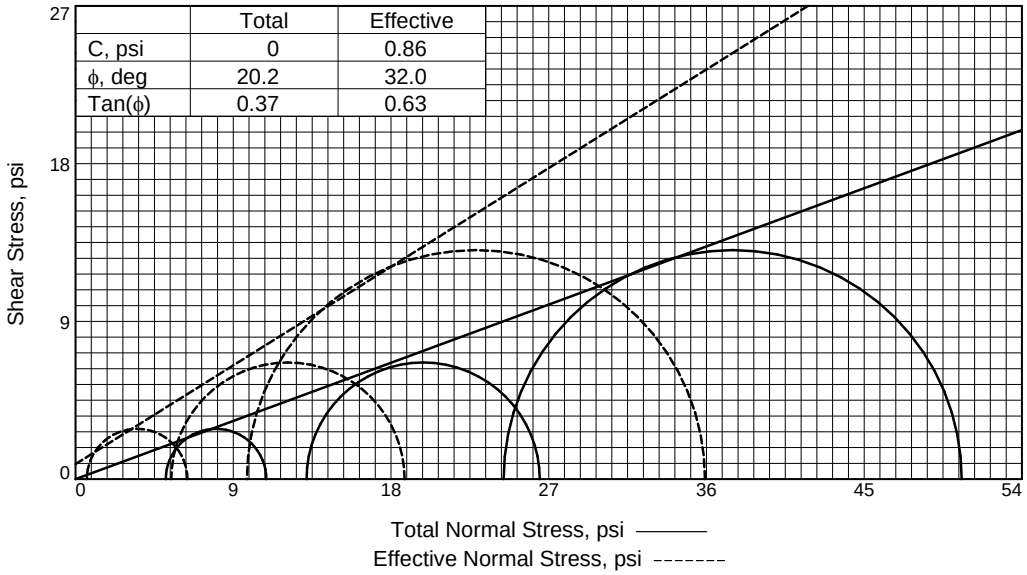


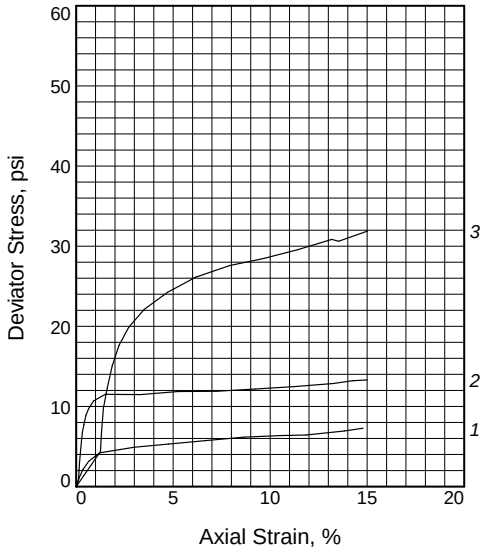


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 206 **Depth:** 5-7 feet **Sample Number:** W-2
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SM/SR





Sample No.		1	2	3
Initial	Water Content, %	30.8	30.8	30.8
	Dry Density, pcf	87.6	84.2	93.5
	Saturation, %	89.9	82.9	103.5
	Void Ratio	0.9241	1.0028	0.8034
	Diameter, in.	1.35	1.36	1.35
	Height, in.	2.85	2.79	2.94
At Test	Water Content, %	34.2	37.1	29.8
	Dry Density, pcf	87.6	84.2	93.5
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.9241	1.0028	0.8034
	Diameter, in.	1.35	1.36	1.35
	Height, in.	2.85	2.79	2.94
Strain rate, in./min.		0.001	0.001	0.001
Back Pressure, psi		53.9	49.3	50.0
Cell Pressure, psi		59.1	62.5	74.4
Fail. Stress, psi		5.7	13.3	26.1
Excess Pore Pr., psi		4.5	7.7	14.6
Ult. Stress, psi				
Excess Pore Pr., psi				
$\bar{\sigma}_1$ Failure, psi		6.4	18.8	35.9
$\bar{\sigma}_3$ Failure, psi		0.6	5.5	9.8

Type of Test:
CU with Pore Pressures
Sample Type: 5" diameter wax tube
Description: LEAN CLAY

LL= 45 **PL=** 22 **PI=** 23
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

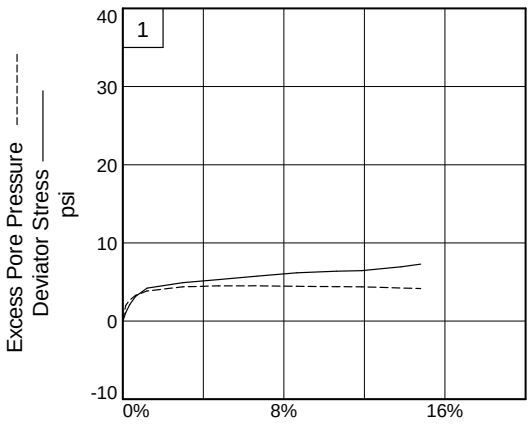
Client: USACE

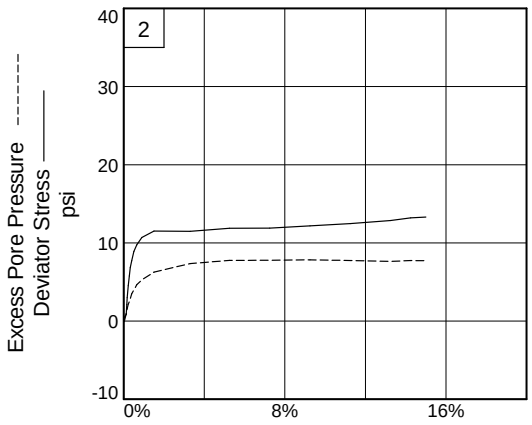
Project: Upper Turkey Creek

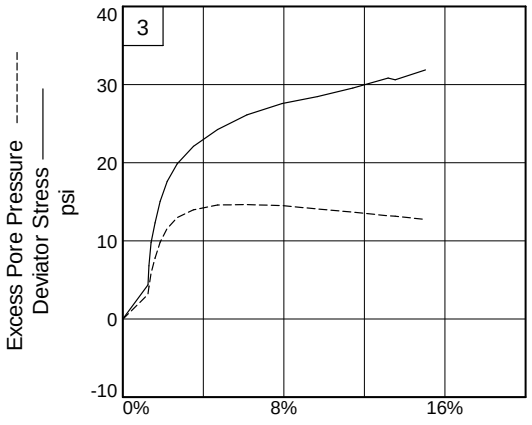
Source of Sample: 208 **Depth:** 1-3
Sample Number: W-1
Proj. No.: 02221476 **Date Sampled:** 3/9/23

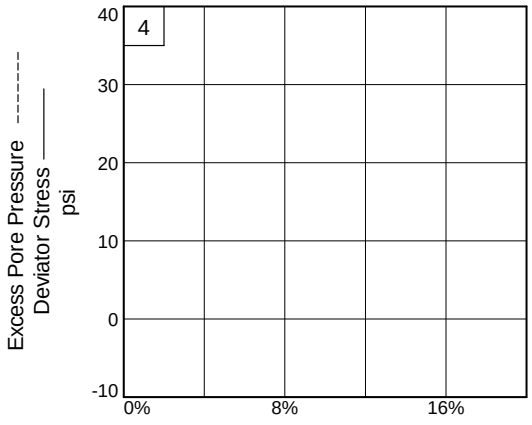
TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

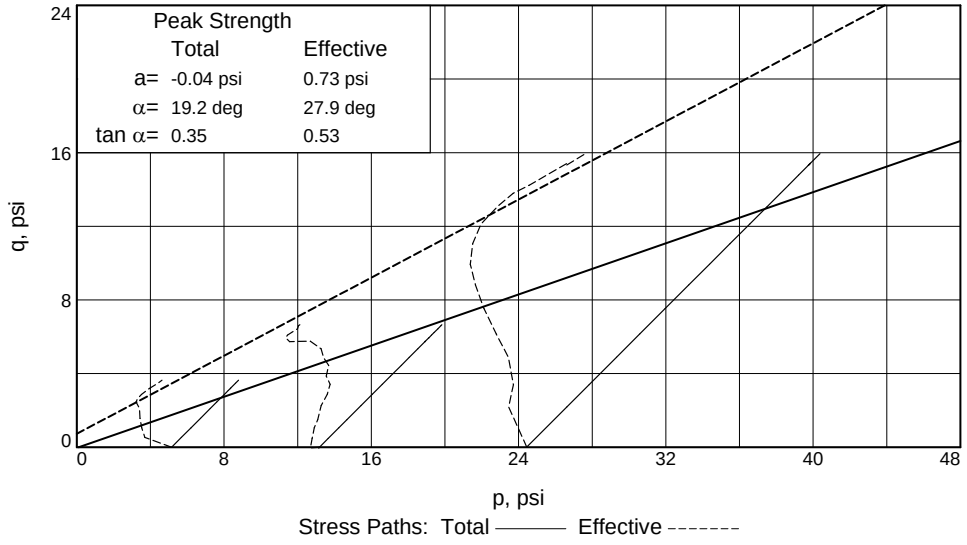
Tested By: SM/SR









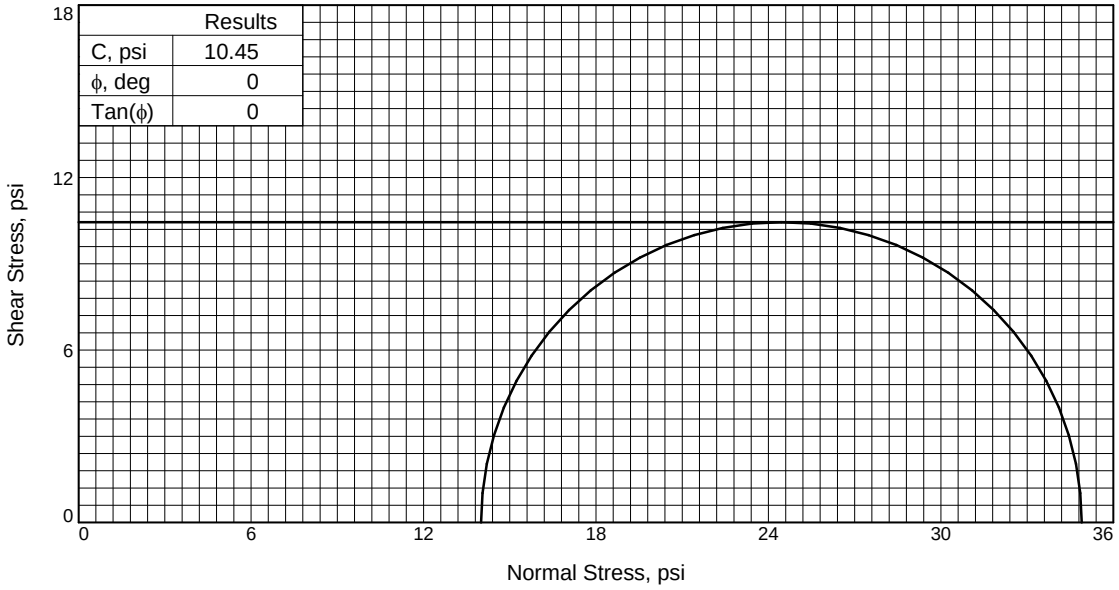


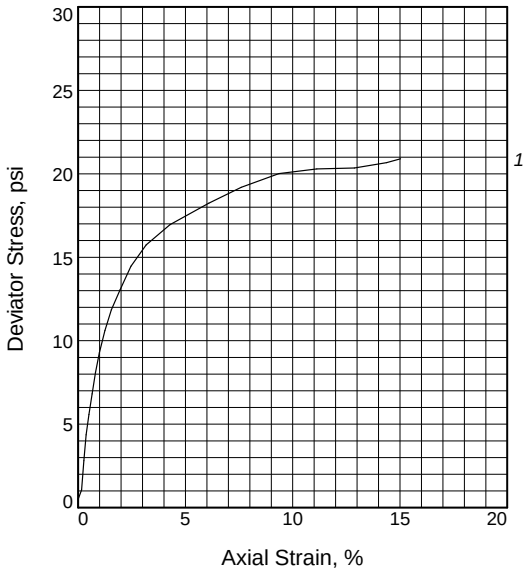
Client: USACE
Project: Upper Turkey Creek
Source of Sample: 208 **Depth:** 1-3 **Sample Number:** W-1
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SM/SR

Triaxial UU
ASTM D2850





Sample No.		1
Initial	Water Content, %	26.0
	Dry Density, pcf	97.0
	Saturation, %	95.3
	Void Ratio	0.7373
	Diameter, in.	1.35
	Height, in.	3.10
At Test	Water Content, %	26.0
	Dry Density, pcf	97.0
	Saturation, %	95.3
	Void Ratio	0.7373
	Diameter, in.	1.35
	Height, in.	3.10
Strain rate, in./min.		0.136
Back Pressure, psi		0.0
Cell Pressure, psi		14.0
Fail. Stress, psi		20.9
Ult. Stress, psi		
σ_1 Failure, psi		34.9
σ_3 Failure, psi		14.0

Type of Test:
Unconsolidated Undrained

Sample Type: 5" diameter wax tube

Description: LEAN CLAY(CL)

LL= 47 **PL=** 19 **PI=** 28

Assumed Specific Gravity= 2.7

Remarks: Trimmed down to a 1.4" diameter specimen.

Client: USACE

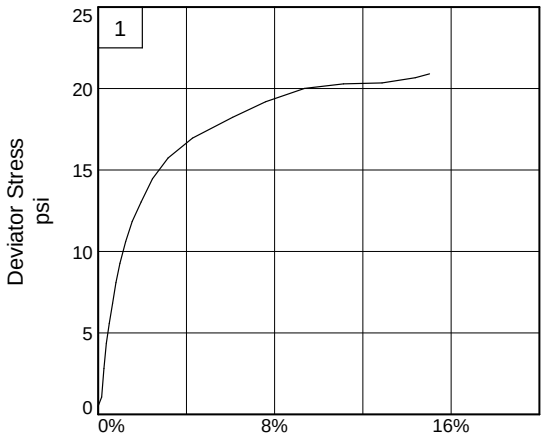
Project: Upper Turkey Creek

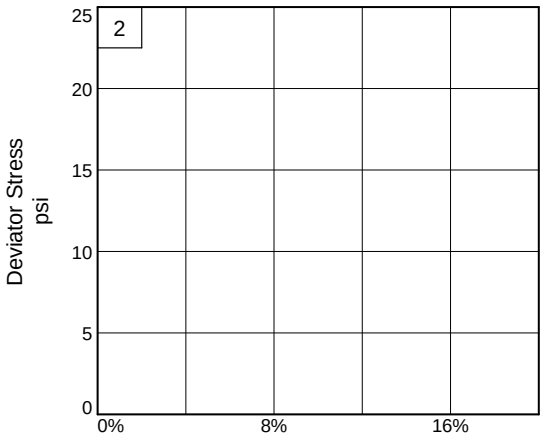
Source of Sample: 197 **Depth:** 10-12 feet

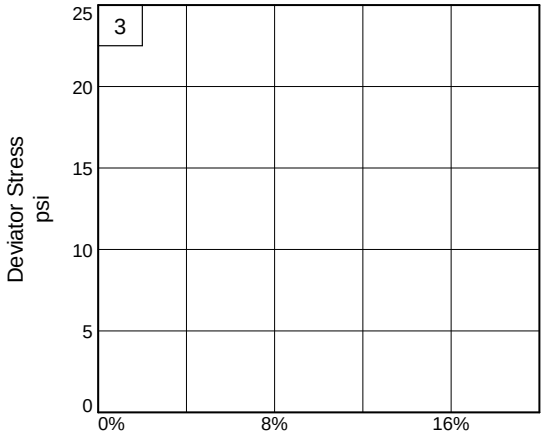
Sample Number: W-2 14 psi

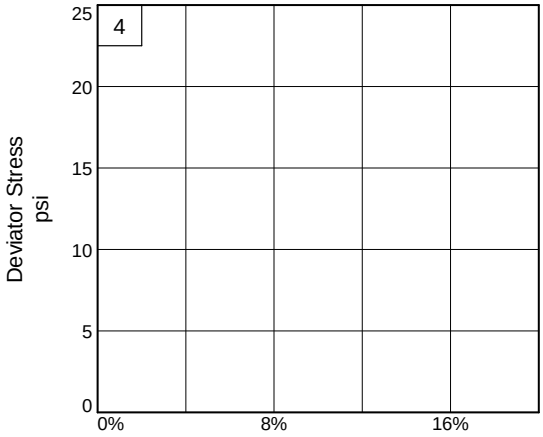
Proj. No.: 02221476 **Date Sampled:** N/A

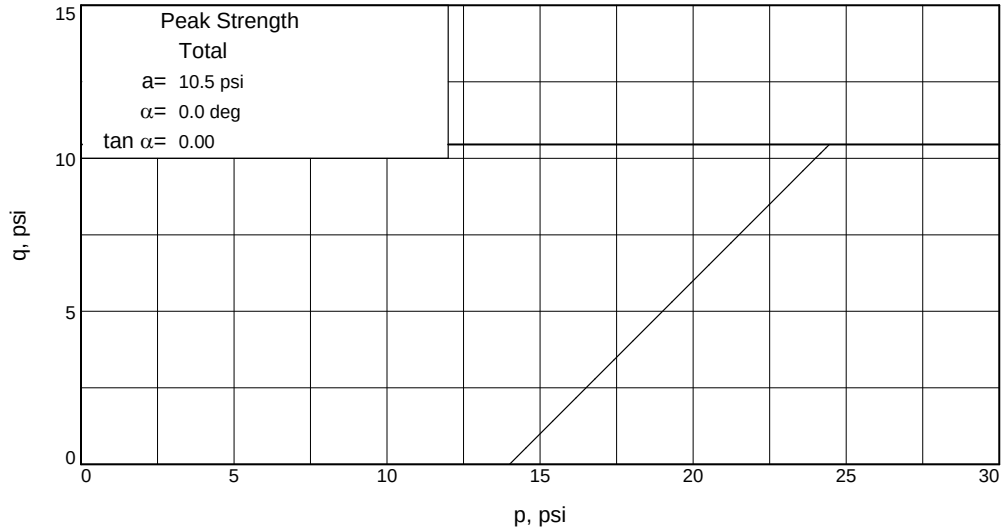
TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS





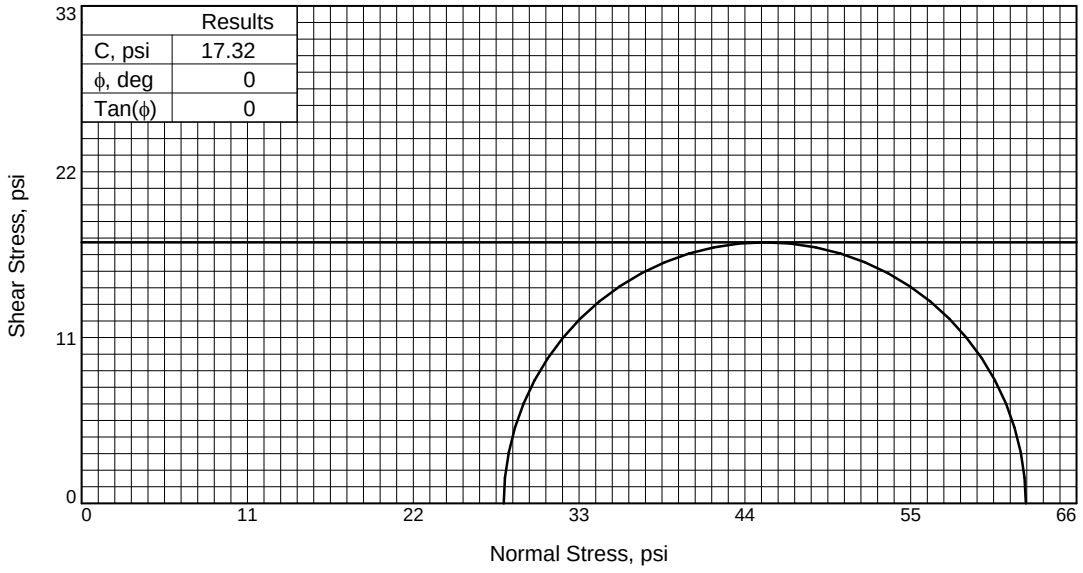


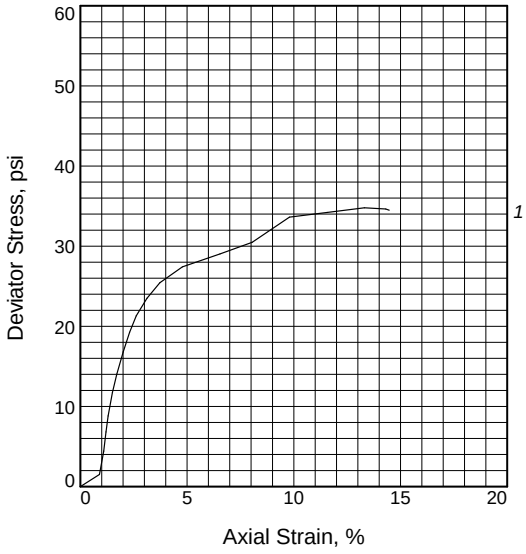




Client: USACE
Project: Upper Turkey Creek
Source of Sample: 197 **Depth:** 10-12 feet **Sample Number:** W-2 14 psi
Project No.: 02221476

Terracon Consultants, Inc.





Sample No.		1
Initial	Water Content, %	26.0
	Dry Density, pcf	97.9
	Saturation, %	97.2
	Void Ratio	0.7224
	Diameter, in.	1.36
	Height, in.	2.88
At Test	Water Content, %	26.8
	Dry Density, pcf	97.9
	Saturation, %	100.0
	Void Ratio	0.7224
	Diameter, in.	1.36
	Height, in.	2.88
Strain rate, in./min.		0.146
Back Pressure, psi		0.0
Cell Pressure, psi		28.0
Fail. Stress, psi		34.6
Ult. Stress, psi		
σ_1 Failure, psi		62.6
σ_3 Failure, psi		28.0

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter wax tube
Description: LEAN CLAY(CL)

LL= 47 **PL=** 19 **PI=** 28
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

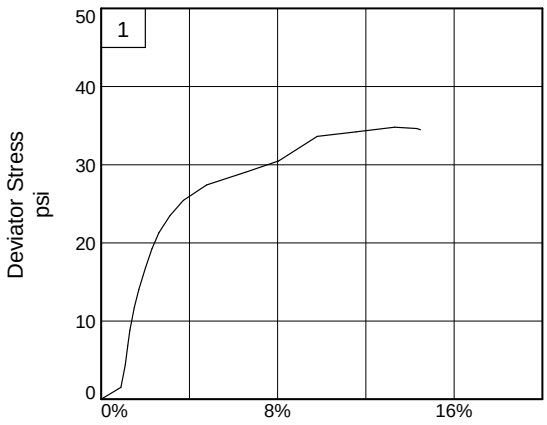
Client: USACE

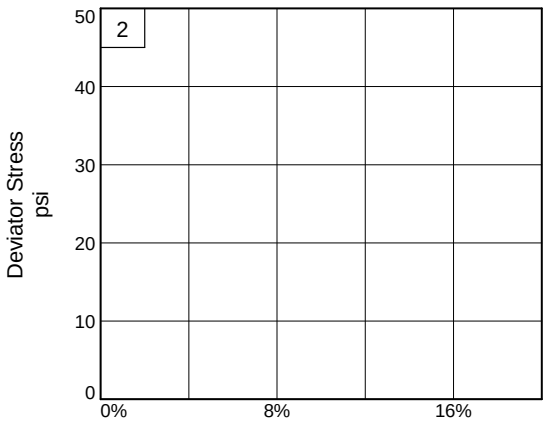
Project: Upper Turkey Creek

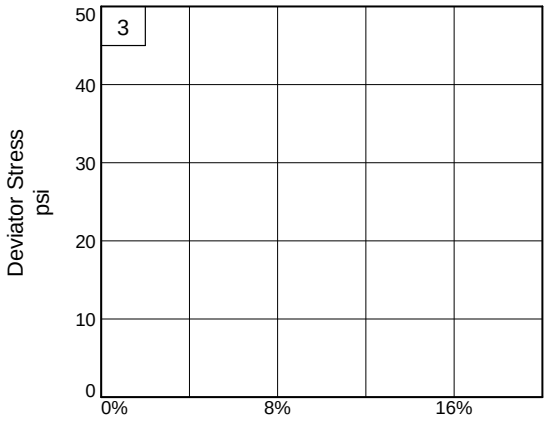
Source of Sample: 197 **Depth:** 10-12 feet
Sample Number: W-2 28 psi
Proj. No.: 02221476 **Date Sampled:** N/A

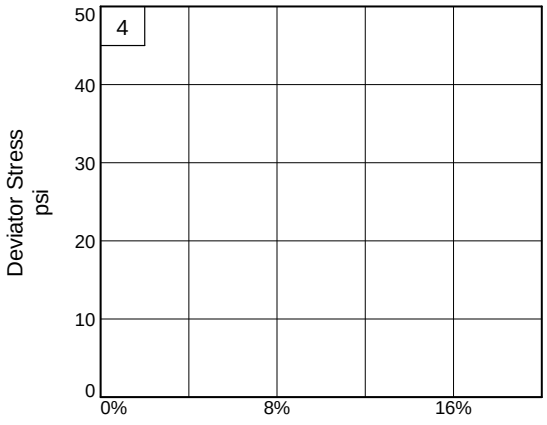
TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

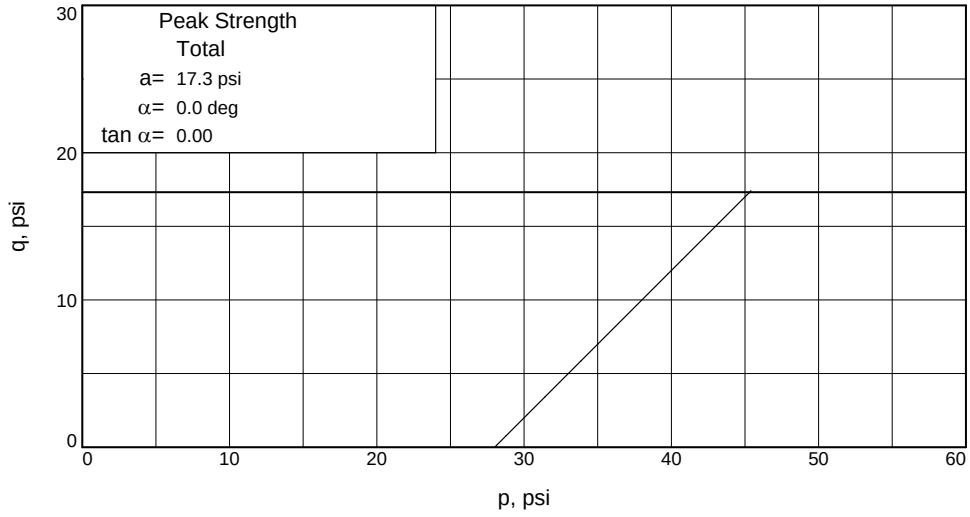
Tested By: SR







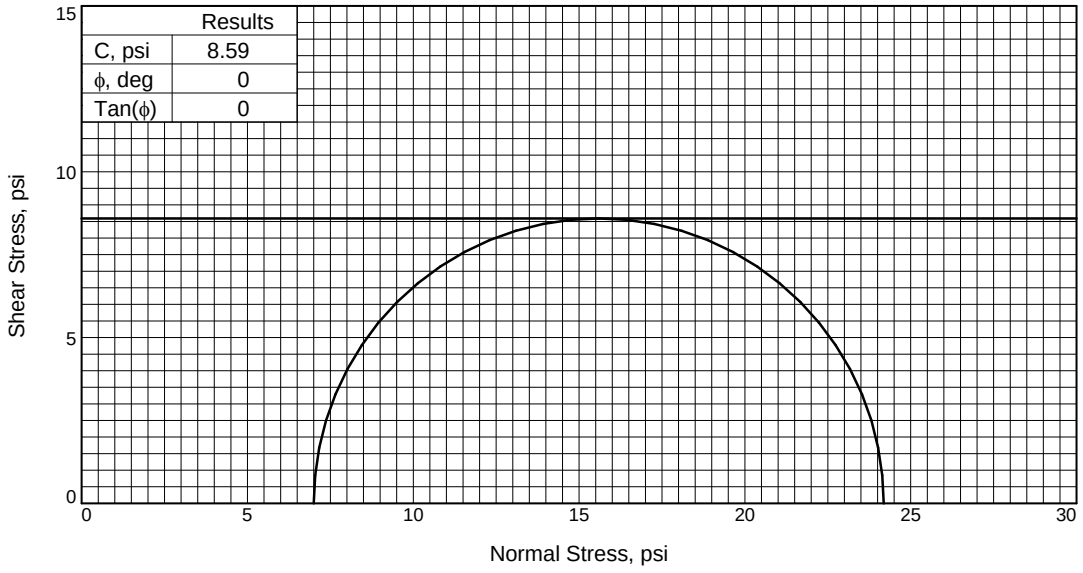


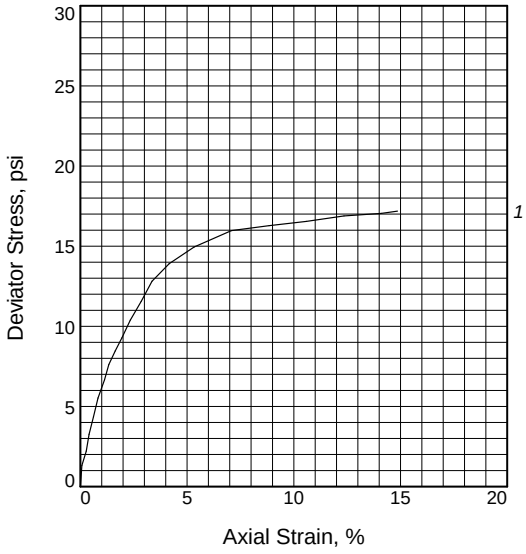


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 197 **Depth:** 10-12 feet **Sample Number:** W-2 28 psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SR





Sample No.		1
Initial	Water Content, %	26.0
	Dry Density, pcf	95.5
	Saturation, %	91.9
	Void Ratio	0.7644
	Diameter, in.	1.34
	Height, in.	3.19
At Test	Water Content, %	26.0
	Dry Density, pcf	95.5
	Saturation, %	91.9
	Void Ratio	0.7644
	Diameter, in.	1.34
	Height, in.	3.19
Strain rate, in./min.		0.132
Back Pressure, psi		0.0
Cell Pressure, psi		7.0
Fail. Stress, psi		17.2
Ult. Stress, psi		
σ_1 Failure, psi		24.2
σ_3 Failure, psi		7.0

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter wax tube
Description: LEAN CLAY(CL)

LL= 47 **PL=** 19 **PI=** 28
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

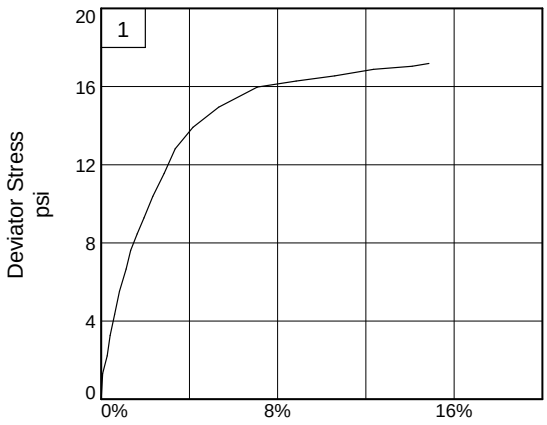
Client: USACE

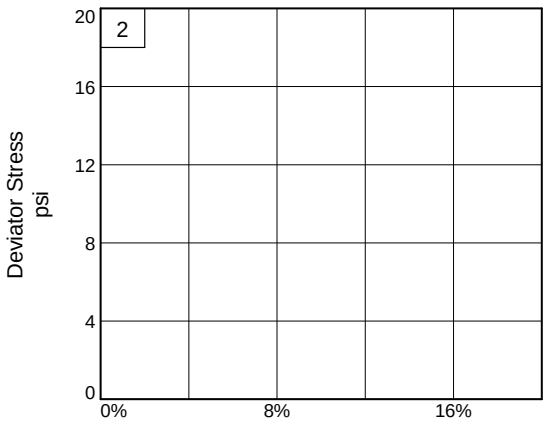
Project: Upper Turkey Creek

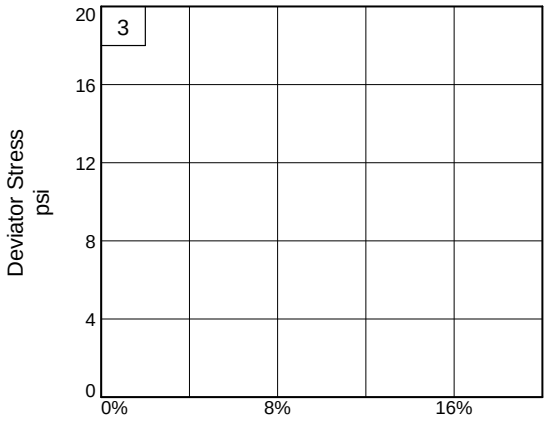
Source of Sample: 197 **Depth:** 10-12 feet
Sample Number: W-2 7 psi
Proj. No.: 02221476 **Date Sampled:** 3/9/23

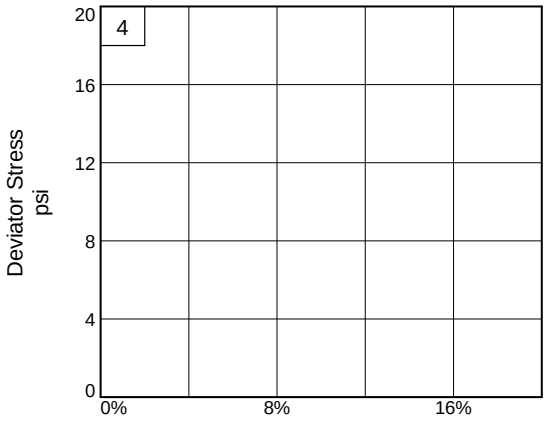
TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

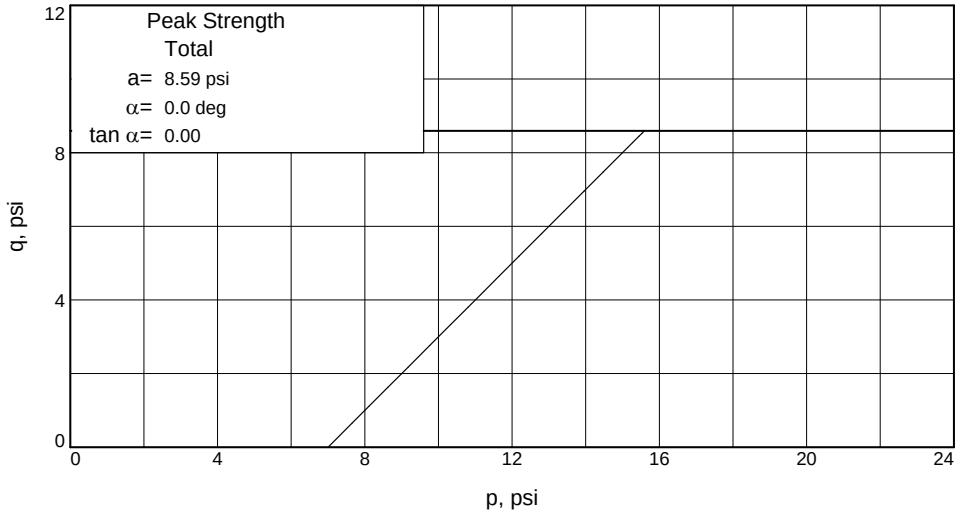
Tested By: SR







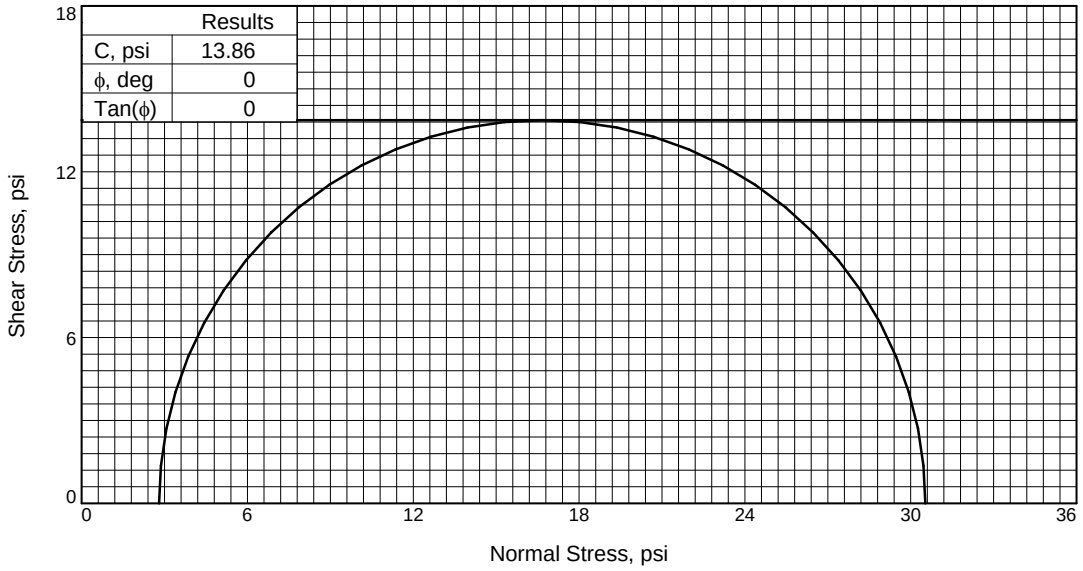


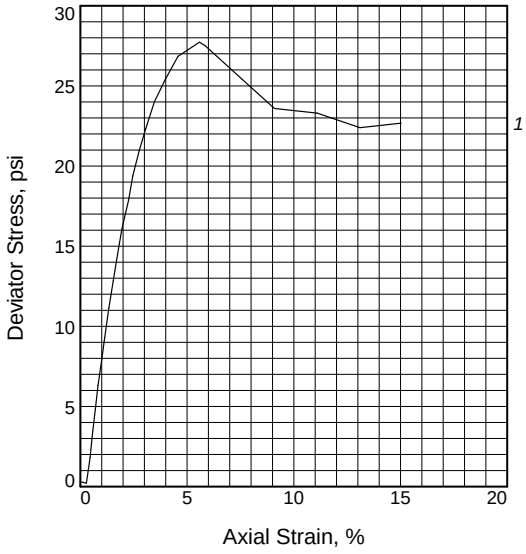


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 197 **Depth:** 10-12 feet **Sample Number:** W-2 7 psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SR





Sample No.		1
Initial	Water Content, %	21.9
	Dry Density, pcf	97.1
	Saturation, %	80.6
	Void Ratio	0.7351
	Diameter, in.	1.35
	Height, in.	2.81
At Test	Water Content, %	21.9
	Dry Density, pcf	97.1
	Saturation, %	80.6
	Void Ratio	0.7351
	Diameter, in.	1.35
	Height, in.	2.81
Strain rate, in./min.		0.160
Back Pressure, psi		0.0
Cell Pressure, psi		2.8
Fail. Stress, psi		27.7
Ult. Stress, psi		
σ_1 Failure, psi		30.5
σ_3 Failure, psi		2.8

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter wax tube
Description: CLAYEY GRAVEL with SAND(GC)

LL= 38 **PL=** 20 **PI=** 18
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

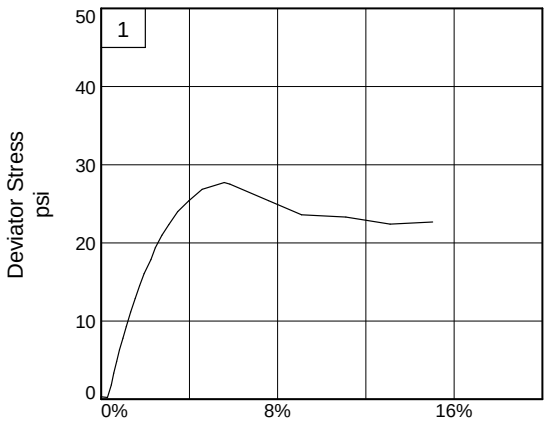
Client: USACE

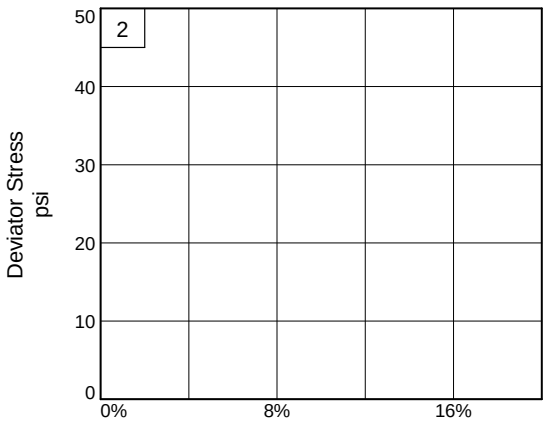
Project: Upper Turkey Creek

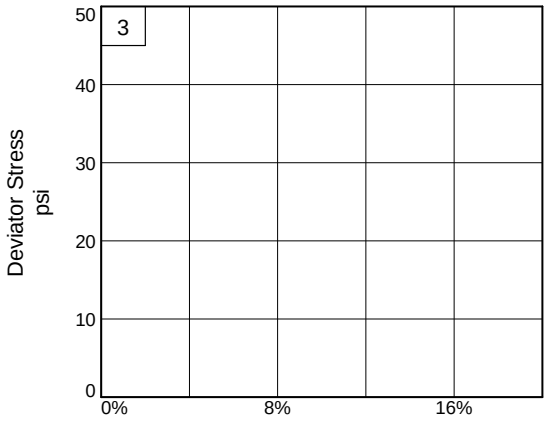
Source of Sample: 204 **Depth:** 2-4 feet
Sample Number: W-1 2.8 psi
Proj. No.: 02221476 **Date Sampled:** 3/1/23

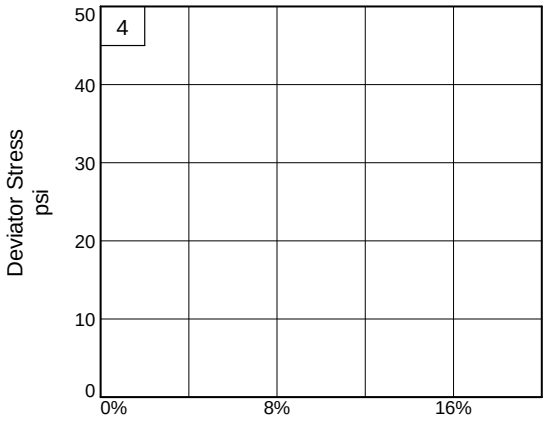
TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

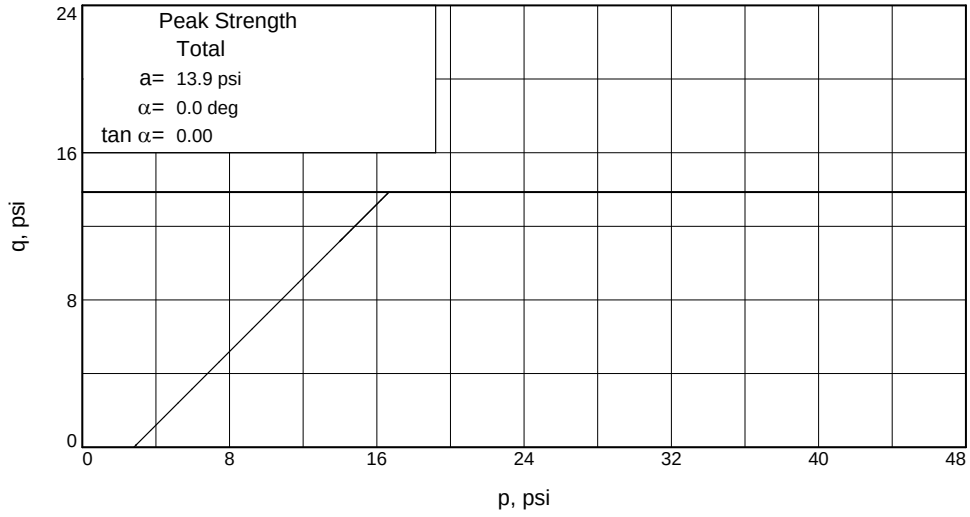
Tested By: SR







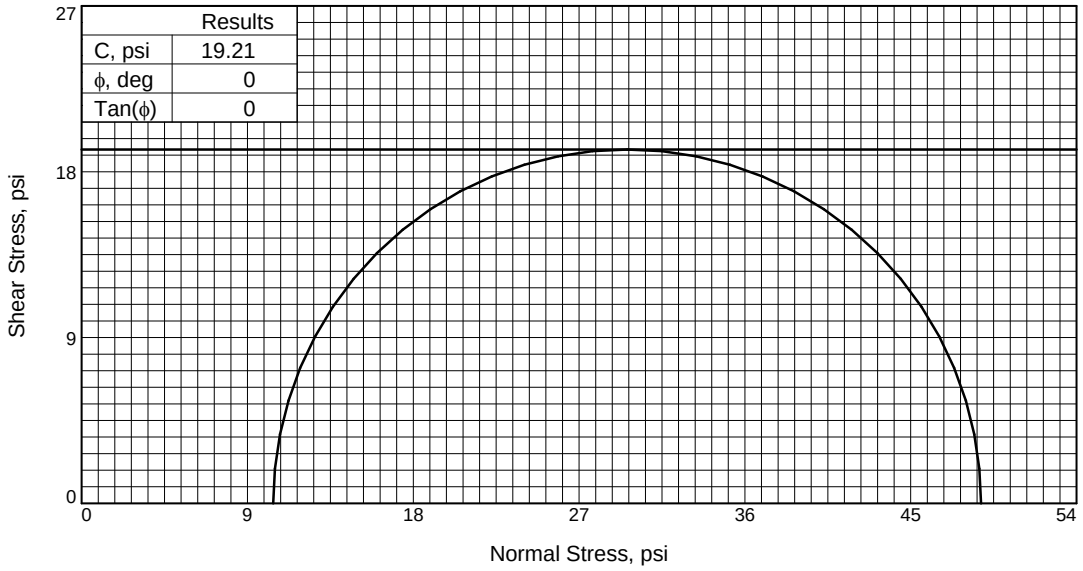


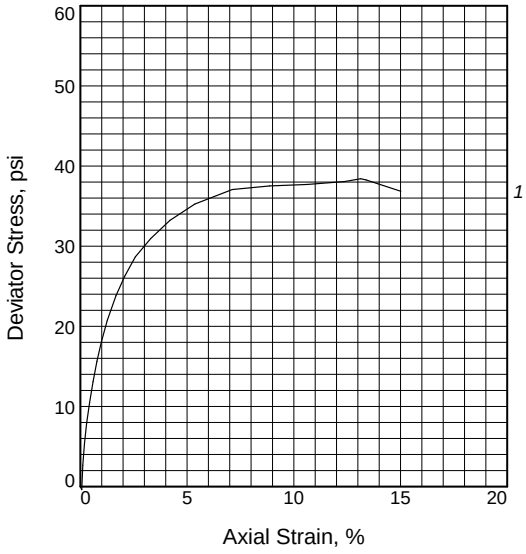


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 204 **Depth:** 2-4 feet **Sample Number:** W-1 2.8 psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SR





Sample No.		1
Initial	Water Content, %	22.3
	Dry Density, pcf	102.9
	Saturation, %	94.1
	Void Ratio	0.6386
	Diameter, in.	1.33
	Height, in.	3.20
At Test	Water Content, %	23.7
	Dry Density, pcf	102.9
	Saturation, %	100.0
	Void Ratio	0.6386
	Diameter, in.	1.33
	Height, in.	3.20
Strain rate, in./min.		0.129
Back Pressure, psi		0.0
Cell Pressure, psi		10.4
Fail. Stress, psi		38.4
Ult. Stress, psi		
σ_1 Failure, psi		48.8
σ_3 Failure, psi		10.4

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter Wax tube
Description: CLAYEY GRAVEL with SAND(GC)

LL= 38 **PL=** 20 **PI=** 18
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

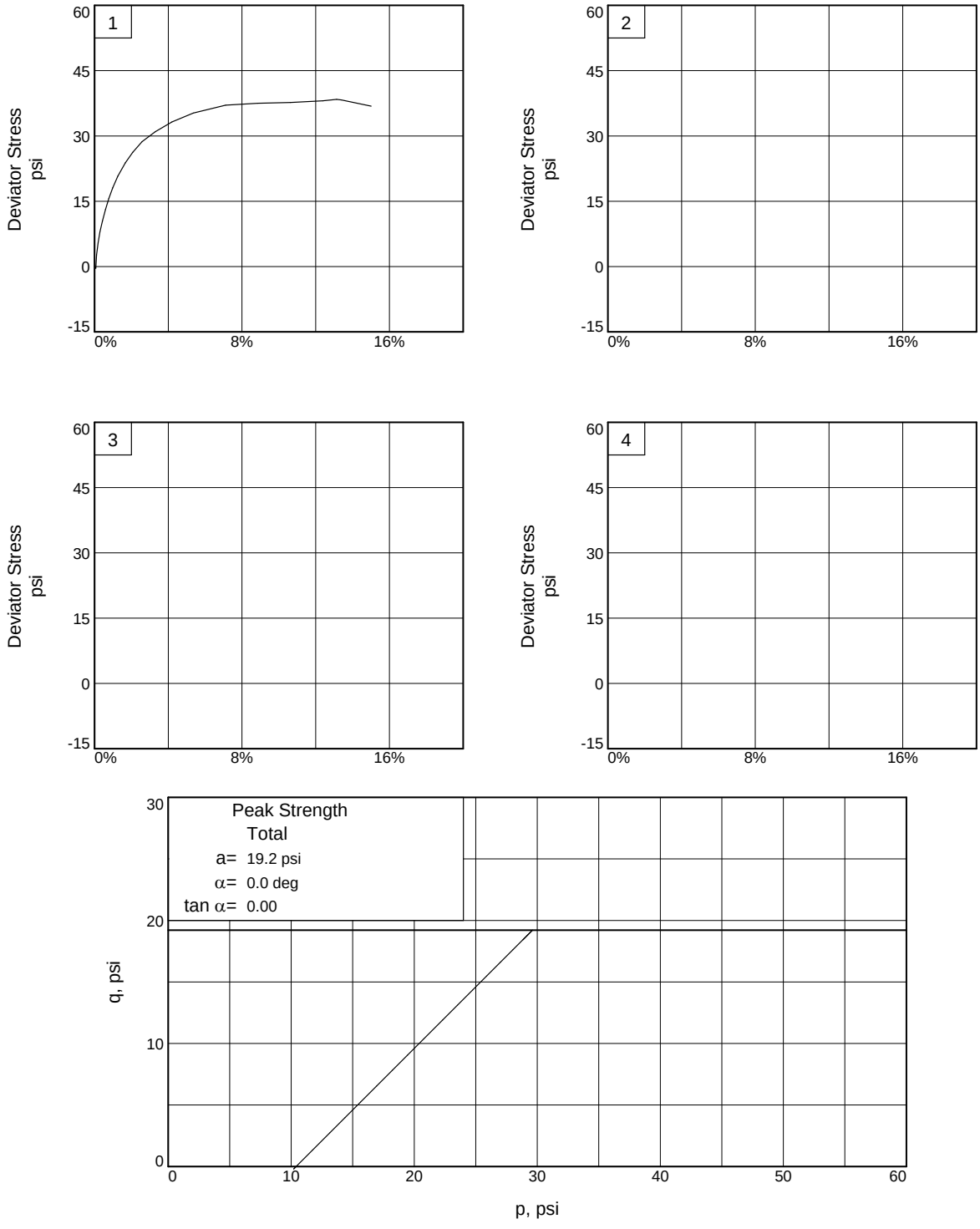
Client: USACE

Project: Upper Turkey Creek

Source of Sample: 204 **Depth:** 2-4 feet
Sample Number: W-1 10.4psi
Proj. No.: 02221476 **Date Sampled:** N/A

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

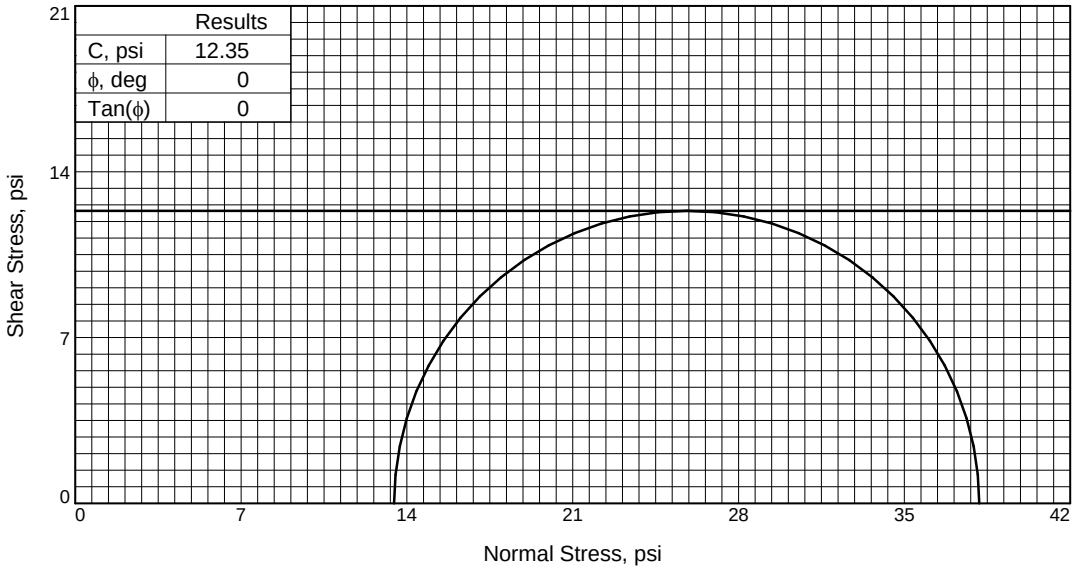
Tested By: SR

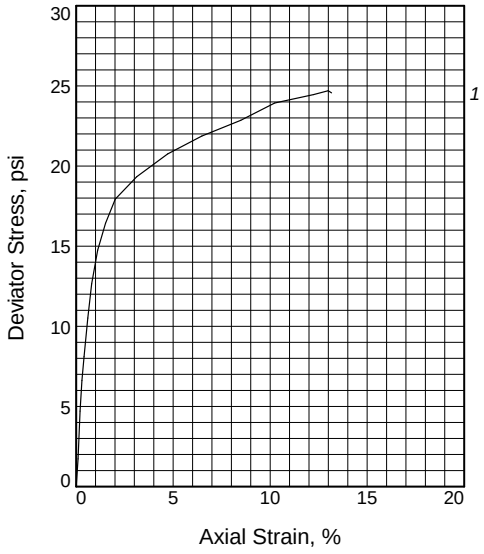


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 204 **Depth:** 2-4 feet **Sample Number:** W-1 10.4psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SR





Sample No.		1
Initial	Water Content, %	23.6
	Dry Density, pcf	96.4
	Saturation, %	85.3
	Void Ratio	0.7477
	Diameter, in.	1.36
	Height, in.	2.80
At Test	Water Content, %	23.6
	Dry Density, pcf	96.4
	Saturation, %	85.3
	Void Ratio	0.7477
	Diameter, in.	1.36
	Height, in.	2.80
Strain rate, in./min.		0.222
Back Pressure, psi		0.0
Cell Pressure, psi		13.5
Fail. Stress, psi		24.7
Ult. Stress, psi		
σ_1 Failure, psi		38.2
σ_3 Failure, psi		13.5

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter wax tube
Description: LEAN CLAY(CL)

LL= 44 **PL=** 21 **PI=** 23
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

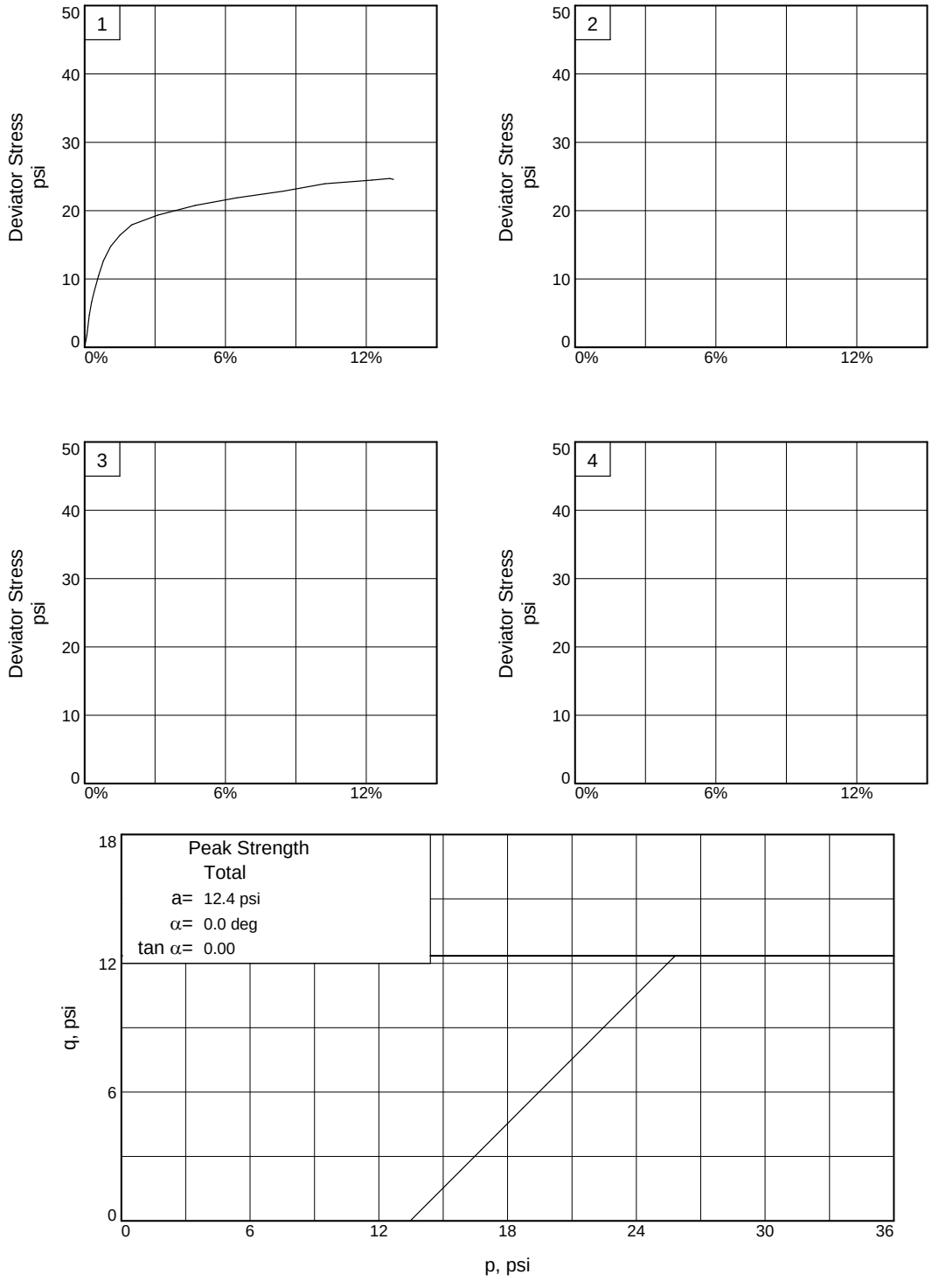
Client: USACE

Project: Upper Turkey Creek

Source of Sample: 207 **Depth:** 5-7 feet
Sample Number: W-2 20.8psi
Proj. No.: 02221476 **Date Sampled:** N/A

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

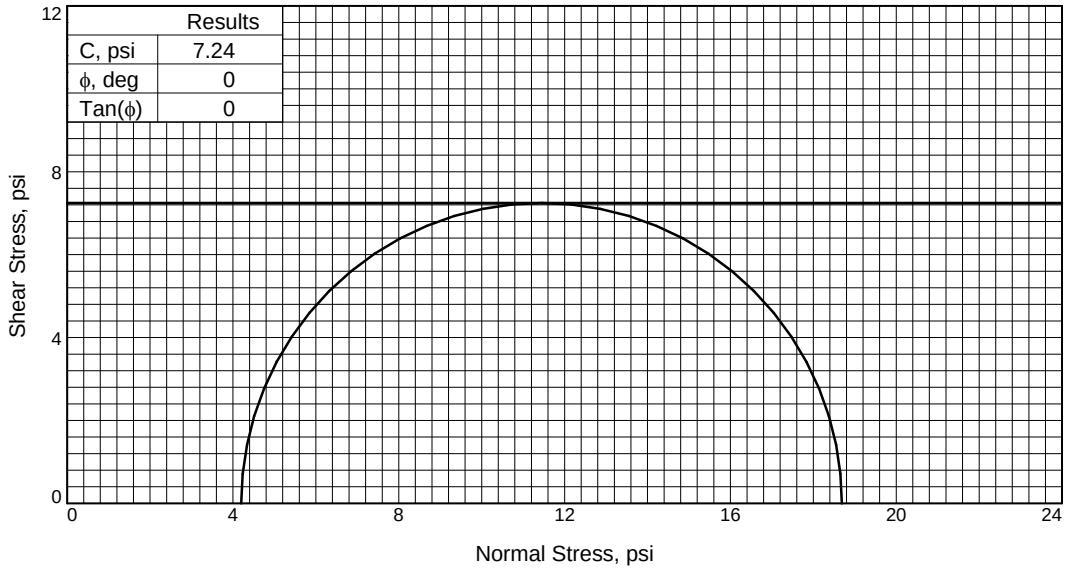
Tested By: SM

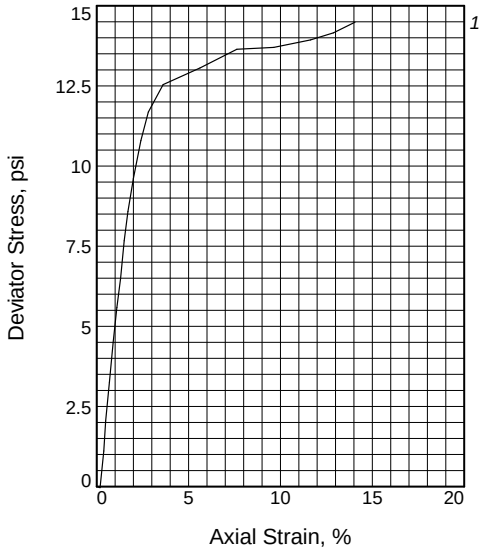


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 207 **Depth:** 5-7 feet **Sample Number:** W-2 20.8psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SM





Sample No.		1
Initial	Water Content, %	25.0
	Dry Density, pcf	92.3
	Saturation, %	81.6
	Void Ratio	0.8265
	Diameter, in.	1.37
	Height, in.	2.65
At Test	Water Content, %	25.0
	Dry Density, pcf	92.3
	Saturation, %	81.6
	Void Ratio	0.8265
	Diameter, in.	1.37
	Height, in.	2.65
Strain rate, in./min.		0.233
Back Pressure, psi		0.0
Cell Pressure, psi		4.2
Fail. Stress, psi		14.5
Ult. Stress, psi		
σ_1 Failure, psi		18.7
σ_3 Failure, psi		4.2

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter wax tube
Description: LEAN CLAY(CL)

LL= 44 **PL=** 21 **PI=** 23
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

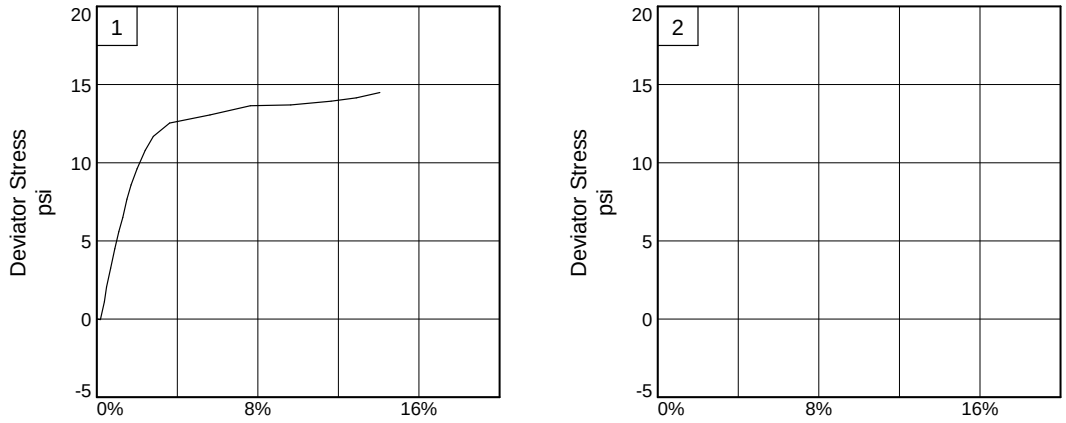
Client: USACE

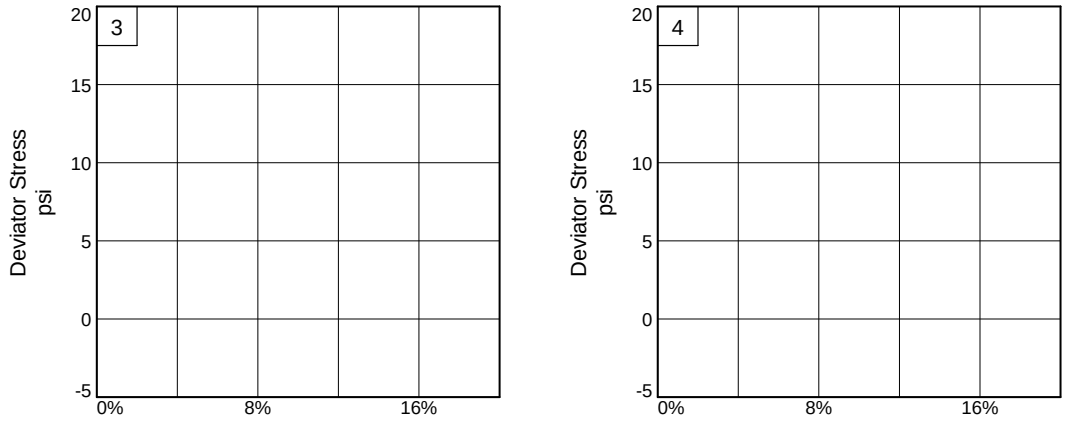
Project: Upper Turkey Creek

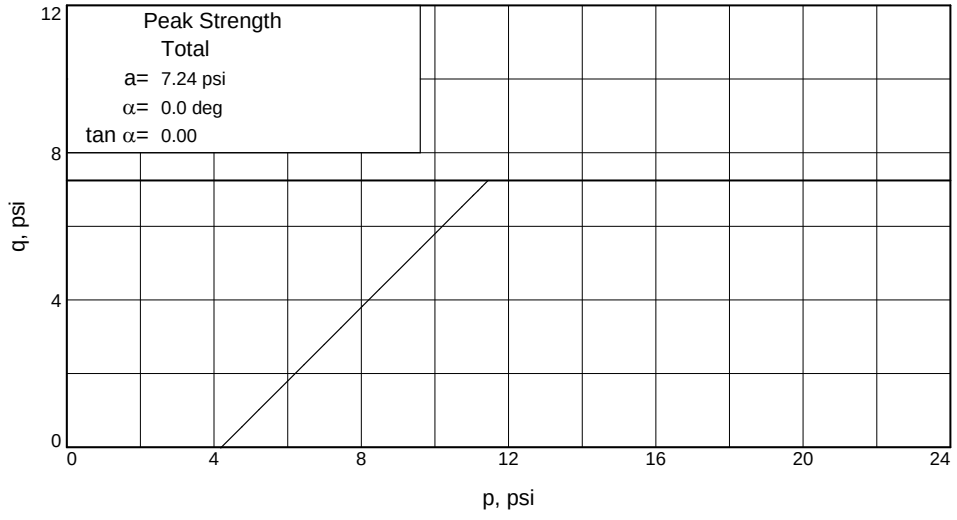
Source of Sample: 207 **Depth:** 5-7 feet
Sample Number: W-2 4.2psi
Proj. No.: 02221476 **Date Sampled:** 3/2/23

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

Tested By: SR



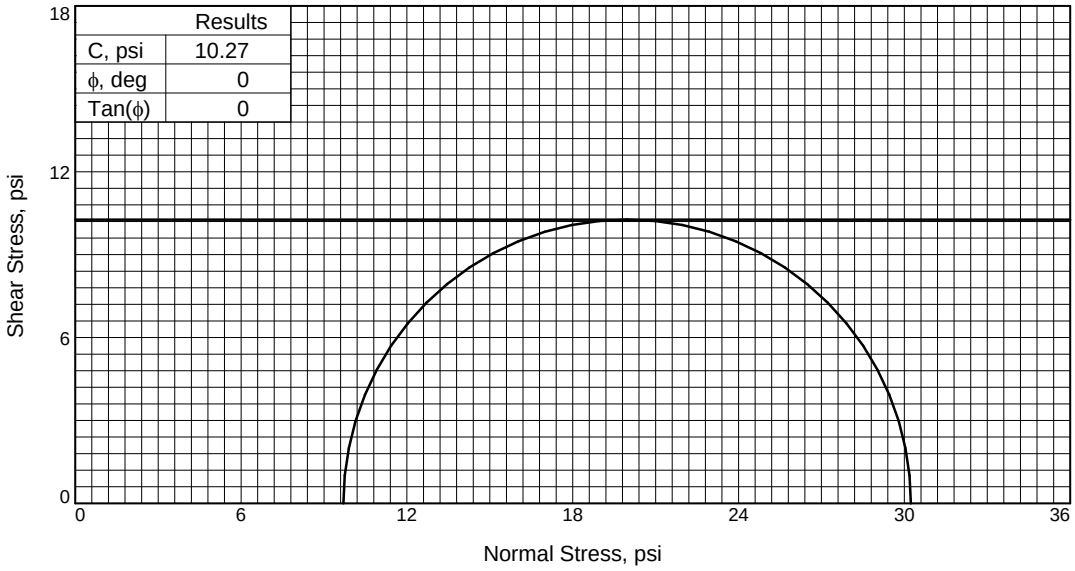


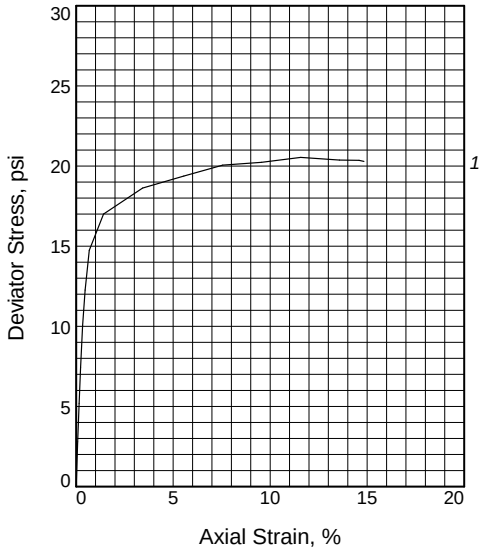


Client: USACE
Project: Upper Turkey Creek
Source of Sample: 207 **Depth:** 5-7 feet **Sample Number:** W-2 4.2psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SR





Sample No.		1
Initial	Water Content, %	23.6
	Dry Density, pcf	98.6
	Saturation, %	89.9
	Void Ratio	0.7088
	Diameter, in.	1.33
	Height, in.	2.54
At Test	Water Content, %	23.6
	Dry Density, pcf	98.6
	Saturation, %	89.9
	Void Ratio	0.7088
	Diameter, in.	1.33
	Height, in.	2.54
Strain rate, in./min.		0.102
Back Pressure, psi		0.0
Cell Pressure, psi		9.7
Fail. Stress, psi		20.5
Ult. Stress, psi		
σ_1 Failure, psi		30.2
σ_3 Failure, psi		9.7

Type of Test:
Unconsolidated Undrained
Sample Type: 5" diameter wax tube
Description: LEAN CLAY(CL)

LL= 44 **PL=** 21 **PI=** 23
Assumed Specific Gravity= 2.7
Remarks: Trimmed down to a 1.4" diameter specimen.

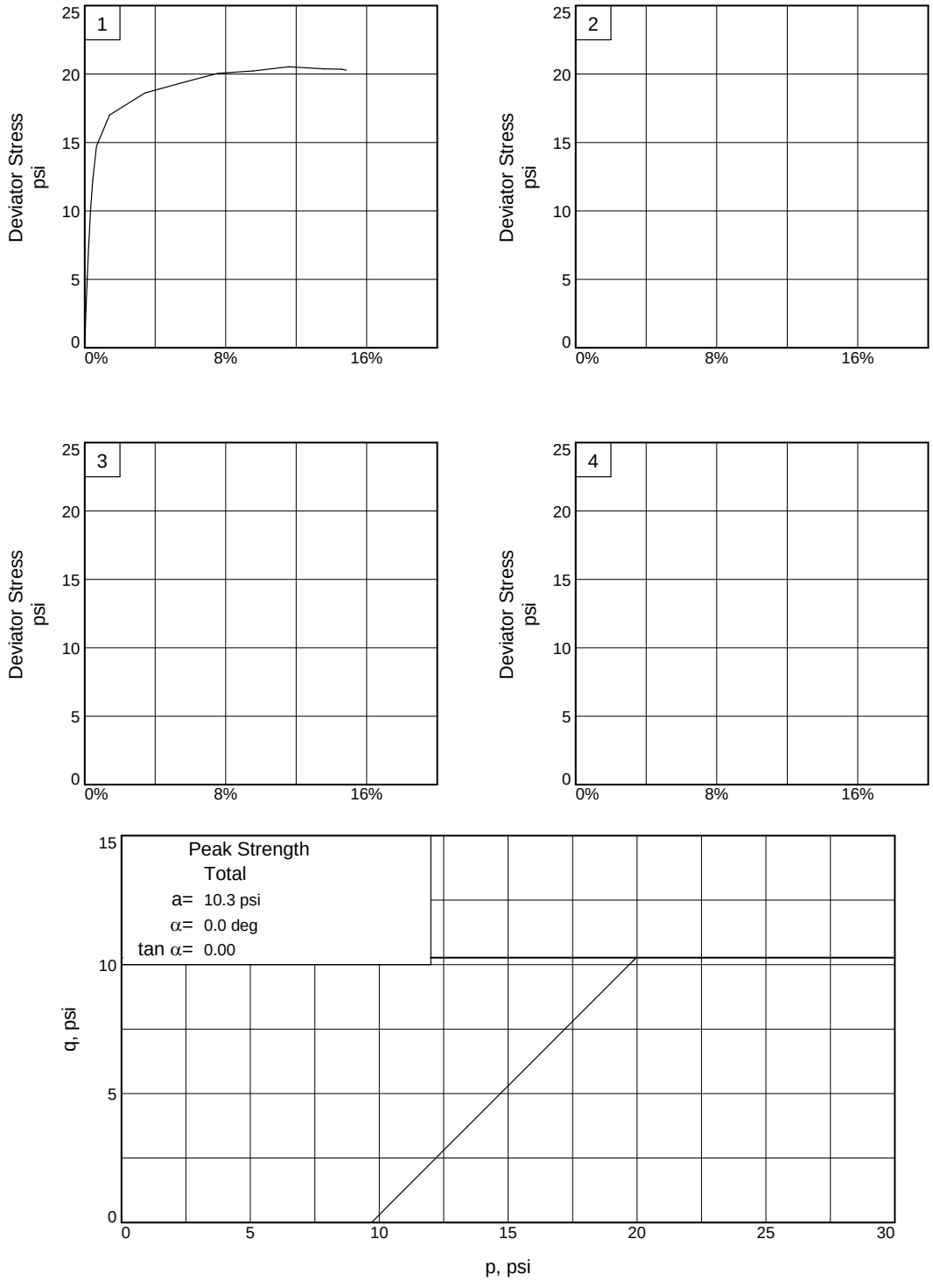
Client: USACE

Project: Upper Turkey Creek

Source of Sample: 207 **Depth:** 5-7 feet
Sample Number: W-2 9.7psi
Proj. No.: 02221476 **Date Sampled:** N/A

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

Tested By: SM



Client: USACE
Project: Upper Turkey Creek
Source of Sample: 207 **Depth:** 5-7 feet **Sample Number:** W-2 9.7psi
Project No.: 02221476

Terracon Consultants, Inc.

Tested By: SM

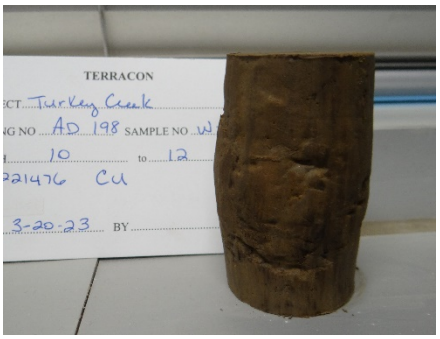
USACE UPPER TURKEY CREEK LABORATORY TESTING
MERRIAM, KANSAS
02221476
3/2/2023

UNCONFINED COMPRESSIVE STRENGTH OF
INTACT ROCK CORE SPECIMENS
ASTM D7012 METHOD C

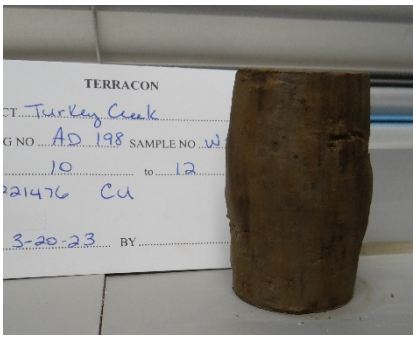
BORING NO.	RUN NO.	DEPTH, feet	TYPE OF SPECIMEN	DIAMETER, inch	CUT LENGTH, inch	CAPPED LENGTH, inch	MAXIMUM LOAD, pounds	L/D RATIO	CORRECTED COMPRESSIVE STRENGTH*, psi
202	WAX 1	12.2 - 12.6	SHALE	1.91	3.81	3.99	177	2.09	60
205	WAX-1	15.1 - 15.5	LIMESTONE	1.87	3.68	3.88	34500	2.07	12560
205	WAX-2	18.4 - 18.8	LIMESTONE	1.87	3.74	3.89	23450	2.08	8540
ADUC-196	WAX-1	28.3 - 28.7	SHALE	1.91	3.91	4.05	122	2.12	40

*NOTE: CORRECTED TO L/D RATIO OF 2.00 IF L/D RATIO IS LESS THAN 2.00.
NOTE: CAPPING COMPOUND USED TO CAP BOTH BEARING SURFACES.

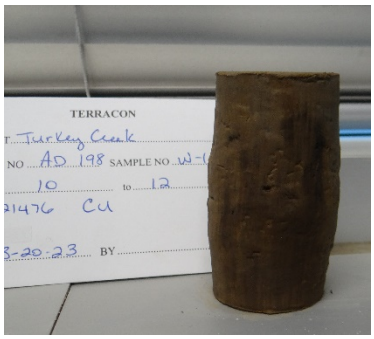
BORING NO.	RUN NO.	DEPTH, feet	WATER CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
202	WAX 1	12.2 - 12.6	15.4	139	120
205	WAX-1	15.1 - 15.5		162	
205	WAX-2	18.4 - 18.8		162	
ADUC-196	WAX-1	28.3 - 28.7	12.8	140	124



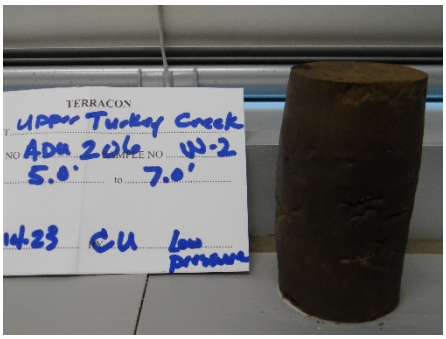
ADU-198 W-1 10-12 feet 7 psi - CU



ADU-198 W-1 10-12 feet 14 psi - CU



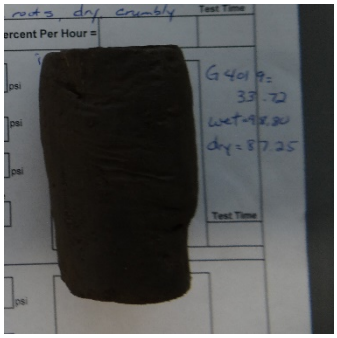
ADU-198 W-1 10-12 feet 28 psi - CU



ADU-206 W-2 5-7 feet 2.8 psi CU

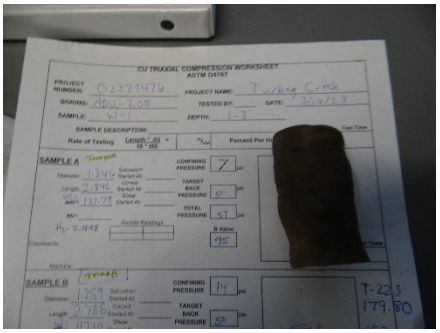


ADU-206 W-2 5-7 feet 7 psi - CU



ADU-206 W-2 5-7 feet 10.4psi - CU

USACE Upper Turkey Creek Laboratory Testing	Site Plan:	Terracon
	Report Number: 02221476.0001	
Task Order No. W912DQ23F1003	Technician: Suzy Robinett	15620 W 113th St
Contract No. W912DQ19D1017	Date: 04/26/23	Lenexa, KS 66219-5102
Lenexa, KS	Scale: Not to Scale	**913-492-7777**



ADU-208 W-1 1-3 feet 7 psi - CU



ADU-208 W-1 1-3 feet 14 psi - CU

USACE Upper Turkey Creek Laboratory Testing	Site Plan:	Terracon
	Report Number: 02221476.0001	
Task Order No. W912DQ23F1003	Technician: Suzy Robinett	15620 W 113th St
Contract No. W912DQ19D1017	Date: 04/26/23	Lenexa, KS 66219-5102
Lenexa, KS	Scale: Not to Scale	**913-492-7777**

LETTER OF TRANSMITTAL

15620 West 113th Street
Lenexa, Kansas 66219
Phone: (913) 492-7777
FAX: (913) 492-7443

Attention: **Derek Jensen** Date: **November 2, 2023**

Company: **U.S. Army Corps of Engineers**
Kansas City District Office, EDG-D, Room 526
601 East 12th St.
Kansas City, Missouri 64106-2896

Project Name: **Upper Turkey Creek Soil Testing 2**

Project Location: **Merriam, Kansas**

Terracon Project No: **02231300** Client Project No: **W912DQ19D1017**

We are transmitting
- ☒ Herewith
- ☐ Under separate cover

☐ Field Data ☒ Laboratory Data ☐ Report

Regarding:

Geotechnical laboratory testing for the Upper Turkey Creek Project including USCS Classification, Water Content, Particle Size Distribution, and Consolidated Undrained shear strengths.

Comments:

The following issues were found with the samples:

- Samples from ADU-206 and ADU-207 were not able to be tested for Consolidated Undrained shear strengths or accurate water content due to the samples being dried out and breaking when trimmed. The USCS Classification was not performed on these samples.

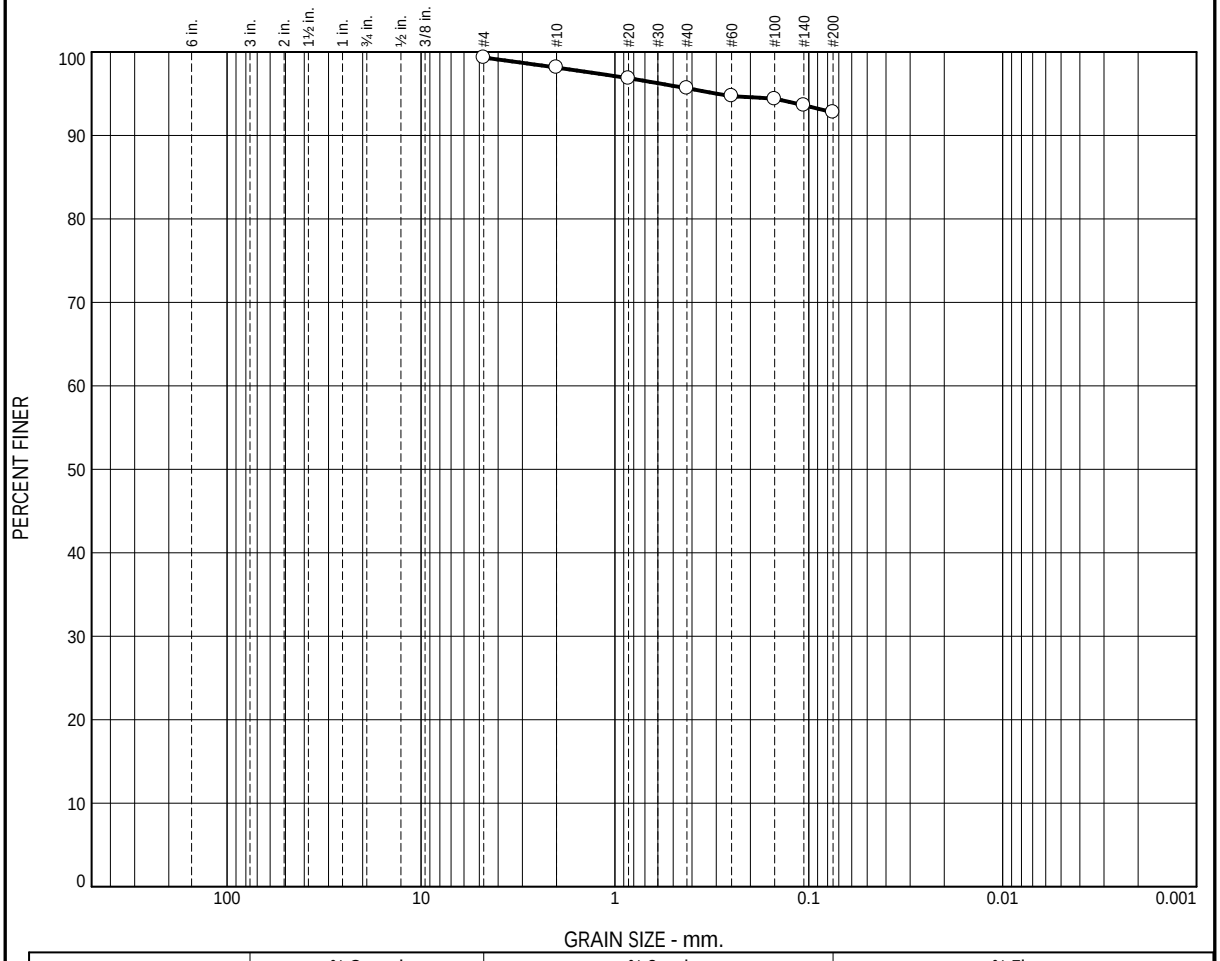
We have not been asked to interpret the data or make design and/or construction recommendations based on the data and cannot assume responsibility or liability for interpretation of this data by others.

Mohamad Ibrawish, PE

Submit more samples for testing after received results from the first round, most too dry/disturbed for testing

Particle Size Distribution Report

ASTM D6913



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○				1.2	2.4	2.9	92.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	48	21								

Material Description	Test Date	USCS	NM
○ Lean CLAY (CL) brown and dark brown with trace rock fragments.	9-18-23	CL	

Project No. 02231300 Client: USACE

Project: Upper Turkey Creek

○Source of Sample: ADU-210 Depth: 1.0-3.0 feet Sample Number: W-1

Terracon Consultants, Inc.

Lenexa, KS

Remarks:

○None

GRAIN SIZE DISTRIBUTION TEST DATA 10/4/2023

Client: USACE
Project: Upper Turkey Creek
Project Number: 02231300
Location: ADU-210
Depth: 1.0-3.0 feet Sample Number: W-1
Material Description: Lean CLAY (CL) brown and dark brown with trace rock fragments.
Liquid Limit: 48 Plastic Limit: 21
Test Date: 9-18-23

Sieve Test Data (ASTM D6913)

Test Date: 9-18-23
Test remarks: None

Post #200 Wash Test Weights (grams): Dry Specimen+Tare = 17.66
Tare Wt. = 0.00

Minus #200 from wash = 92.7%

Specimen Weights
Dry specimen+tare (gms.) = 241.89
Tare (gms.) = 0.00

Cumulative pan tare (gms.) = 0.00

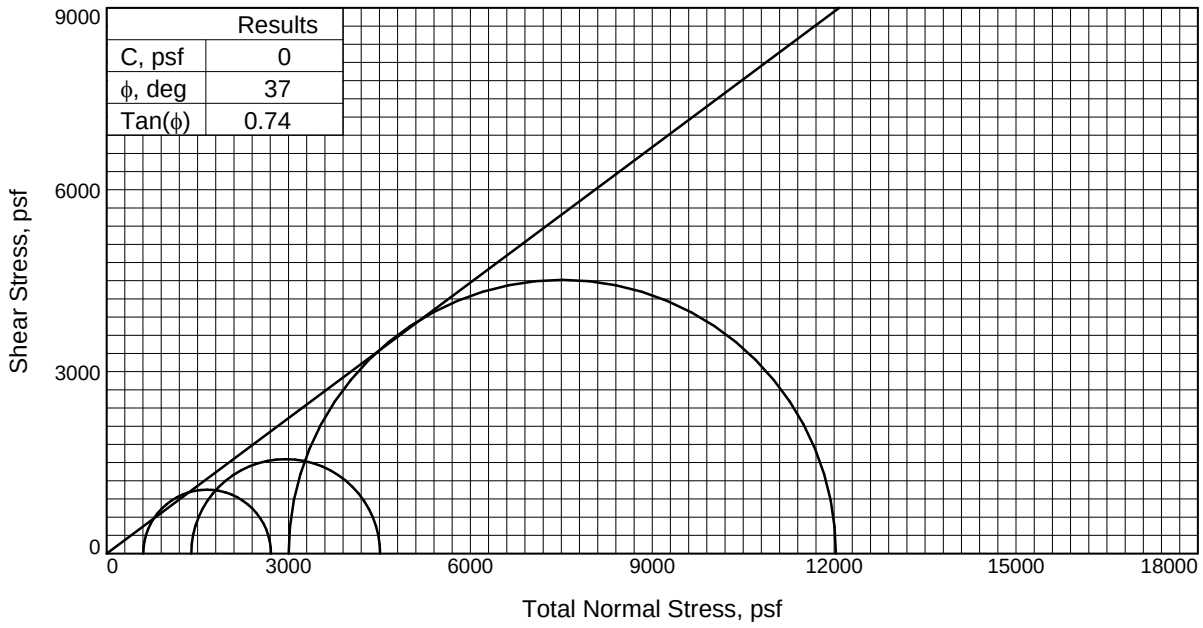
Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Passing
#4	1.63	99.3
#10	4.53	98.1
#20	7.63	96.8
#40	10.52	95.7
#60	12.73	94.7
#100	13.65	94.4
#140	15.46	93.6
#200	17.51	92.8

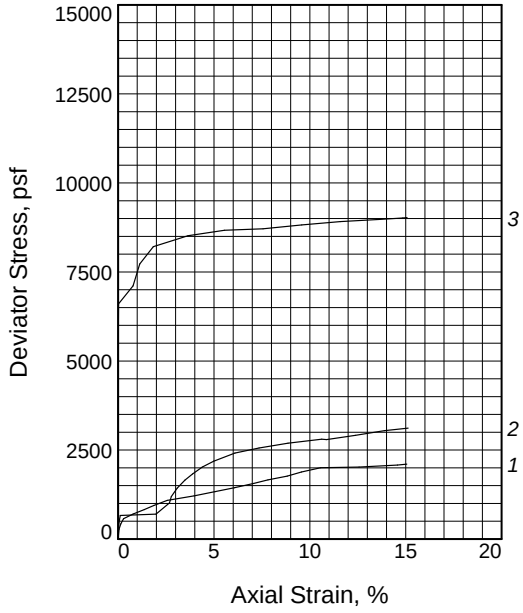
Results

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
				1.2	2.4	2.9	6.5			92.8

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
											0.2978

Fineness Modulus
0.19





Sample No.		1	2	3
Initial	Water Content, %	22.1	20.1	22.1
	Dry Density, pcf	99.8	103.4	100.7
	Saturation, %	86.4	86.3	88.3
	Void Ratio	0.6890	0.6294	0.6744
	Diameter, in.	1.36	1.33	1.35
	Height, in.	2.79	2.79	2.81
At Test	Water Content, %	25.5	22.4	24.1
	Dry Density, pcf	99.8	105.0	102.1
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.6890	0.6050	0.6512
	Diameter, in.	1.36	1.32	1.35
	Height, in.	2.79	2.78	2.79
Strain rate, in./min.		1.000	1.000	1.000
Back Pressure, psi		50.00	50.00	50.00
Cell Pressure, psi		54.20	59.70	70.85
Fail. Stress, psf		2106	3114	9026
Total Pore Pr., psf		7098	8483	9062
Ult. Stress, psf		2106	3114	9026
Total Pore Pr., psf		7098	8483	9062
$\bar{\sigma}_1$ Failure, psf		2812	3228	10167
$\bar{\sigma}_3$ Failure, psf		707	113	1141

Type of Test:
CU with Pore Pressures

Sample Type: 5" diameter wax tube

Description: Lean CLAY (CL) brown and dark brown

LL= 48 **PL=** 21 **PI=** 27

Assumed Specific Gravity= 2.7

Remarks: Trimmed down to a 1.4" diameter specimen.

Client: USACE

Project: Upper Turkey Creek

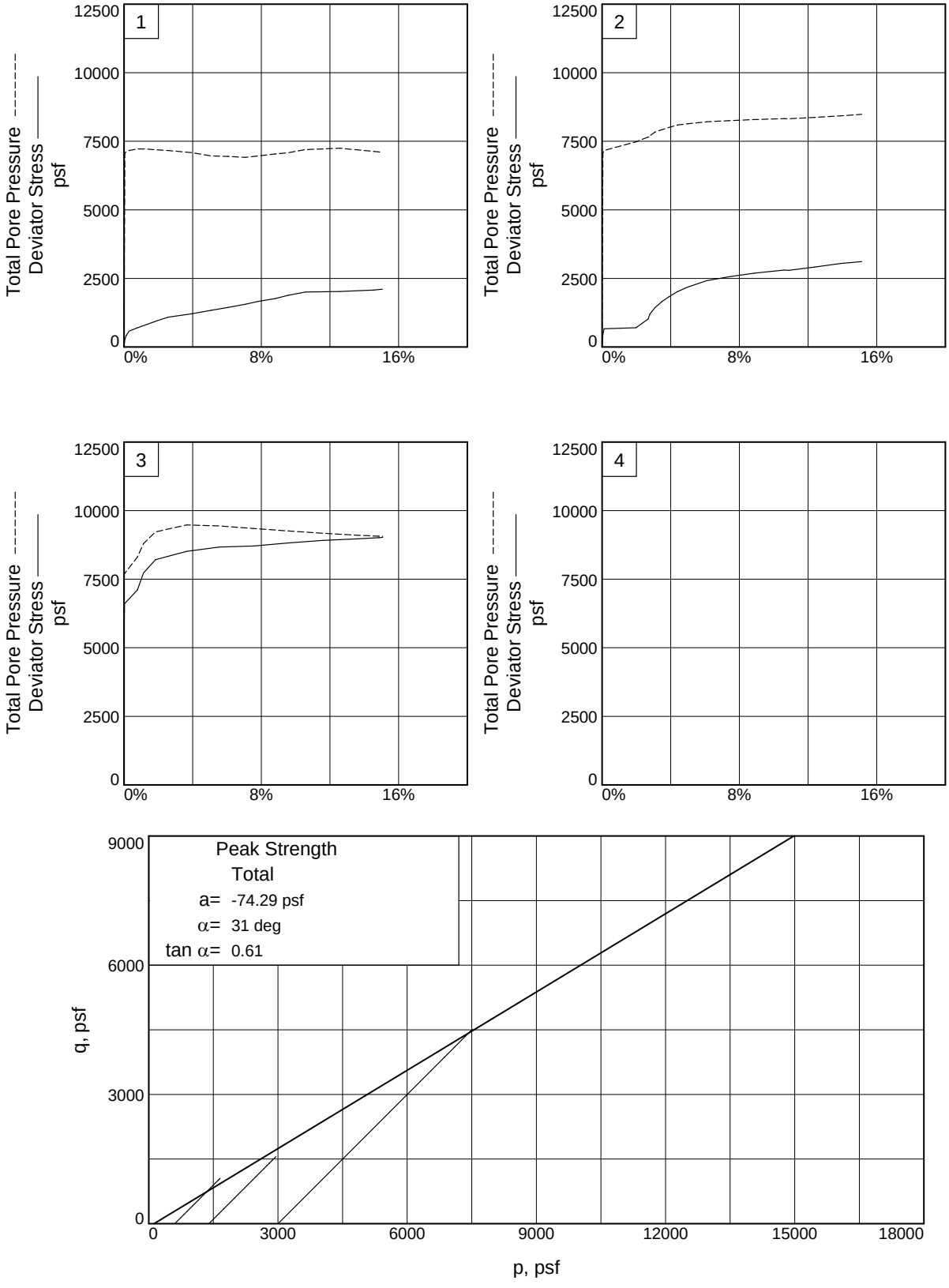
Source of Sample: ADU-210 **Depth:** 1.0-3.0 feet

Sample Number: W-1

Proj. No.: 02231300 **Date Sampled:** 9-13-23

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Lenexa, KS

Tested By: SR



Client: USACE
Project: Upper Turkey Creek
Source of Sample: ADU-210 **Depth:** 1.0-3.0 feet **Sample Number:** W-1
Project No.: 02231300

Terracon Consultants, Inc.

Tested By: SR

TRIAXIAL COMPRESSION TEST 10/25/2023

CU with Pore Pressures 11:20 AM

Date: 9-13-23
Client: USACE
Project: Upper Turkey Creek
Project No.: 02231300
Location: ADU-210
Depth: 1.0-3.0 feet **Sample Number:** W-1
Description: Lean CLAY (CL) brown and dark brown
Remarks: Trimmed down to a 1.4" diameter specimen.
Type of Sample: 5" diameter wax tube
Assumed Specific Gravity=2.7 **LL=**48 **PL=**21 **PI=**27
Test Method: COE uniform strain

Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	107.440			264.880
Moisture content: Dry soil+tare, gms.	94.230			238.710
Moisture content: Tare, gms.	34.340			134.530
Moisture, %	22.1	25.5	25.5	25.1
Moist specimen weight, gms.	129.2			
Diameter, in.	1.36	1.36	1.36	
Area, in.²	1.45	1.45	1.45	
Height, in.	2.79	2.79	2.79	
Net decrease in height, in.		0.00	0.00	
Wet density, pcf	121.8	125.3	125.3	
Dry density, pcf	99.8	99.8	99.8	
Void ratio	0.6890	0.6890	0.6890	
Saturation, %	86.4	100.0	100.0	

Test Readings for Specimen No. 1

Membrane modulus = 0.124105 kN/cm²
Membrane thickness = 0.02 cm
Consolidation cell pressure = 54.20 psi (7805 psf)
Consolidation back pressure = 50.00 psi (7200 psf)
Consolidation effective confining stress = 605 psf
Strain rate, in./min. = 1.000
Fail. Stress = 2106 psf **at reading no.** 18
Ult. Stress = 2106 psf **at reading no.** 18

Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf	Minor Eff. Stress psf	Major Eff. Stress psf	1:3 Ratio	Pore Press. psi	P psf	Q psf
0	0.0000	0.000	0.0	0.0	0	7805	7805	1.00	0.00	7805	0
1	0.0013	2.729	2.7	0.0	271	737	1008	1.37	49.08	872	136
2	0.0039	4.288	4.3	0.1	426	684	1110	1.62	49.45	897	213
3	0.0081	5.787	5.8	0.3	574	642	1215	1.89	49.74	929	287
4	0.0220	7.130	7.1	0.8	703	583	1286	2.21	50.15	935	352
5	0.0381	8.428	8.4	1.4	826	591	1417	2.40	50.10	1004	413
6	0.0520	9.687	9.7	1.9	945	615	1560	2.54	49.93	1088	473
7	0.0716	11.193	11.2	2.6	1084	640	1724	2.69	49.76	1182	542
8	0.1106	12.708	12.7	4.0	1213	719	1932	2.69	49.21	1326	607
9	0.1413	14.131	14.1	5.1	1334	839	2173	2.59	48.37	1506	667
10	0.1692	15.431	15.4	6.1	1441	857	2298	2.68	48.25	1577	721
11	0.1971	16.879	16.9	7.1	1560	892	2451	2.75	48.01	1671	780
12	0.2181	18.134	18.1	7.8	1662	838	2499	2.98	48.38	1669	831
13	0.2460	19.474	19.5	8.8	1765	767	2532	3.30	48.88	1649	883
14	0.2669	20.949	20.9	9.6	1884	723	2606	3.61	49.18	1664	942
15	0.2948	22.538	22.5	10.6	2004	608	2612	4.30	49.98	1610	1002
16	0.3506	23.267	23.3	12.6	2022	558	2580	4.63	50.33	1569	1011
17	0.4065	24.446	24.4	14.6	2076	675	2751	4.08	49.51	1713	1038
18	0.4200	24.931	24.9	15.1	2106	707	2812	3.98	49.29	1760	1053

Parameters for Specimen No. 2

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	107.440			260.390
Moisture content: Dry soil+tare, gms.	95.200			235.620
Moisture content: Tare, gms.	34.340			132.220
Moisture, %	20.1	23.3	22.4	24.0
Moist specimen weight, gms.	126.6			
Diameter, in.	1.33	1.33	1.32	
Area, in.²	1.39	1.39	1.38	
Height, in.	2.79	2.79	2.78	
Net decrease in height, in.		0.00	0.01	
Wet density, pcf	124.2	127.6	128.6	
Dry density, pcf	103.4	103.4	105.0	
Void ratio	0.6294	0.6294	0.6050	
Saturation, %	86.3	100.0	100.0	

Test Readings for Specimen No. 2

Membrane modulus = 0.124105 kN/cm²
Membrane thickness = 0.02 cm
Consolidation cell pressure = 59.70 psi (8597 psf)
Consolidation back pressure = 50.00 psi (7200 psf)
Consolidation effective confining stress = 1397 psf
Strain rate, in./min. = 1.000
Fail. Stress = 3114 psf **at reading no.** 18
Ult. Stress = 3114 psf **at reading no.** 18

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf	Minor Eff. Stress psf	Major Eff. Stress psf	1:3 Ratio	Pore Press. psi	P psf	Q psf
0	0.0000	0.000	0.0	0.0	0	8597	8597	1.00	0.00	8597	0
1	0.0013	4.362	4.4	0.0	456	1535	1991	1.30	49.04	1763	228
2	0.0032	6.357	6.4	0.1	664	1432	2096	1.46	49.76	1764	332
3	0.0548	6.830	6.8	2.0	700	1118	1818	1.63	51.94	1468	350
4	0.0744	9.981	10.0	2.7	1015	943	1958	2.08	53.15	1450	508
5	0.0772	11.746	11.7	2.8	1194	899	2093	2.33	53.46	1496	597
6	0.0855	14.114	14.1	3.1	1430	761	2191	2.88	54.42	1476	715
7	0.0967	16.389	16.4	3.5	1654	676	2330	3.45	55.01	1503	827
8	0.1079	18.188	18.2	3.9	1828	594	2422	4.08	55.57	1508	914
9	0.1218	20.210	20.2	4.4	2020	505	2525	5.00	56.19	1515	1010
10	0.1386	21.998	22.0	5.0	2185	457	2642	5.78	56.53	1549	1092
11	0.1693	24.616	24.6	6.1	2417	386	2802	7.26	57.02	1594	1208
12	0.2041	26.390	26.4	7.4	2556	346	2902	8.39	57.30	1624	1278
13	0.2460	28.247	28.2	8.9	2691	304	2995	9.86	57.59	1649	1346
14	0.2948	30.026	30.0	10.6	2806	270	3076	11.39	57.82	1673	1403
15	0.3018	29.977	30.0	10.9	2793	275	3068	11.17	57.79	1671	1397
16	0.3437	31.783	31.8	12.4	2911	226	3137	13.90	58.13	1681	1456
17	0.3855	33.817	33.8	13.9	3044	170	3214	18.92	58.52	1692	1522
18	0.4199	35.096	35.1	15.1	3114	113	3228	28.45	58.91	1670	1557

Parameters for Specimen No. 3

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	107.440			261.460
Moisture content: Dry soil+tare, gms.	94.230			236.480
Moisture content: Tare, gms.	34.340			131.770
Moisture, %	22.1	24.8	24.1	23.9
Moist specimen weight, gms.	129.9			
Diameter, in.	1.35	1.35	1.35	
Area, in.²	1.44	1.43	1.42	
Height, in.	2.81	2.80	2.79	
Net decrease in height, in.		0.00	0.01	
Wet density, pcf	122.9	126.0	126.7	
Dry density, pcf	100.7	101.0	102.1	
Void ratio	0.6744	0.6690	0.6512	
Saturation, %	88.3	100.0	100.0	

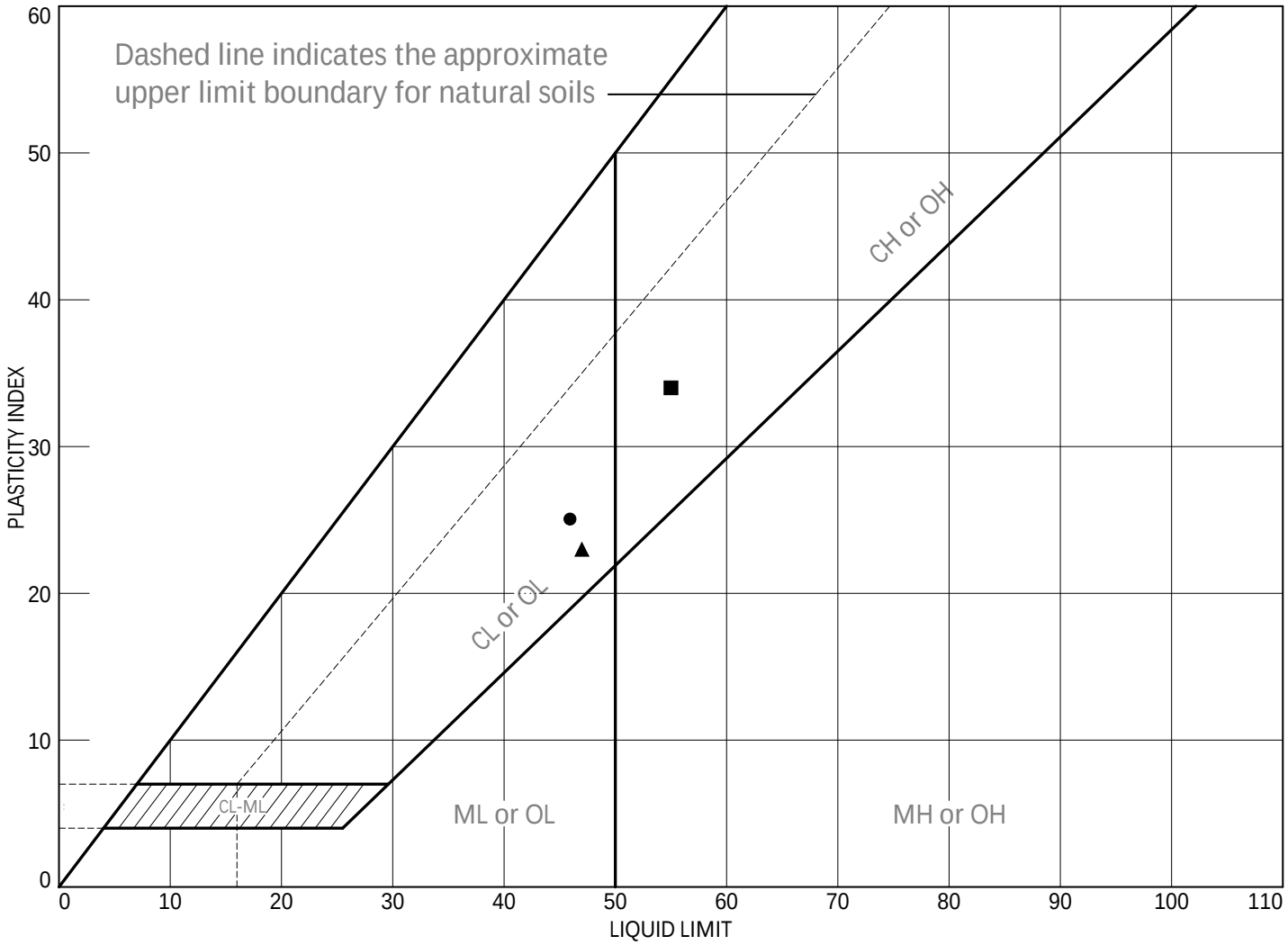
Test Readings for Specimen No. 3

Membrane modulus = 0.124105 kN/cm²
Membrane thickness = 0.02 cm
Consolidation cell pressure = 70.85 psi (10202 psf)
Consolidation back pressure = 50.00 psi (7200 psf)
Consolidation effective confining stress = 3002 psf
Strain rate, in./min. = 1.000
Fail. Stress = 9026 psf **at reading no.** 13
Ult. Stress = 9026 psf **at reading no.** 13

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf	Minor Eff. Stress psf	Major Eff. Stress psf	1:3 Ratio	Pore Press. psi	P psf	Q psf
0	0.0000	0.000	0.0	0.0	0	10202	10202	1.00	0.00	10202	0
1	0.0000	51.144	51.1	0.0	5178	2540	7718	3.04	53.21	5129	2589
2	0.0007	65.237	65.2	0.0	6603	2505	9108	3.64	53.46	5806	3302
3	0.0218	70.763	70.8	0.8	7108	1893	9001	4.76	57.70	5447	3554
4	0.0316	77.159	77.2	1.1	7723	1407	9130	6.49	61.08	5268	3862
5	0.0513	82.632	82.6	1.8	8212	977	9189	9.41	64.07	5083	4106
6	0.1018	87.302	87.3	3.6	8517	725	9242	12.74	65.81	4984	4258
7	0.1551	90.688	90.7	5.6	8671	760	9432	12.41	65.57	5096	4336
8	0.2113	93.086	93.1	7.6	8711	855	9566	11.19	64.91	5211	4356
9	0.2674	96.365	96.4	9.6	8822	943	9765	10.36	64.30	5354	4411
10	0.3235	99.596	99.6	11.6	8915	1026	9941	9.69	63.72	5484	4458
11	0.3797	102.580	102.6	13.6	8974	1099	10072	9.17	63.22	5586	4487
12	0.4148	104.560	104.6	14.9	9014	1136	10149	8.94	62.96	5642	4507
13	0.4210	104.981	105.0	15.1	9026	1141	10167	8.91	62.93	5654	4513

Borrow Area
Laboratory Testing Results

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	AD 300	Bulk 1	0	20.5	21	46	25	CL
■	AD 300	Jar 5	9.5	25.9	21	55	34	CH
▲	AD 300	Jar 7	13.5	17.1	24	47	23	CL

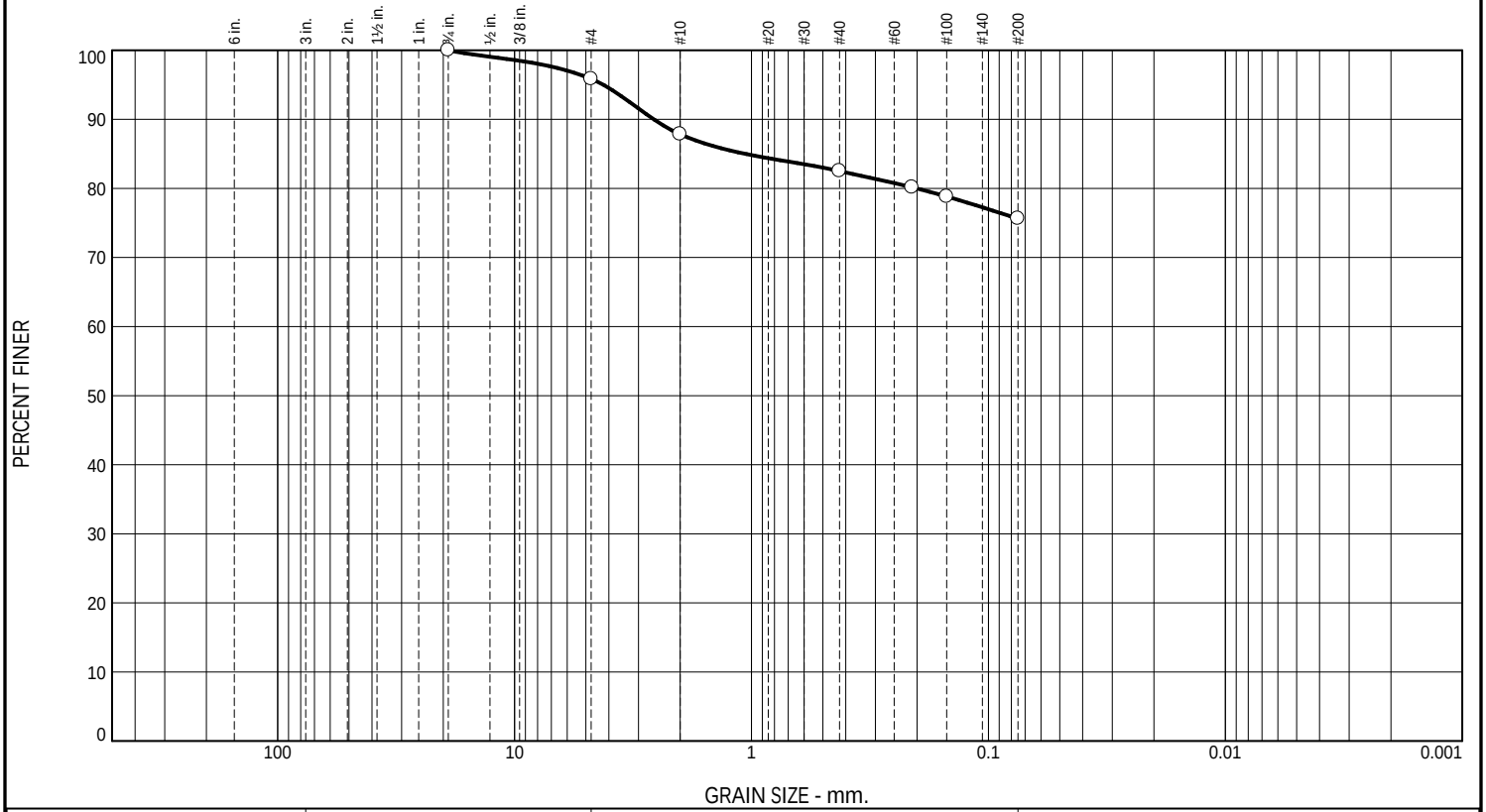
US ARMY
CORPS OF ENGINEERS
ST. LOUIS

Client: USACE NWK
Project: Upper Turkey Creek
Project No.:

Figure

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.1	8.1	5.3	6.9	75.6	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4	100.0			
#4	95.9			
#10	87.8			
#40	82.5			
#70	80.2			
#100	78.8			
#200	75.6			

* (no specification provided)

Material Description

lean clay with sand

Atterberg Limits

PL= 21 LL= 46 PI= 25

Classification

USCS= CL AASHTO= A-7-6(19)

Test Remarks

Source of Sample: AD 300 Depth: 0
Sample Number: Bulk 1

Sample Date:

US ARMY
CORPS OF ENGINEERS
ST. LOUIS

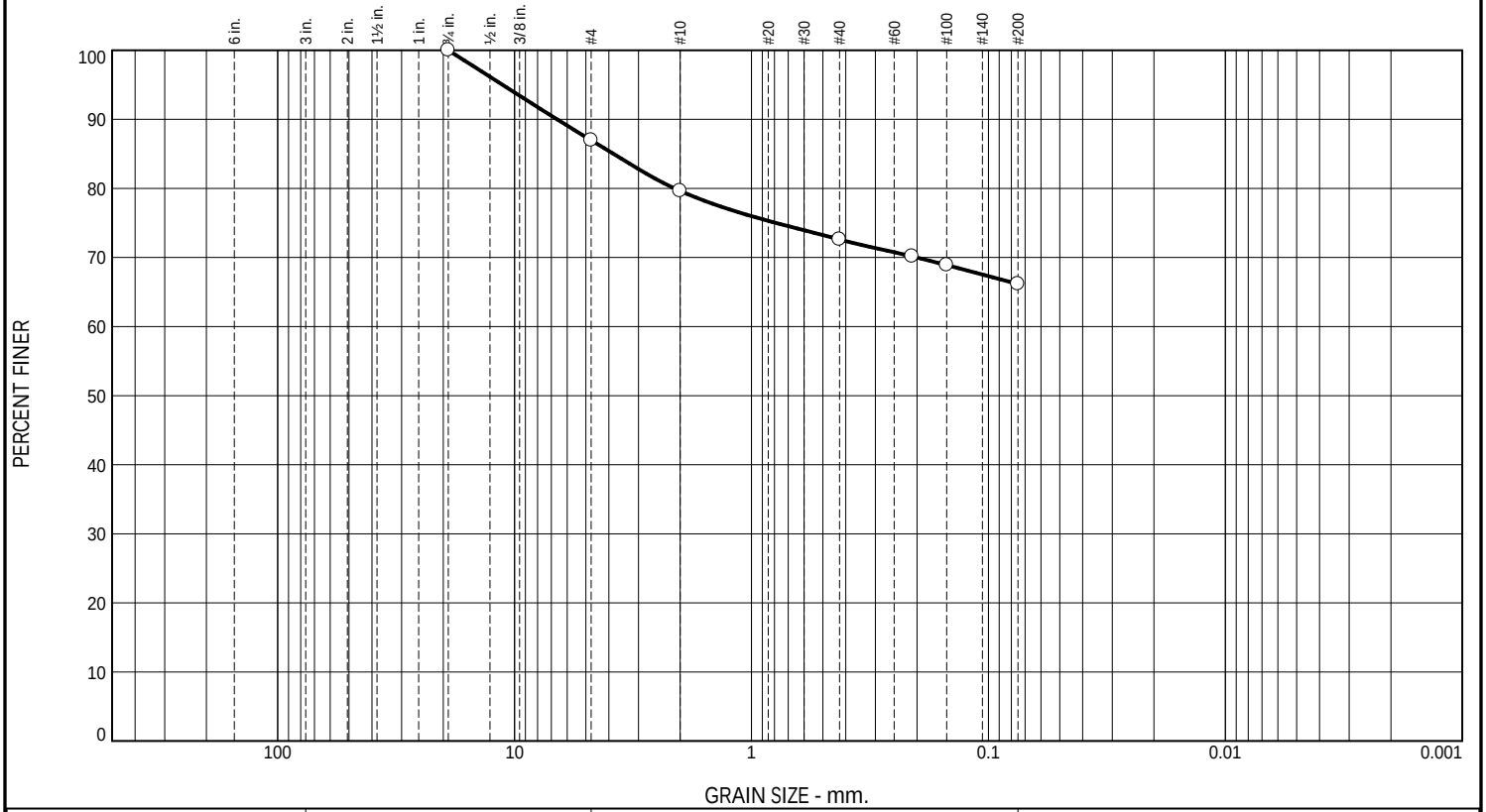
Client: USACE NWK
Project: Upper Turkey Creek

Project No: Figure

Checked By: JPC

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.0	7.4	7.0	6.4	66.2	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4	100.0			
#4	87.0			
#10	79.6			
#40	72.6			
#70	70.2			
#100	68.9			
#200	66.2			

* (no specification provided)

Material Description

sandy fat clay

Atterberg Limits

PL= 21 LL= 55 PI= 34

Classification

USCS= CH AASHTO= A-7-6(21)

Test Remarks

Source of Sample: AD 300 Depth: 9.5
Sample Number: Jar 5

Sample Date:

US ARMY
CORPS OF ENGINEERS
ST. LOUIS

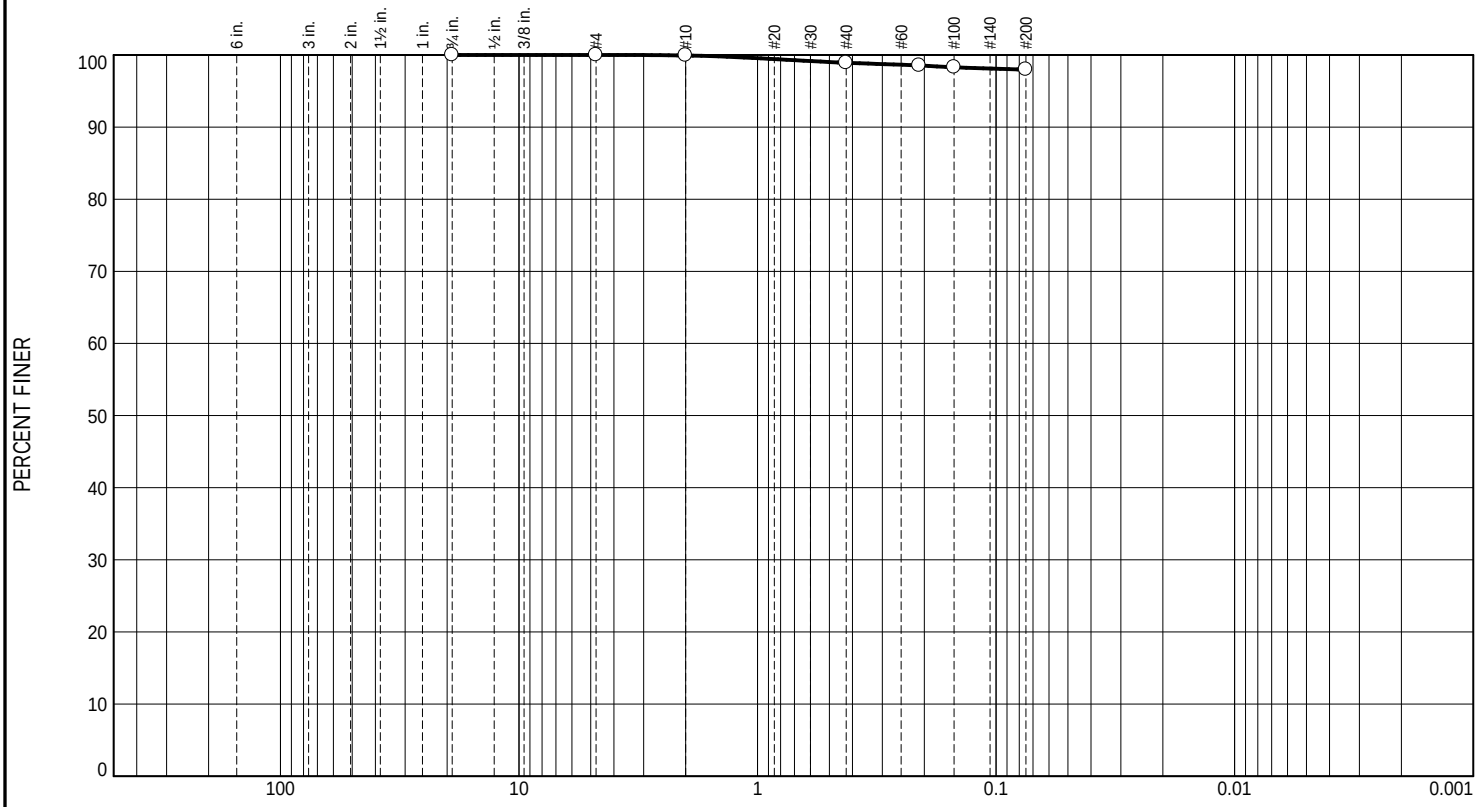
Client: USACE NWK
Project: Upper Turkey Creek

Project No: Figure

Checked By: JPC

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	1.0	0.9	98.0	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4	100.0			
#4	100.0			
#10	99.9			
#40	98.9			
#70	98.5			
#100	98.3			
#200	98.0			

* (no specification provided)

Material Description

lean clay

Atterberg Limits

PL= 24 LL= 47 PI= 23

Classification

USCS= CL AASHTO= A-7-6(26)

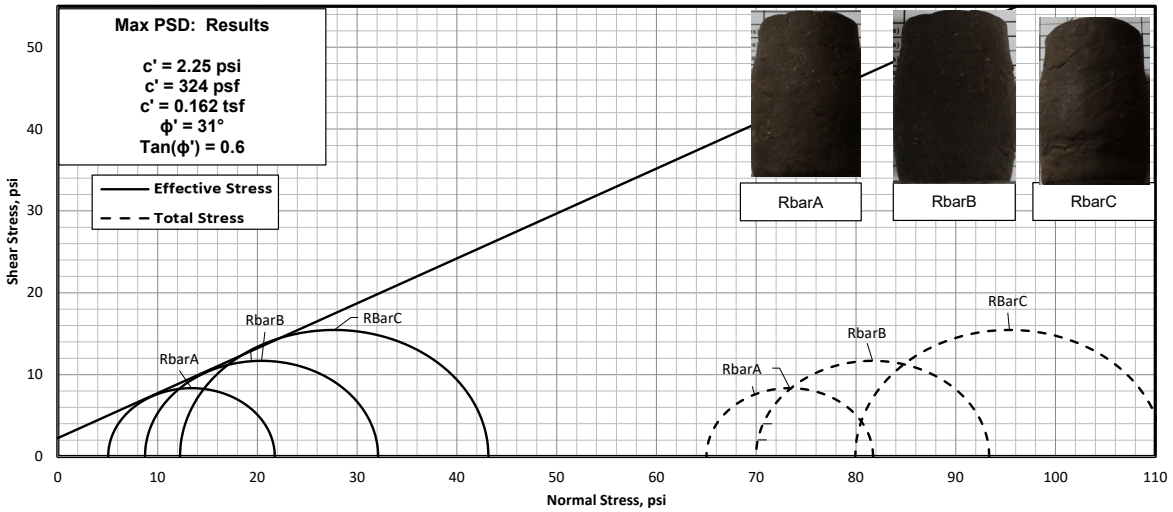
Test Remarks

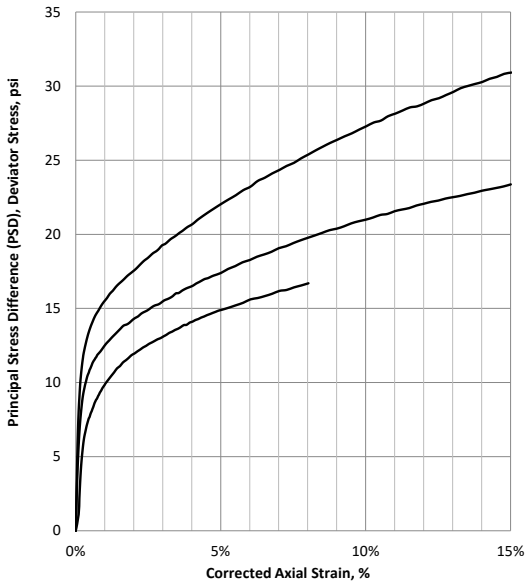
Source of Sample: AD 300 Depth: 13.5
Sample Number: Jar 7 Sample Date:

US ARMY
CORPS OF ENGINEERS
ST. LOUIS

Client: USACE NWK
Project: Upper Turkey Creek

Project No: Figure





	Sample No.	RbarA	RbarB	RbarC
Initial	Water Content, %	20.9%	20.6%	20.6%
	Dry Density, pcf	101.5	101.4	102.2
	Wet Density, pcf	122.7	122.2	123.2
	Avg Wet Density		122.7 pcf	
	Saturation, %	88.7%	86.9%	88.6%
	Void Ratio	0.624	0.625	0.613
	Diameter, in	1.420	1.420	1.420
	Height, in	2.810	2.820	2.810
BP	Total Back Pressure, psi	49.98	55.00	50.01
	Parameter B	0.97	0.97	0.95
After Consolidation	Water Content, %	23.3%	22.9%	21.5%
	Dry Density, pcf	101.7	102.2	105.1
	Wet Density, pcf	125.5	125.6	127.7
	Avg Wet Density		126.2 pcf	
	Saturation, %	99%	99%	100%
	Void Ratio	0.619	0.612	0.568
	Diameter, in	1.418	1.412	1.403
	Height, in	2.810	2.820	2.810
	$K_{o\ (Consolidation)}$	0.88	1.05	1.01
	$\sigma_{v'\ (Consolidation)}$, psi	5.79	9.29	19.86
	A_c (Method A), in^2	1.58	1.57	1.55
At Failure	Strain rate, %/hr	0.49%	0.48%	0.48%
	$\sigma_{1'\ (Max\ PSD)}$	21.75	32.11	43.16
	$\sigma_{3'\ (Max\ PSD)}$	5.05	8.74	12.25
	$u_{(Max\ PSD)}$	59.97	61.25	67.68
	Axial Strain $_{(Max\ PSD)}$	8.03%	15.01%	15.03%
	$PSR_{(MAX\ PSD)}$	4.31	3.67	3.52
	PSD_{MAX}	16.70	23.37	30.91

Type of Test: Consolidated Undrained, $\bar{R}$

Sample Type: Remold @ 95% Target

Assumed Specific Gravity: 2.64

LL: 46
PL: 21
PI: 25
USCS: CL

Remarks: Tests were conducted in general accordance with ASTM's D4767

District: NWK

Project: Upper Turkey Creek

Location:

Tube No: Bulk1

Boring: AD300

Depth:

Sample No: RbarA, RbarB, RbarC

Project No:

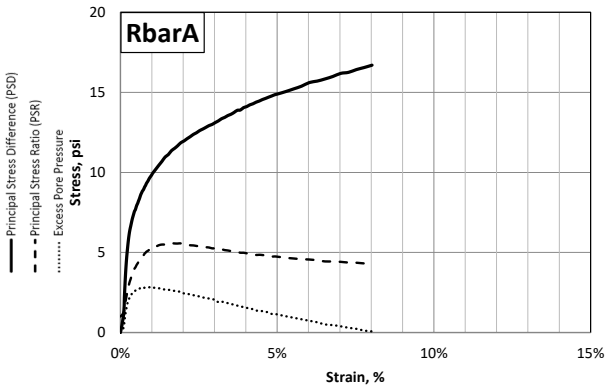
Date Sample: 1/23/2026

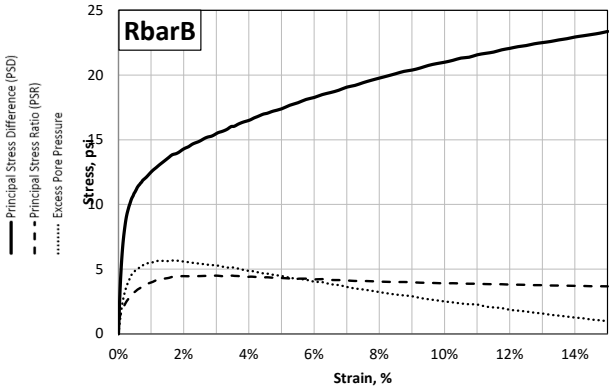
$\bar{R}$ TRIAXIAL SHEAR TEST REPORT

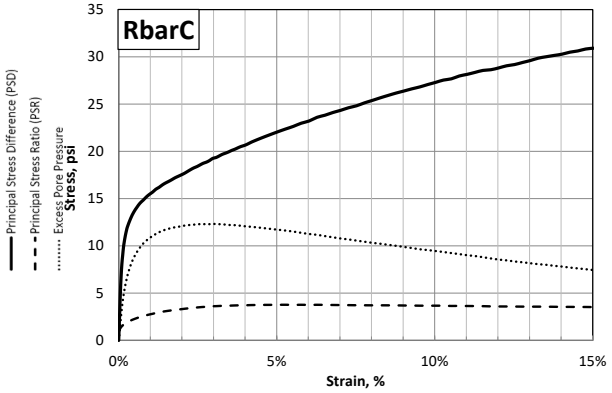
Maximum Principal Stress Difference, PSD_{Max}

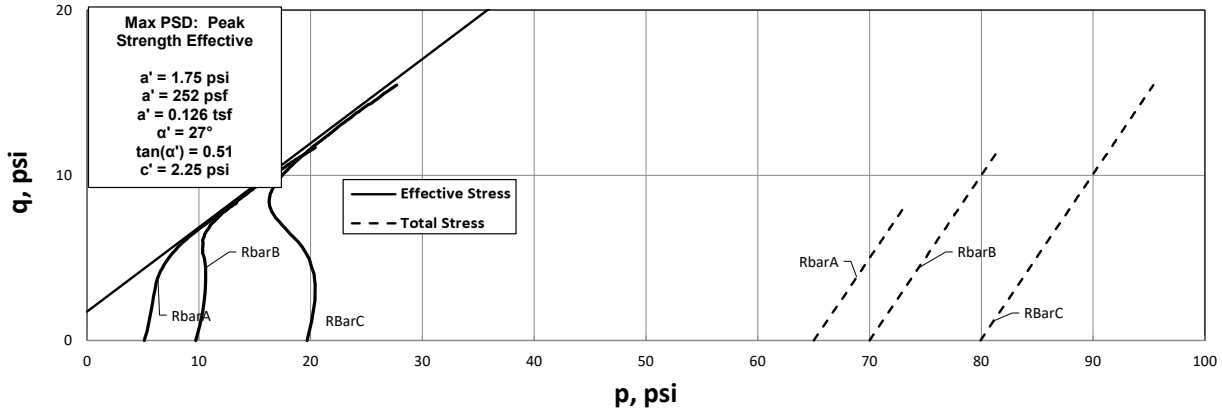
US ARMY CORPS OF ENGINEERS

Tested By:JPC





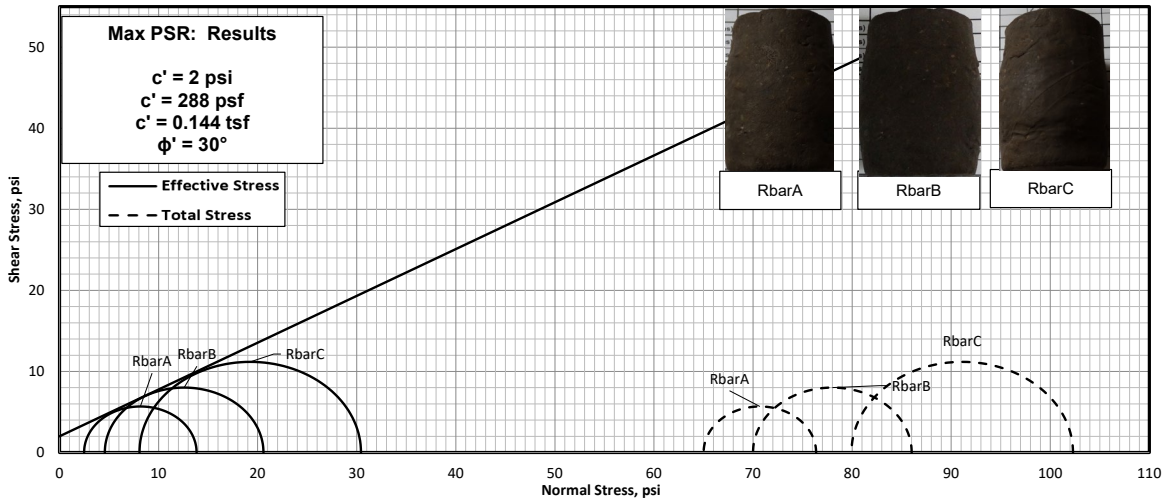


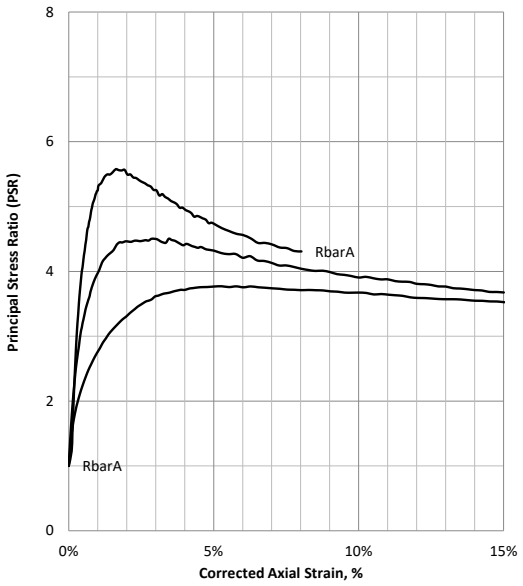


District:	NWK	**Location:**	0
Project:	Upper Turkey Creek	**Tube No:**	Bulk1
Boring:	AD300	**Depth:**	
Sample No:	RbarA, RbarB, RbarC		
Project No:		**Date Sample:**	1/23/2026

US ARMY CORPS OF ENGINEERS

Tested By:JPC





	Sample No.	RbarA	RbarB	RbarC
Initial	Water Content, %	20.9%	20.6%	20.6%
	Dry Density, pcf	101.5	101.4	102.2
	Wet Density, pcf	122.7	122.2	123.2
	Avg Wet Density		122.7 pcf	
	Saturation, %	88.7%	86.9%	88.6%
	Void Ratio	0.624	0.625	0.613
	Diameter, in	1.420	1.420	1.420
	Height, in	2.810	2.820	2.810
BP	Total Back Pressure, psi	49.98	55.00	50.01
	Parameter B	0.97	0.97	0.95
After Consolidation	Water Content, %	23.3%	22.9%	21.5%
	Dry Density, pcf	101.7	102.2	105.1
	Wet Density, pcf	125.5	125.6	127.7
	Avg Wet Density		126.2 pcf	
	Saturation, %	99%	99%	100%
	Void Ratio	0.619	0.612	0.568
	Diameter, in	1.418	1.412	1.403
	Height, in	2.810	2.820	2.810
	$K_{o\ (Consolidation)}$	0.88	1.05	1.01
	σ_v' (Consolidation), psi	5.79	9.29	19.86
	A_c (Method A), in^2	1.58	1.57	1.55
At Failure	Strain rate, %/hr	0.49%	0.48%	0.48%
	σ_1' (Max PSR)	13.83	20.58	30.43
	σ_3' (Max PSR)	2.48	4.57	8.07
	u (Max PSR)	62.53	65.43	71.87
	Axial Strain (Max PSR)	1.63%	3.45%	5.28%
	PSR_{MAX}	5.58	4.50	3.77
	PSD (Max PSR)	11.35	16.01	22.36

Type of Test: Consolidated Undrained, $\bar{R}$

Sample Type: Remold @ 95% Target

Assumed Specific Gravity: 2.64

LL: 46
PL: 21
PI: 25
USCS: CL

Remarks: Other tests were conducted in general accordance with ASTM's D4767

District: NWK

Project: Upper Turkey Creek

Location:

Tube No: Bulk1

Boring: AD300

Depth: 0-10'

Sample No: RbarA, RbarB, RbarC

Project No:

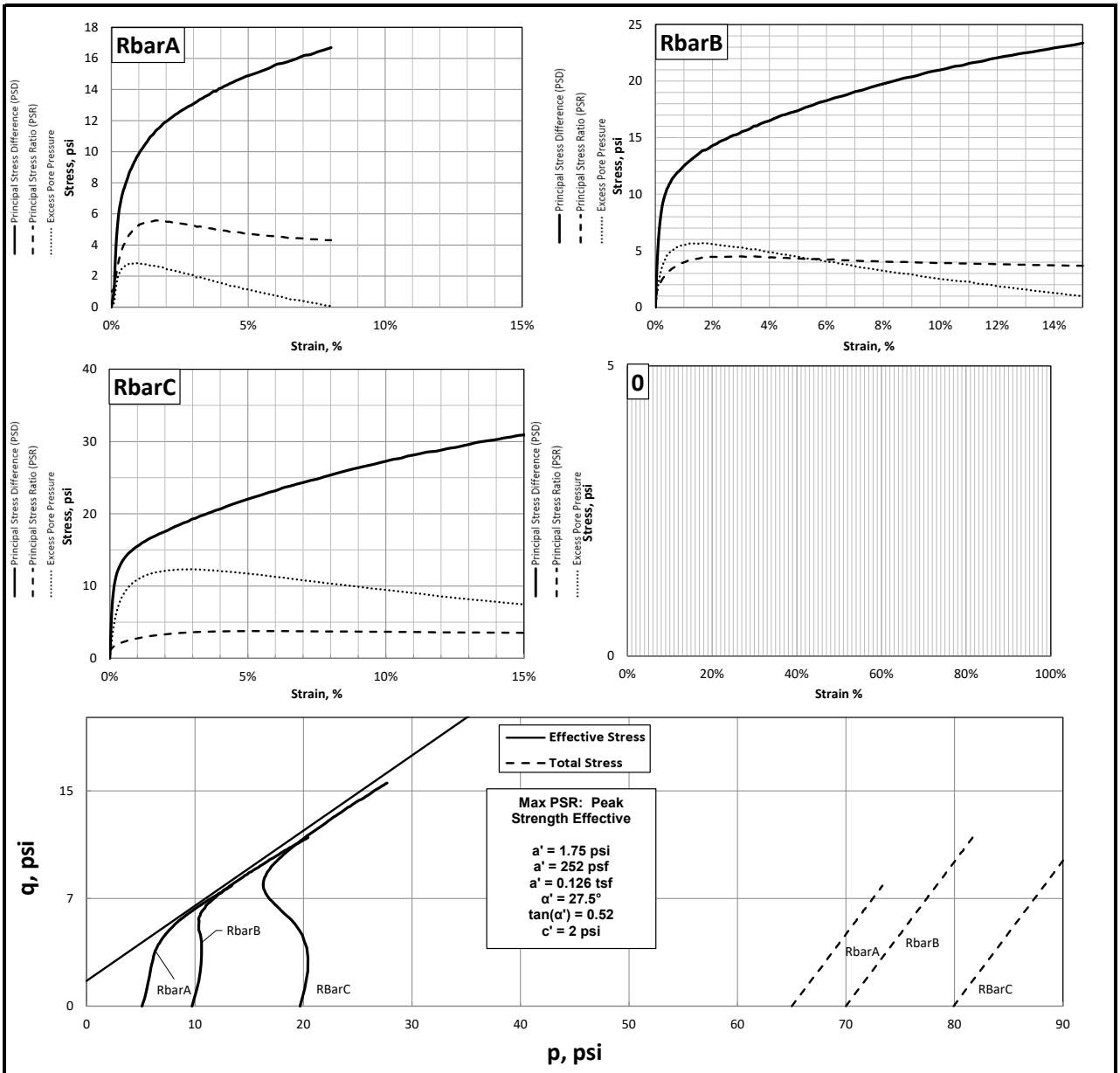
Date Sample: 1/23/2026

$\bar{R}$ TRIAXIAL SHEAR TEST REPORT

Maximum Principal Stress Ratio, PSR_{Max}

US ARMY CORPS OF ENGINEERS

Tested By:JPC



District: NWK **Location:** 0

Project: Upper Turkey Creek **Tube No:** Bulk1

Boring: AD300 **Depth:** 0-10'

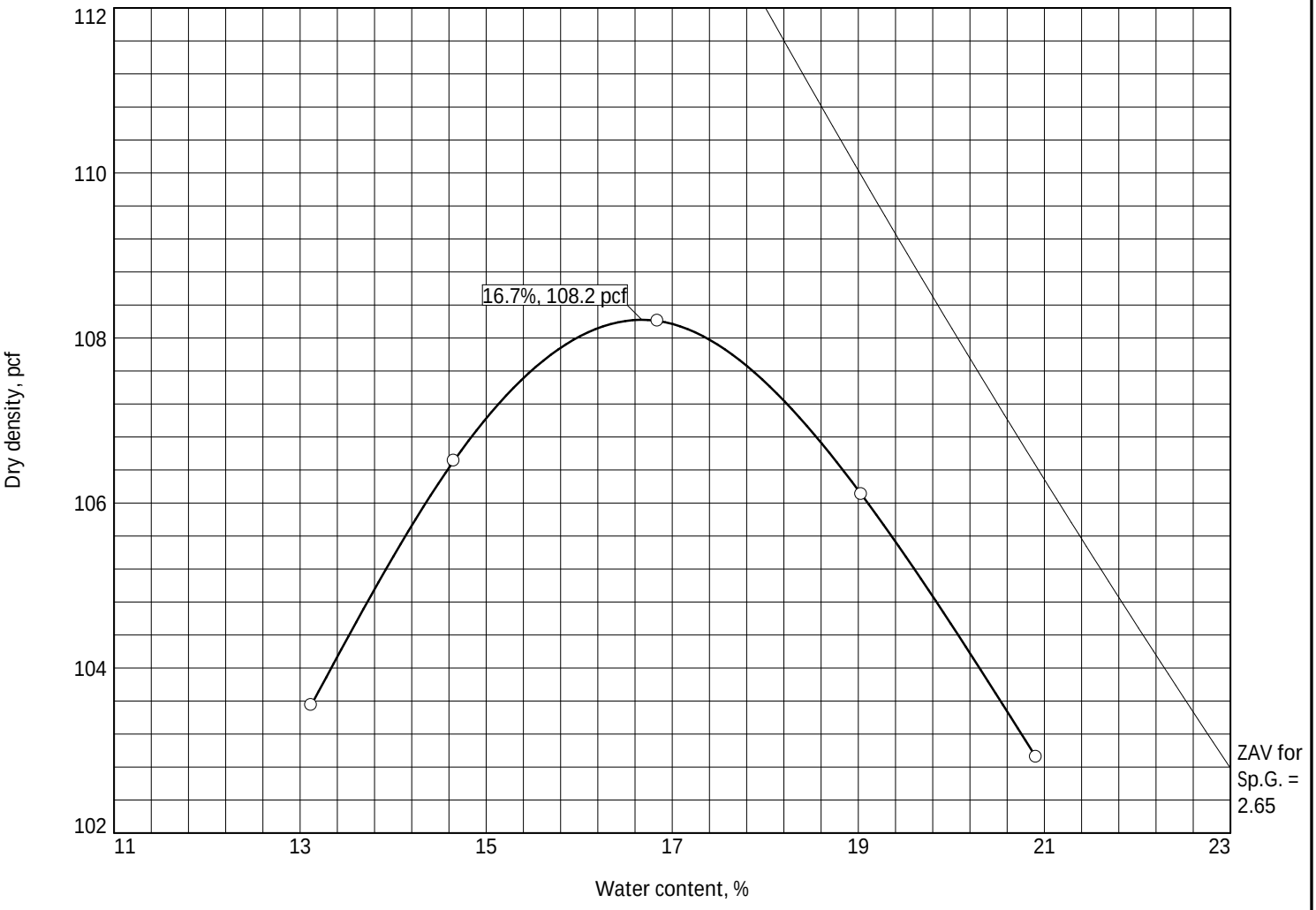
Sample No: RbarA, RbarB, RbarC

Project No: **Date Sample:** 1/23/2026

US ARMY CORPS OF ENGINEERS

Tested By:JPC

COMPACTION TEST REPORT for Curve No. 001



Test specification: ASTM D 698-07 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0	CL	A-7-6(19)	20.5	2.65	46	25	4.1	75.6

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 108.2 pcf Optimum moisture = 16.7 %	lean clay with sand

Project No. Client: USACE NWK

Project: Upper Turkey Creek

Date: 1/15/26

○ Source of Sample: AD 300 Sample Number: Bulk 1

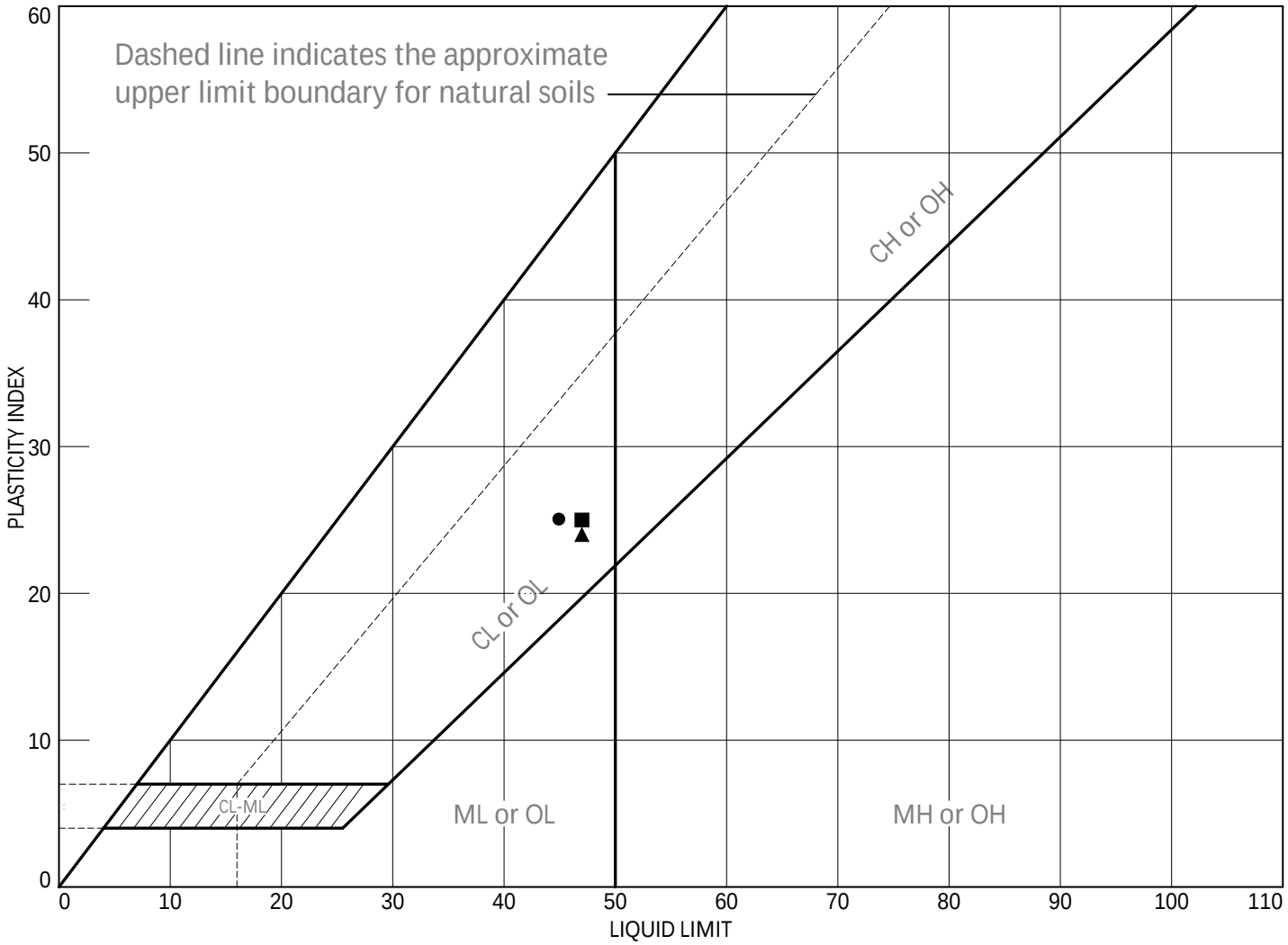
Remarks:

US ARMY CORPS OF ENGINEERS - ST. LOUIS

Figure

Tested By: JTH/JPC Checked By: JPC

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	AD-301	Bulk 1	0	22.0	20	45	25	CL
■	AD-301	Jar 2	2.0	20.4	22	47	25	CL
▲	AD-301	Jar 5	4	20.2	23	47	24	CL

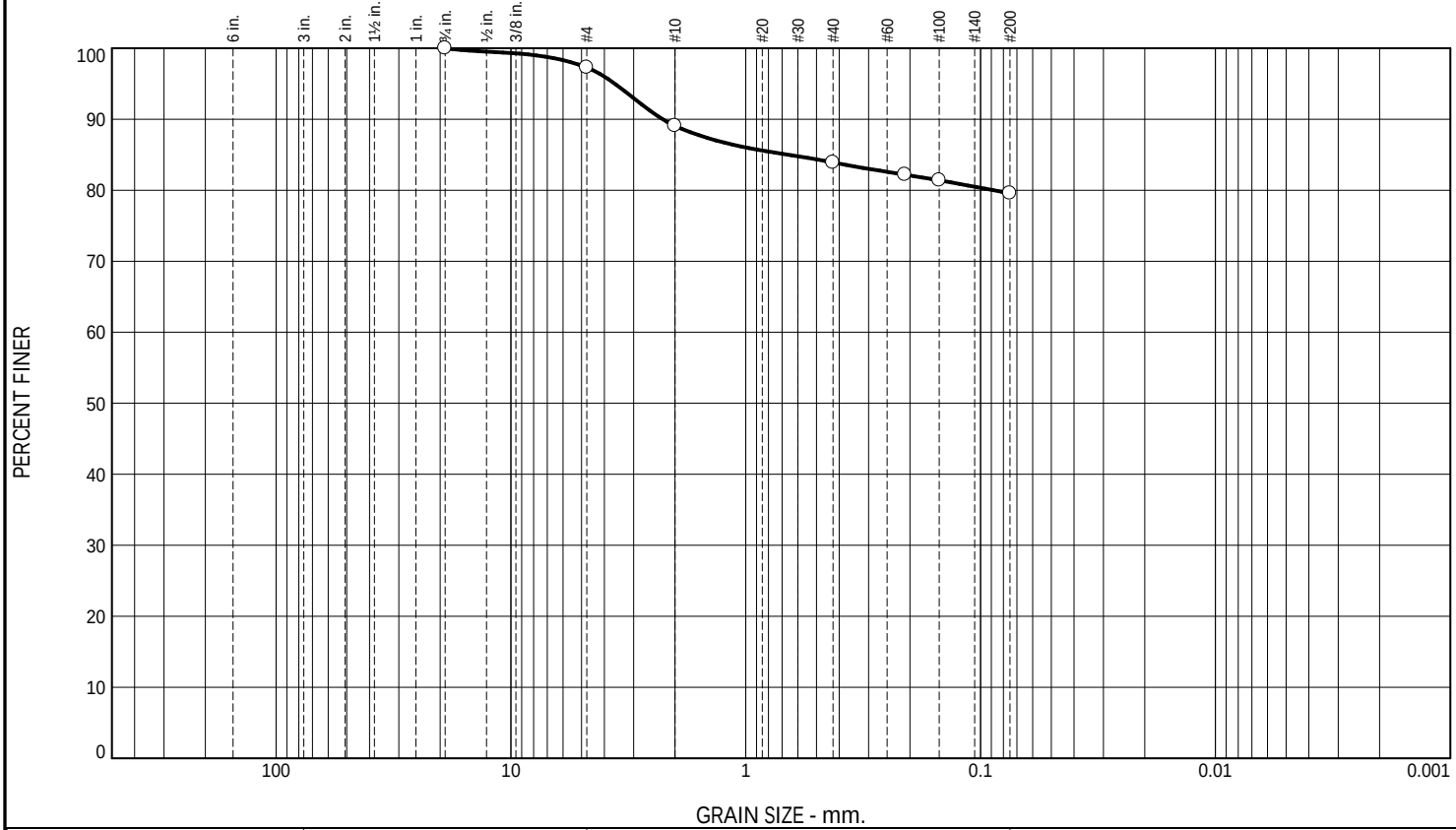
US ARMY
CORPS OF ENGINEERS
ST. LOUIS

Client: USACE NWK
Project: Upper Turkey Creek

Project No.: Figure

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.7	8.2	5.2	4.3	79.6	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4	100.0			
#4	97.3			
#10	89.1			
#40	83.9			
#70	82.2			
#100	81.4			
#200	79.6			

* (no specification provided)

Material Description

lean clay with sand

Atterberg Limits

PL= 20 LL= 45 PI= 25

Classification

USCS= CL AASHTO= A-7-6(20)

Test Remarks

Source of Sample: AD-301 Depth: 0

Sample Number: Bulk 1 Sample Date:

US ARMY
CORPS OF ENGINEERS
ST. LOUIS

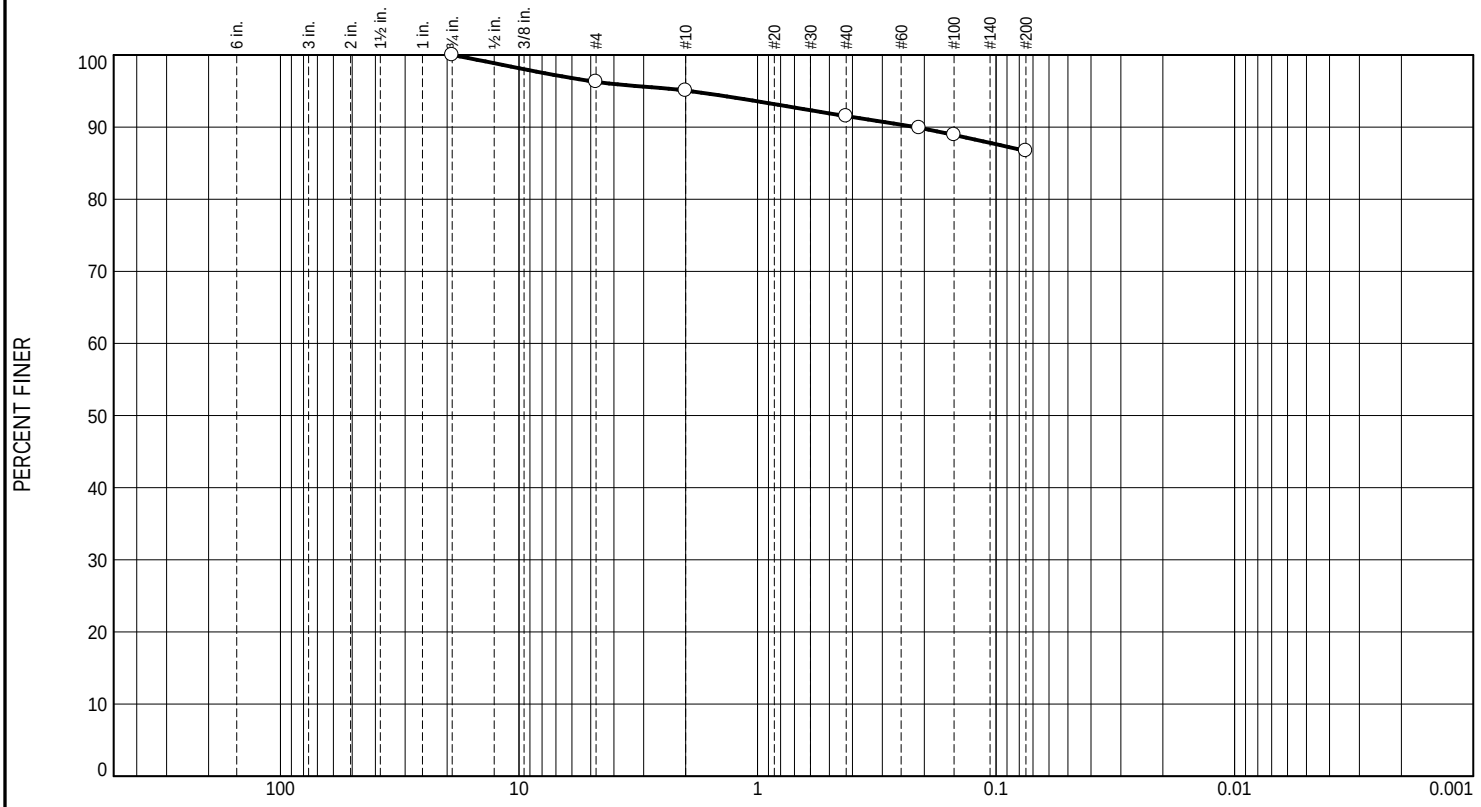
Client: USACE NWK

Project: Upper Turkey Creek

Project No: Figure

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.7	1.2	3.6	4.8	86.7	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4	100.0			
#4	96.3			
#10	95.1			
#40	91.5			
#70	89.9			
#100	88.9			
#200	86.7			

* (no specification provided)

Material Description

lean clay

Atterberg Limits

PL= 22 LL= 47 PI= 25

Classification

USCS= CL AASHTO= A-7-6(23)

Test Remarks

Source of Sample: AD-301 Depth: 2.0
Sample Number: Jar 2

Sample Date:

US ARMY
CORPS OF ENGINEERS
ST. LOUIS

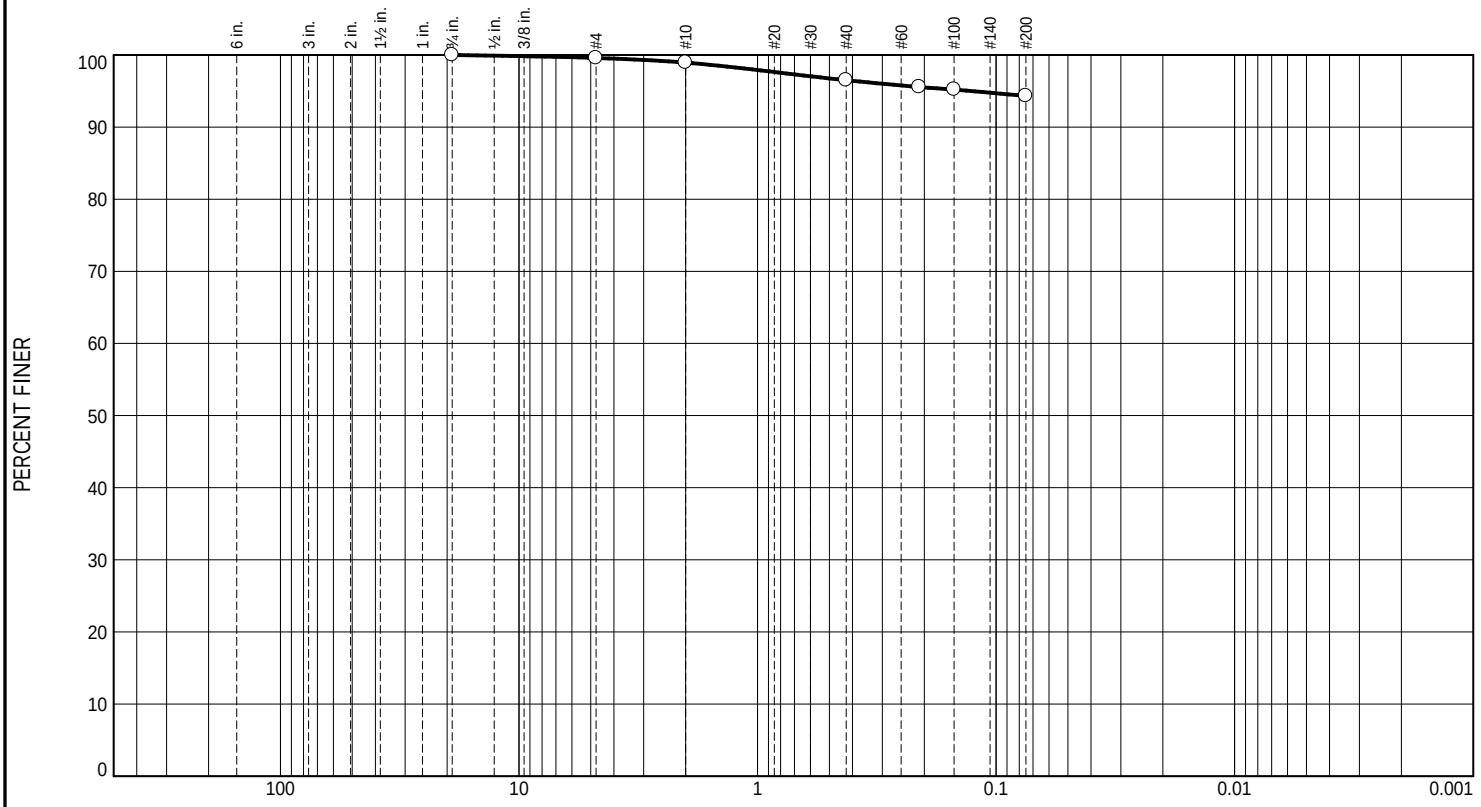
Client: USACE NWK
Project: Upper Turkey Creek

Project No:

Figure

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.4	0.7	2.4	2.2	94.3	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
3/4	100.0			
#4	99.6			
#10	98.9			
#40	96.5			
#70	95.6			
#100	95.2			
#200	94.3			

* (no specification provided)

Material Description

lean clay

Atterberg Limits

PL= 23 LL= 47 PI= 24

Classification

USCS= CL AASHTO= A-7-6(25)

Test Remarks

Source of Sample: AD-301 Depth: 4
Sample Number: Jar 5

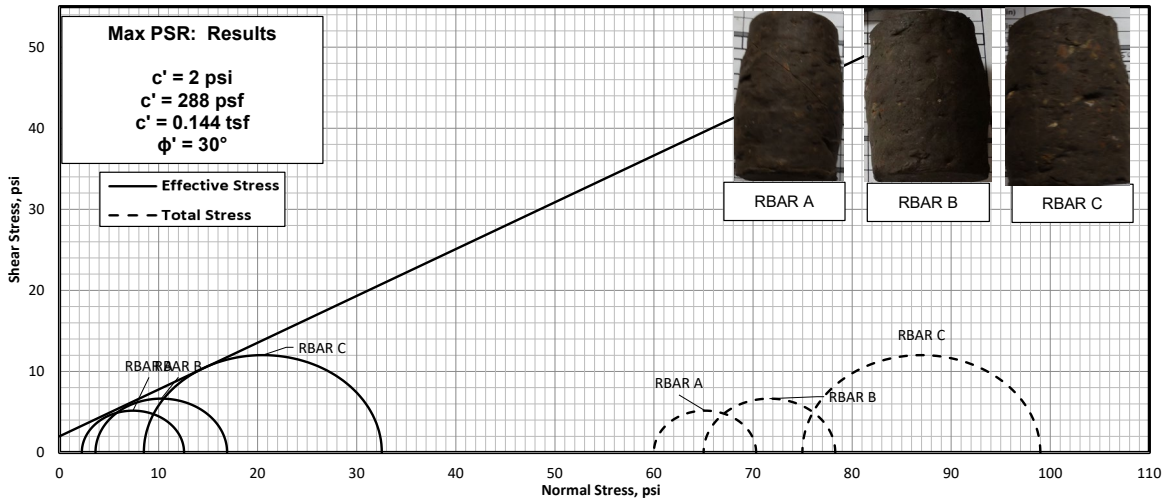
Sample Date:

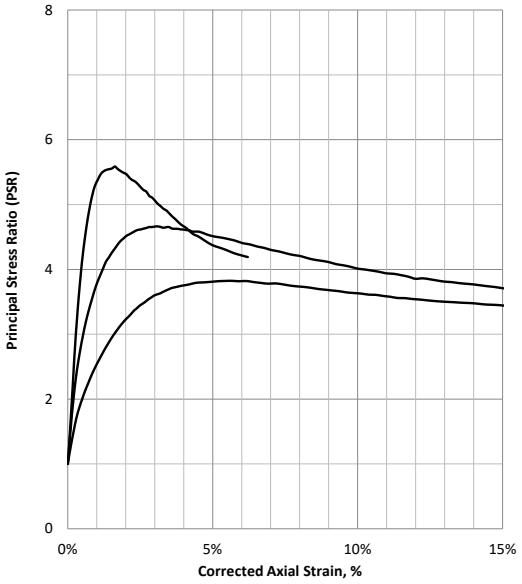
US ARMY
CORPS OF ENGINEERS
ST. LOUIS

Client: USACE NWK
Project: Upper Turkey Creek

Project No:

Figure





	Sample No.	RBAR A	RBAR B	RBAR C
Initial	Water Content, %	19.9%	19.6%	20.2%
	Dry Density, pcf	101.8	102.4	101.7
	Wet Density, pcf	122.0	122.5	122.3
	Avg Wet Density		122.3 pcf	
	Saturation, %	84.3%	84.6%	85.8%
	Void Ratio	0.625	0.614	0.625
	Diameter, in	1.420	1.420	1.420
	Height, in	2.840	2.830	2.840
BP	Total Back Pressure, psi	54.98	54.94	55.00
	Parameter B	0.96	0.97	0.95
After Consolidation	Water Content, %	22.7%	22.0%	21.6%
	Dry Density, pcf	101.8	104.2	104.3
	Wet Density, pcf	124.9	127.2	126.8
	Avg Wet Density		126.3 pcf	
	Saturation, %	97%	100%	98%
	Void Ratio	0.625	0.586	0.586
	Diameter, in	1.420	1.405	1.403
	Height, in	2.840	2.830	2.840
	$K_{o\ (Consolidation)}$	0.88	1.03	1.01
	σ_v' (Consolidation), psi	5.75	9.42	19.97
	A_c (Method A), in^2	1.58	1.55	1.55
At Failure	Strain rate, %/hr	0.49%	0.50%	0.50%
	σ_1' (Max PSR)	12.57	16.92	32.53
	σ_3' (Max PSR)	2.25	3.63	8.51
	u (Max PSR)	57.74	61.38	66.47
	Axial Strain (Max PSR)	1.63%	3.10%	5.66%
	PSR_{MAX}	5.59	4.66	3.82
	PSD (Max PSR)	10.32	13.29	24.02

Type of Test: Consolidated Undrained, $\bar{R}$

Sample Type: Remold @ 95% Target

Assumed Specific Gravity: 2.65

LL: 45
PL: 20
PI: 25
USCS: CL

Remarks: Other tests were conducted in general accordance with ASTM's D4767

District: NWK

Project: Upper Turkey Creek

Location:

Tube No: BULK1

Boring: AD301

Depth: 0-10'

Sample No: RBAR A, RBAR B, RBAR C

Project No:

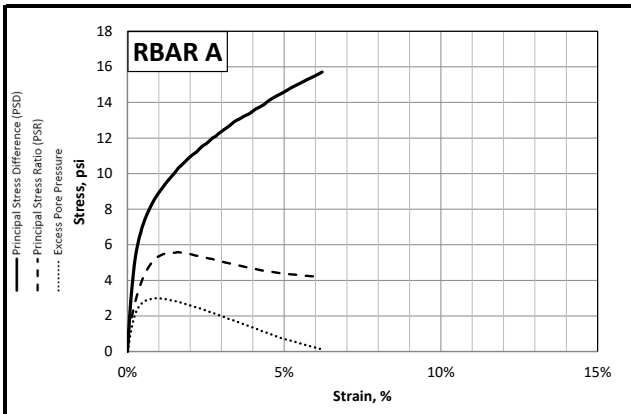
Date Sample: 1/28/2026

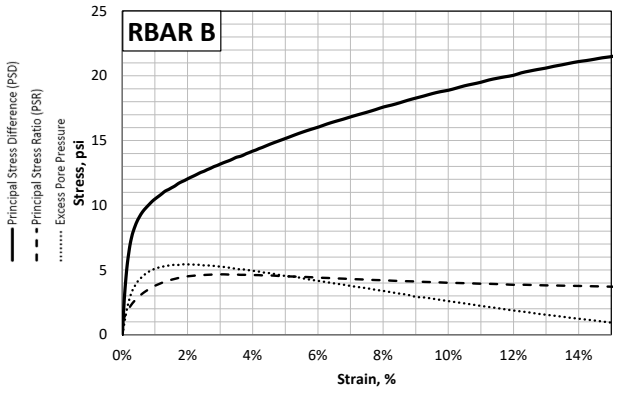
$\bar{R}$ TRIAXIAL SHEAR TEST REPORT

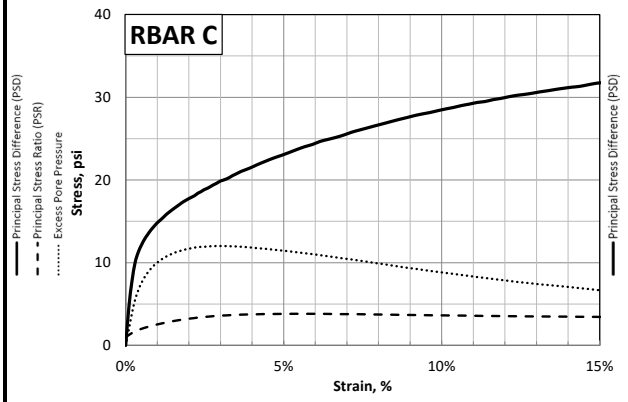
Maximum Principal Stress Ratio, PSR_{Max}

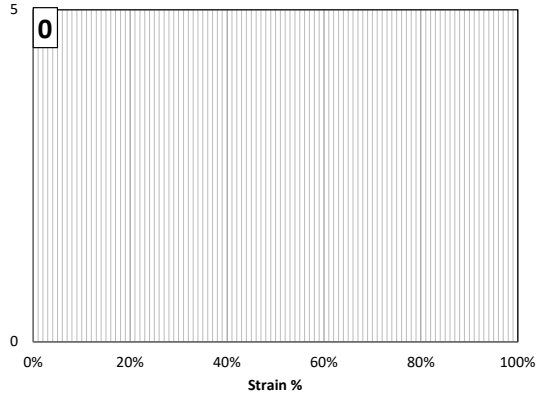
US ARMY CORPS OF ENGINEERS

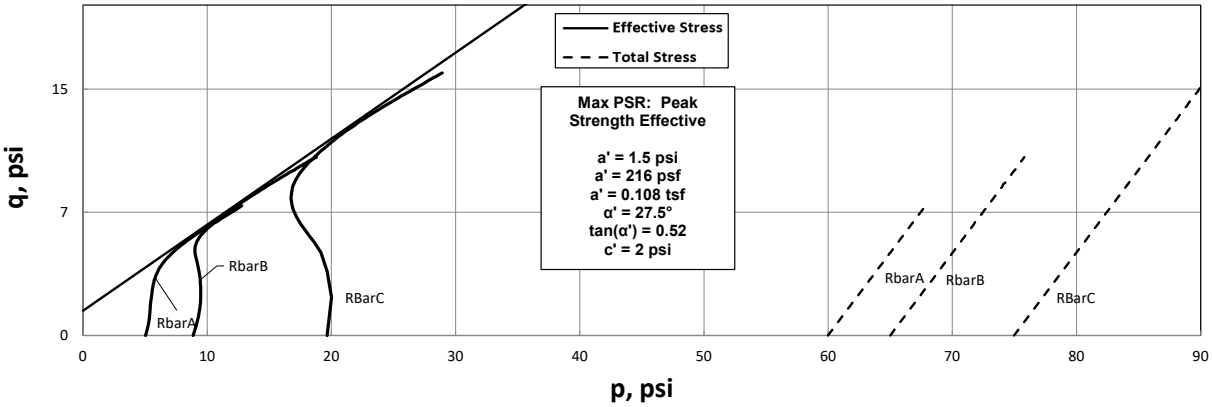
Tested By:JPC







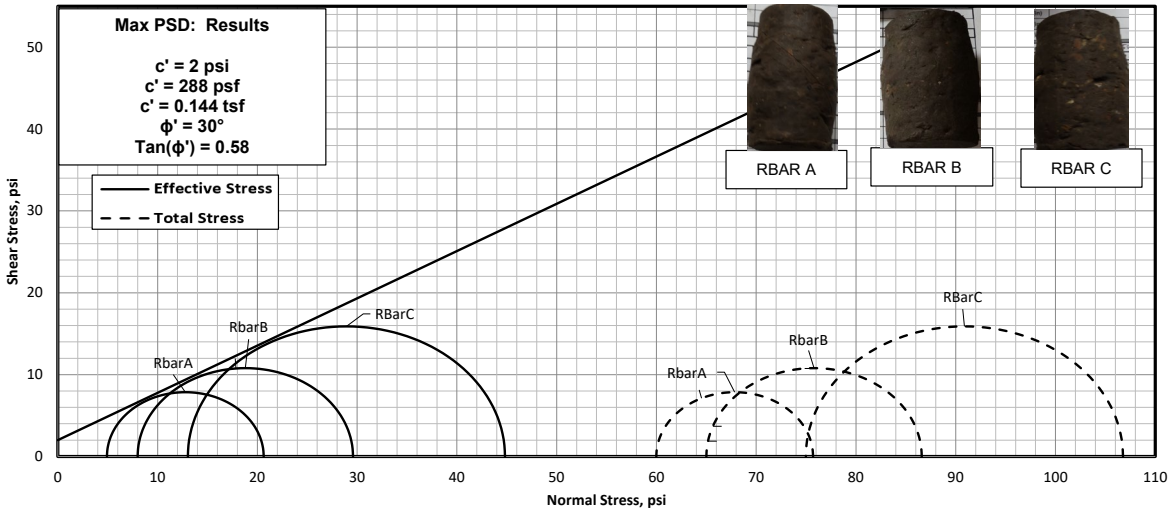


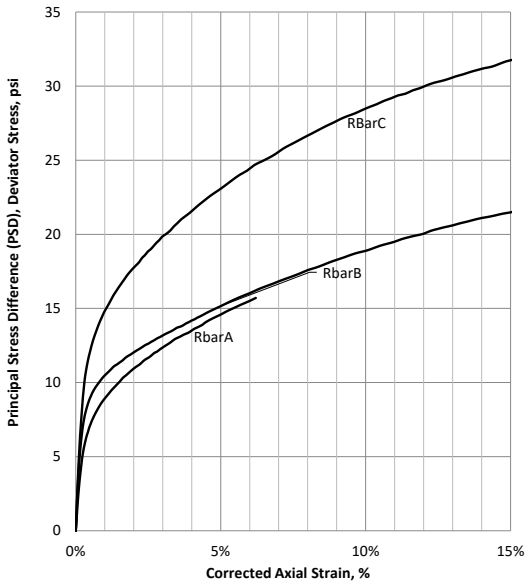


District:	NWK	**Location:**	0
Project:	Upper Turkey Creek	**Tube No:**	BULK1
Boring:	AD301	**Depth:**	0-10'
Sample No:	RBAR A, RBAR B, RBAR C		
Project No:		**Date Sample:**	1/28/2026

US ARMY CORPS OF ENGINEERS

Tested By:JPC





	Sample No.	RBAR A	RBAR B	RBAR C
Initial	Water Content, %	19.9%	19.6%	20.2%
	Dry Density, pcf	101.8	102.4	101.7
	Wet Density, pcf	122.0	122.5	122.3
	Avg Wet Density		122.3 pcf	
	Saturation, %	84.3%	84.6%	85.8%
	Void Ratio	0.625	0.614	0.625
	Diameter, in	1.420	1.420	1.420
	Height, in	2.840	2.830	2.840
BP	Total Back Pressure, psi	54.98	54.94	55.00
	Parameter B	0.96	0.97	0.95
After Consolidation	Water Content, %	22.7%	22.0%	21.6%
	Dry Density, pcf	101.8	104.2	104.3
	Wet Density, pcf	124.9	127.2	126.8
	Avg Wet Density		126.3 pcf	
	Saturation, %	97%	100%	98%
	Void Ratio	0.625	0.586	0.586
	Diameter, in	1.420	1.405	1.403
	Height, in	2.840	2.830	2.840
	$K_{o\ (Consolidation)}$	0.88	1.03	1.01
	$\sigma_v'_{(Consolidation)}$, psi	5.75	9.42	19.97
	A_c (Method A), in^2	1.58	1.55	1.55
At Failure	Strain rate, %/hr	0.49%	0.50%	0.50%
	$\sigma_1'_{(Max\ PSD)}$	20.63	29.59	44.82
	$\sigma_3'_{(Max\ PSD)}$	4.92	8.00	13.03
	$u_{(Max\ PSD)}$	55.06	57.01	61.94
	Axial Strain $_{(Max\ PSD)}$	6.21%	15.22%	15.10%
	$PSR_{(MAX\ PSD)}$	4.19	3.70	3.44
	PSD_{MAX}	15.71	21.59	31.79

Type of Test: Consolidated Undrained, $\bar{R}$

Sample Type: Remold @ 95% Target

Assumed Specific Gravity: 2.65

	RBAR A	RBAR B	RBAR C
LL:	45		
PL:	20		
PI:	25		
USCS:	CL		

Remarks: Tests were conducted in general accordance with ASTM's D4767

District: NWK

Project: Upper Turkey Creek

Location: **Tube No:** BULK1

Boring: AD301 **Depth:** 0-10'

Sample No: RBAR A, RBAR B, RBAR C

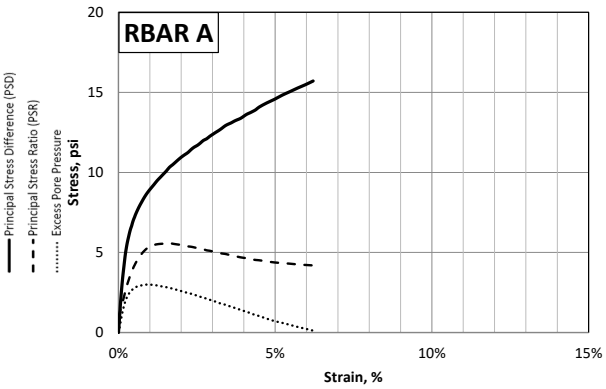
Project No: **Date Sample:** 1/28/2026

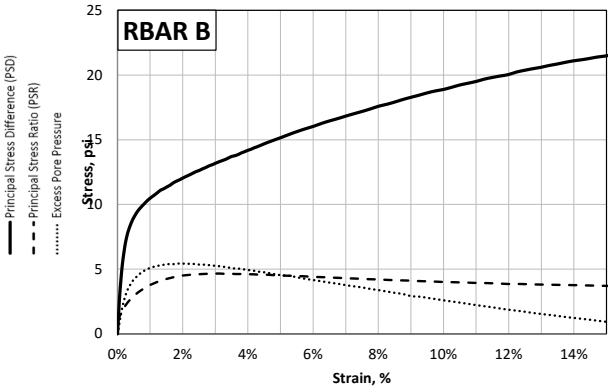
$\bar{R}$ TRIAXIAL SHEAR TEST REPORT

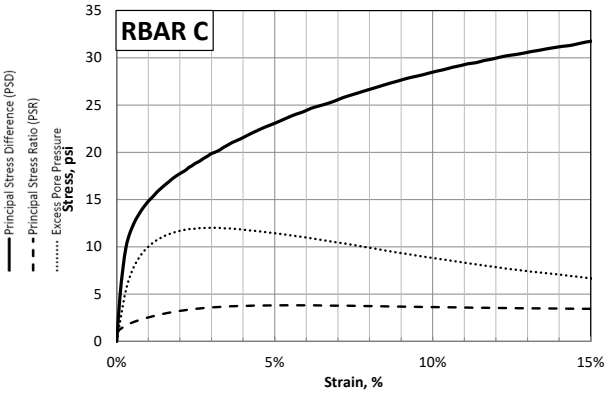
Maximum Principal Stress Difference, PSD_{Max}

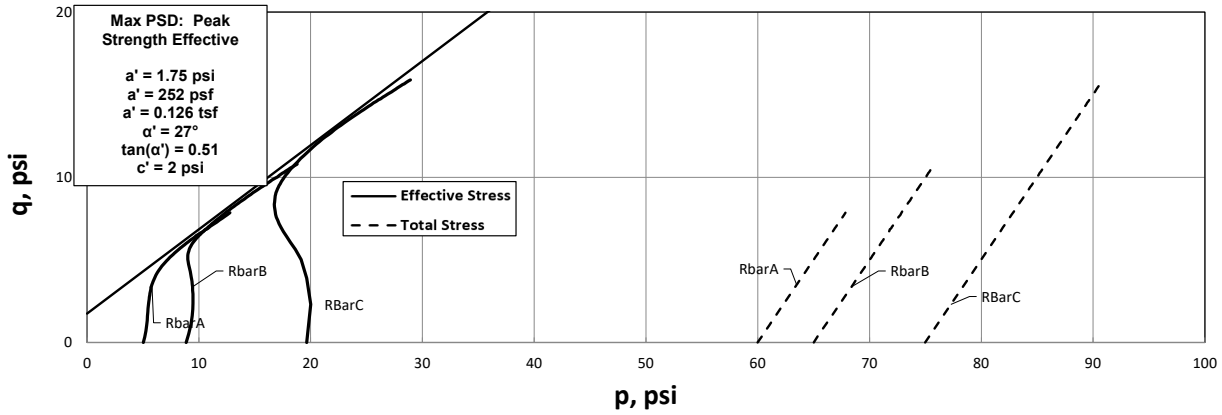
US ARMY CORPS OF ENGINEERS

Tested By:JPC







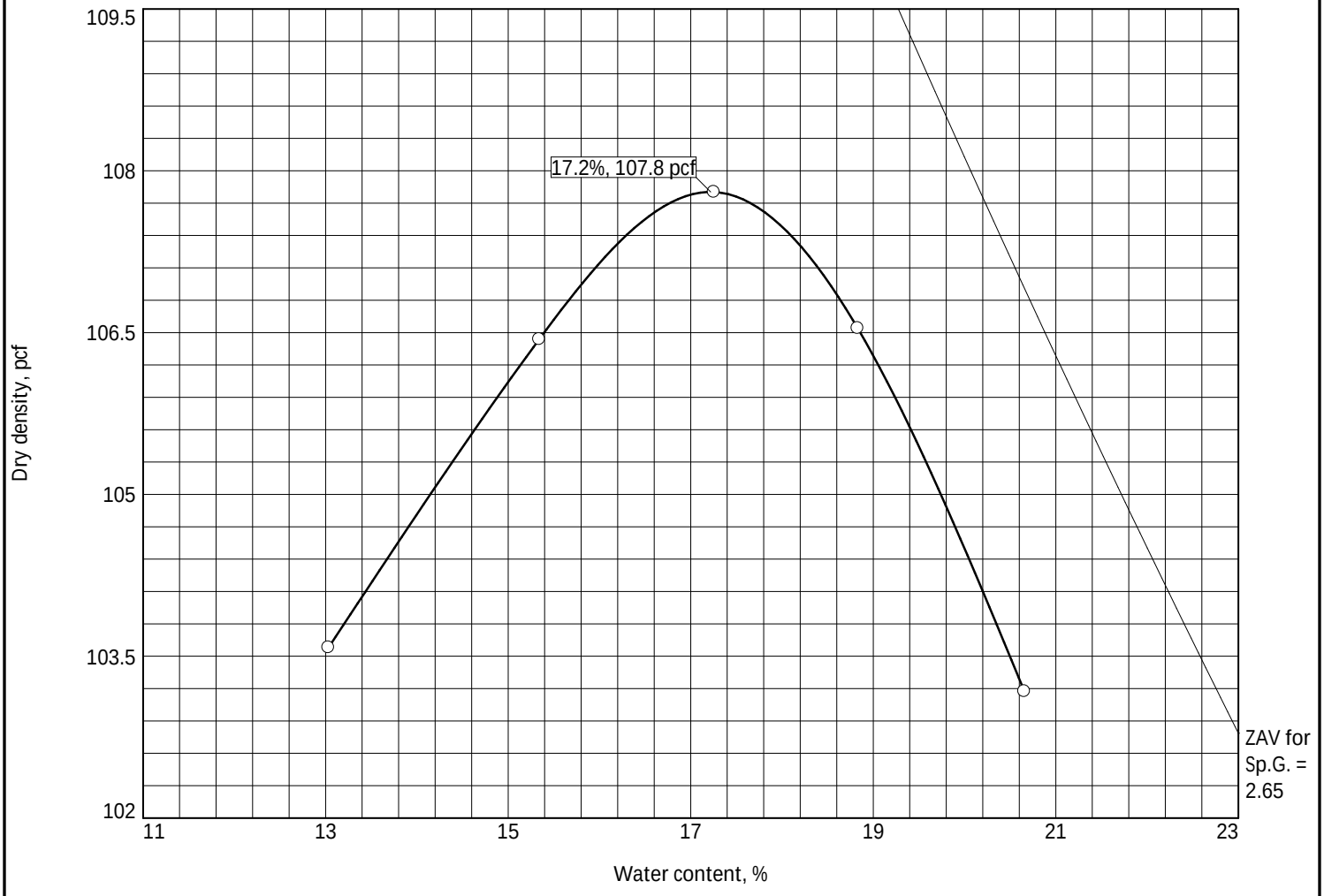


District:	NWK	**Location:**	0
Project:	Upper Turkey Creek	**Tube No:**	BULK1
Boring:	AD301	**Depth:**	0-10'
Sample No:	RBAR A, RBAR B, RBAR C		
Project No:		**Date Sample:**	1/28/2026

US ARMY CORPS OF ENGINEERS

Tested By:JPC

COMPACTION TEST REPORT for Curve No. 002



Test specification: ASTM D 698-07 Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0	CL	A-7-6(20)	22.0	2.65	45	25	2.7	79.6

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 107.8 pcf Optimum moisture = 17.2 %	lean clay with sand

Project No. Client: USACE NWK

Project: Upper Turkey Creek

Date:

○ Source of Sample: AD-301 Sample Number: Bulk 1

Remarks:

US ARMY CORPS OF ENGINEERS - ST. LOUIS

Figure

Tested By: JTH JPC

Checked By: JPC

APPENDIX B – GEOTECHNICAL

B.1 Underseepage
B.2 Global Slope Stability
B.3 I-Wall Design
B.4 Floodwall Bearing Capacity
B.5 Levee and Floodwall Settlement
B.6 Riprap and Bedding Design
B.7 Floodwall Toe Drain Design

This page was intentionally left blank for duplex printing.

APPENDIX B.1 – UNDERSEEPAGE

Underseepage Assumptions

Boring Profile

Underseepage Analysis

Regional Underseepage Criteria

District Underseepage Criteria

Underseepage Assumptions

- Divided the levee alignment into 5 reaches. As the subsurface profile and distance from the levee system to the creek were relatively consistent throughout the alignment, the divisions were made primarily based on top of bedrock elevation.

- Due to existing ground surface elevation differences (and some variation in top of bedrock elevations) between the left and right banks, subdivided the reaches as Reach 1 LB, Reach 1 RB, etc.

- Landside and riverside ground elevations for underseepage are based primarily on the levee centerline existing elevation. The site is generally uphill landward of the levee centerline.

- Levee stationing is given according to the August 2025 alignment, which will likely change, and according to the UTC creek centerline stationing, which will remain constant through the project. If there is a discrepancy, the UTC creek stationing should be referenced.

- Seepage lengths are based on the alignment shown in the August 2025 kmz file.

- Within each reach on each bank, selected the tallest levee and floodwall for underseepage analysis. Not all reaches had all feature types.

- The soil profile shows upper fill soils occasionally listed as gravelly or with debris. It is assumed that the overall soil matrix is clayey and there is no continuous pervious zone. An inspection trench will be excavated during construction. If evidence of a continuous pervious zone is found, an impervious or grouted cutoff may be required.

- The soil profile shows an inconsistent gravelly clay/clayey gravel zone near top of bedrock. It is not found at all in many borings and is up to 8.8 feet thick in other borings, making it difficult to develop a design soil profile for each reach. In general, the pervious zone due to sand or gravel was averaged to one or two feet thick.

- As the general soil profile is clayey fill or clayey natural soil down to shallow bedrock with no consistent foundation sand layer, the underseepage analysis is different from the Argentine Levee Raise, Manhattan Levee Raise, Topeka projects, and other recent analyses. It is not known if the lower gravelly clay zone is pervious enough to transmit a significant amount of seepage or if the more likely seepage source would be the bedrock.

- It was decided to perform the underseepage analysis with a larger pervious zone in the gravel or bedrock (worst case).

- As the permeability of bedrock is difficult to predict, it was assumed there would be no limiting factor as to the thickness of the pervious zone in blanket theory calculations. The top of bedrock elevation (bottom of pervious blanket in the blanket theory spreadsheet calculations) was lowered in even increments of 10 feet (example from EL 915 to EL 910 to EL 900, etc.) until the change in vertical underseepage factor of safety for a 10-foot difference was less than 0.05.

- Underseepage pressures in slope stability analyses were calibrated to match the head at toe elevation in the blanket theory spreadsheet.

- All underseepage and stability runs are based on steady state conditions with top of levee loading from Upper Turkey Creek. As this is a flashy creek, long term steady state conditions are not expected to occur. This assumption brackets the worst-case scenario.

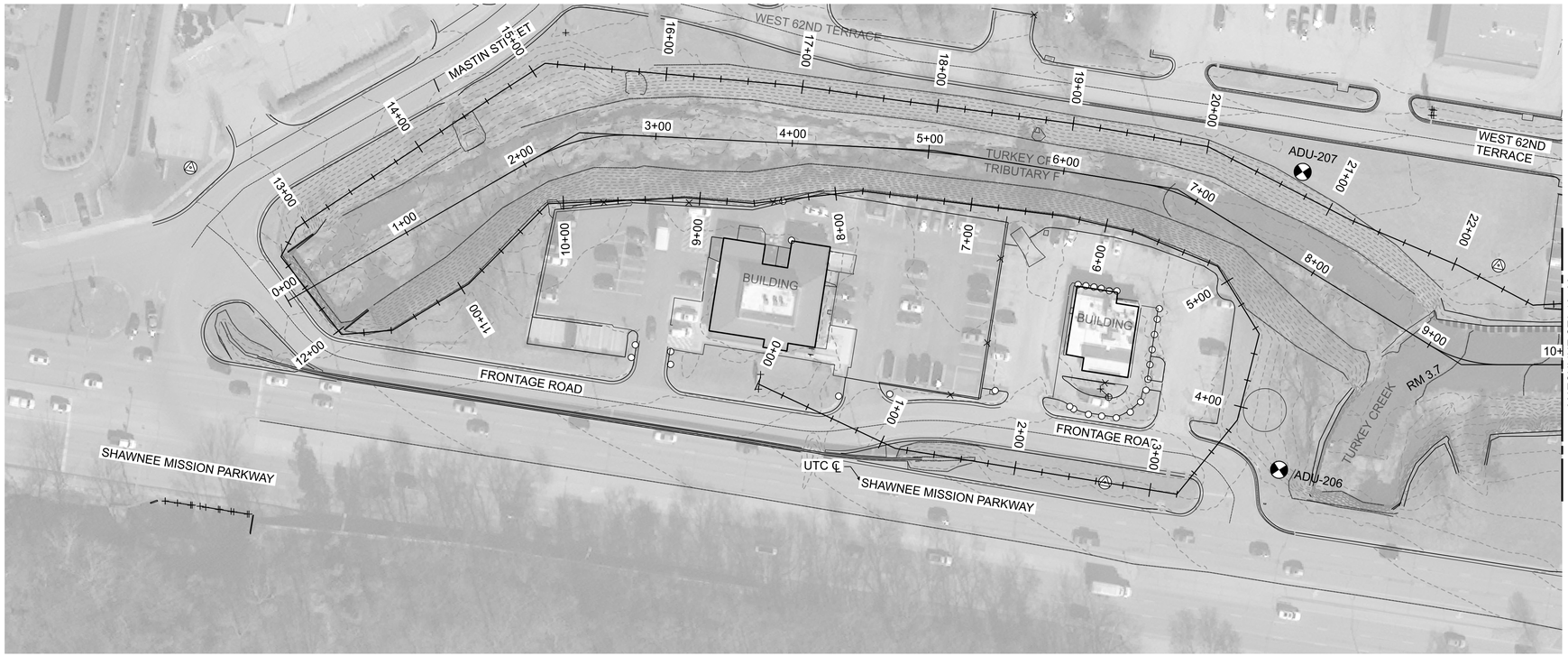
- The impervious blanket is assumed to be infinite on the landside. There is no evidence of a seepage block near the levee.

- Using permeability of 0.003 ft/sec for pervious stratum as typically used for Missouri and Kansas River levees with similar soils (see DM-1 for L-476, p. I-15. See Argentine Levee Raise and South Topeka Floodwall Modification.) This value is also within published typical range of values for coarse sand.

- Used initial permeability for impervious fill and impervious blanket in GeoStudio based on permeability ratio of 400 (7.5 e-06 ft/sec) then adjusted until head at levee toe matched blanket theory spreadsheet so results calibrated to match the spreadsheet.

Underseepage Blanket Theory Spreadsheet Parameters

Parameter	Assumption
Top of Levee Elevation (ft)	Assumed water loading to top of levee/floodwall per the profile from civil dated 13 August 2025
Riverside Ground Elevation (ft)	Used existing ground elevation at levee centerline for both riverside and landside elevations. The ground surface generally slopes uphill landward of the levee, so using ground elevation at centerline results in the largest head difference.
Landside Ground Elevation (ft)	
Landside Tailwater Elevation (ft)	Using landside ground elevation
River Blanket Bottom Elevation (ft)	Not differentiating from riverside to landside w limited info. Assuming no continuous pervious zones in upper fill materials. Taking as average bottom of CL or ML layer.
Land Blanket Bottom Elevation (ft)	
Top of Bedrock Elevation (ft)	Initially taken as SPT or auger refusal or beginning of rock core elevation for best case. Adjusted to elevation where factor of safety no longer changes significantly (as described above) for worst case.
Permeability Ratio (K_f/K_b)	Using 400, corresponding to Silty Lean Clay in KC Underseepage Guidance
Riverside Seepage Length (ft), L_1	Distance from riverside toe of levee or floodwall to the bank of Upper Turkey Creek. There is no significant exposed continuous pervious stratum for ending L1 before the bank of the creek.
Levee Seepage Length (ft), L_2	Width of levee or floodwall. Using 10-foot crest and 1V:3H slopes for levee from top of levee to existing grade. Using footing width for floodwalls.



D1 BORING LOCATIONS UTC STA 0+00 TO STA 10+00

SCALE: 1"=50'

0 50' 100'



A1 EXISTING CONDITIONS UTC STA 0+00 TO STA 10+00

SCALE: 1"=50'

0 50' 100'

LEGEND

- Fill (made ground)
- gCH – Gravelly Fat Clay
- CL – USCS Low Plasticity (Lean) Clay
- SH (WTH) – Shale, Weathered
- SH – Shale
- CLg – USCS Low Plasticity (Lean) Clay with Gravel
- CH – USCS High Plasticity (Fat) Clay
- Asphalt
- SCL – Sandy Lean Clay
- ML – USCS Silt
- LS – Limestone
- gCL – Gravelly Lean Clay
- CL-ML border line lean clay-silt
- GC – USCS Clayey Gravel
- GP-GC – USCS Poorly-graded Gravel with Clay
- SP – USCS Poorly-graded Sand
- SP-SM – USCS Poorly-graded Sand with Silt
- GM – USCS Silty Gravel
- LS (CHT) – Limestone – Charty
- CLs – USCS Low Plasticity (Lean) Clay with Sand

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-101.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

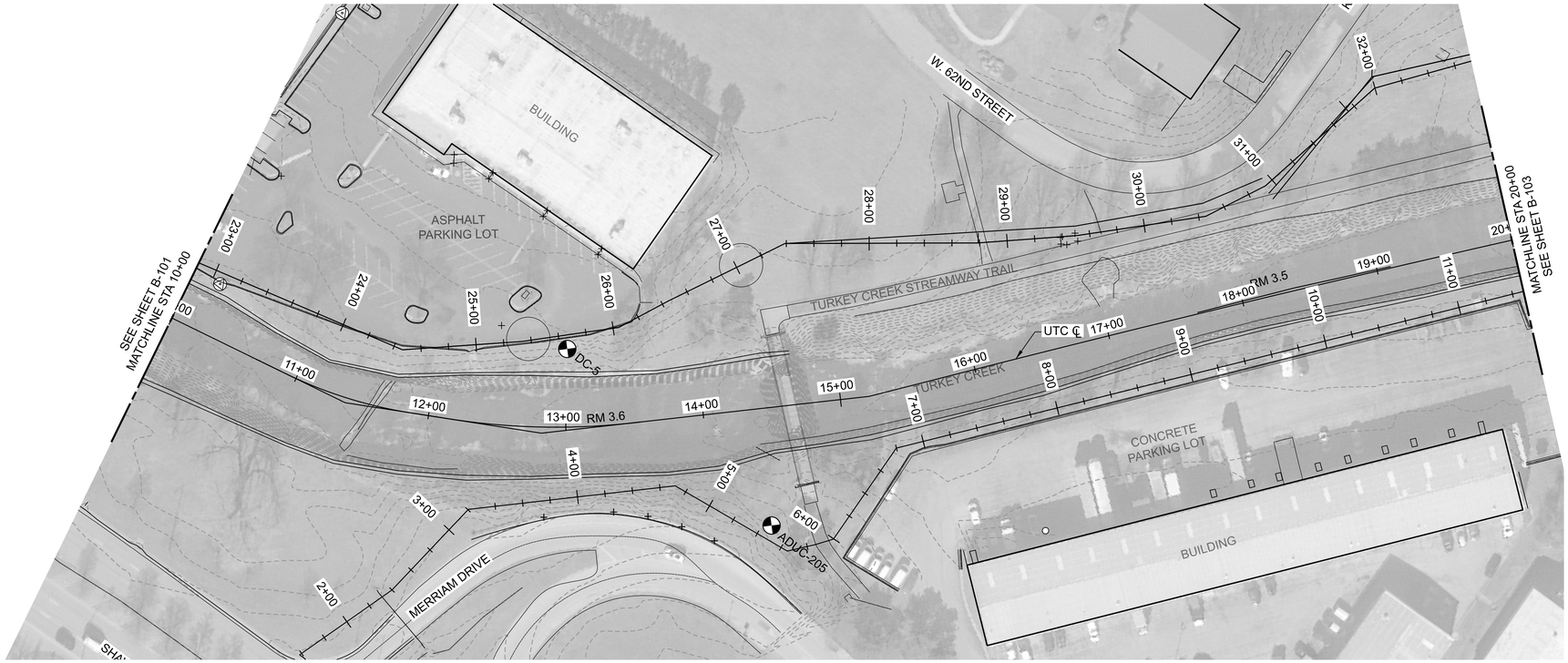
BORING PROFILE
STA 0+00 TO STA 10+00

SHEET ID
UTC
B-101

pw:\\PWINT-MPC.EIS.DG.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CW\TurkeyCr\505229_Upper_Turkey_Creek\6.0 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-101.dgn
11:36:16 AM 10/14/2025
PD:
DS:

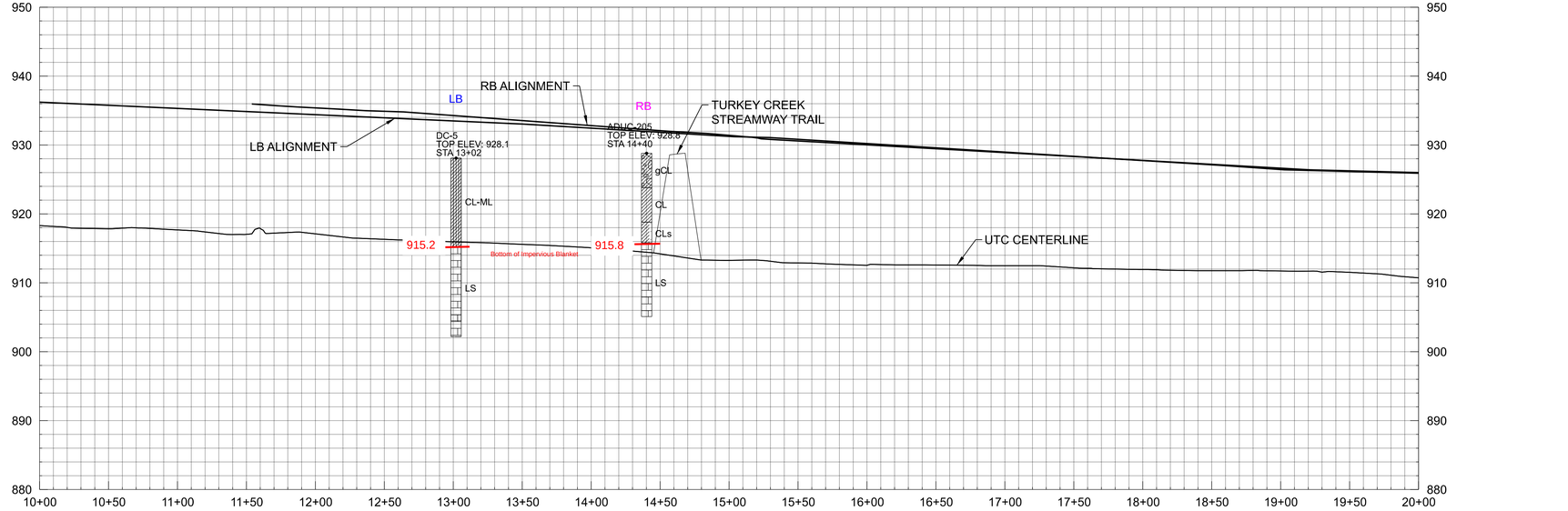
35%



D1 BORING LOCATIONS UTC STA 10+00 TO STA 20+00

SCALE: 1"=50'

0 50' 100'



A1 BORING PROFILE UTC STA 0+00 TO STA 10+00

SCALE: 1"=50'

0 50' 100'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-102.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

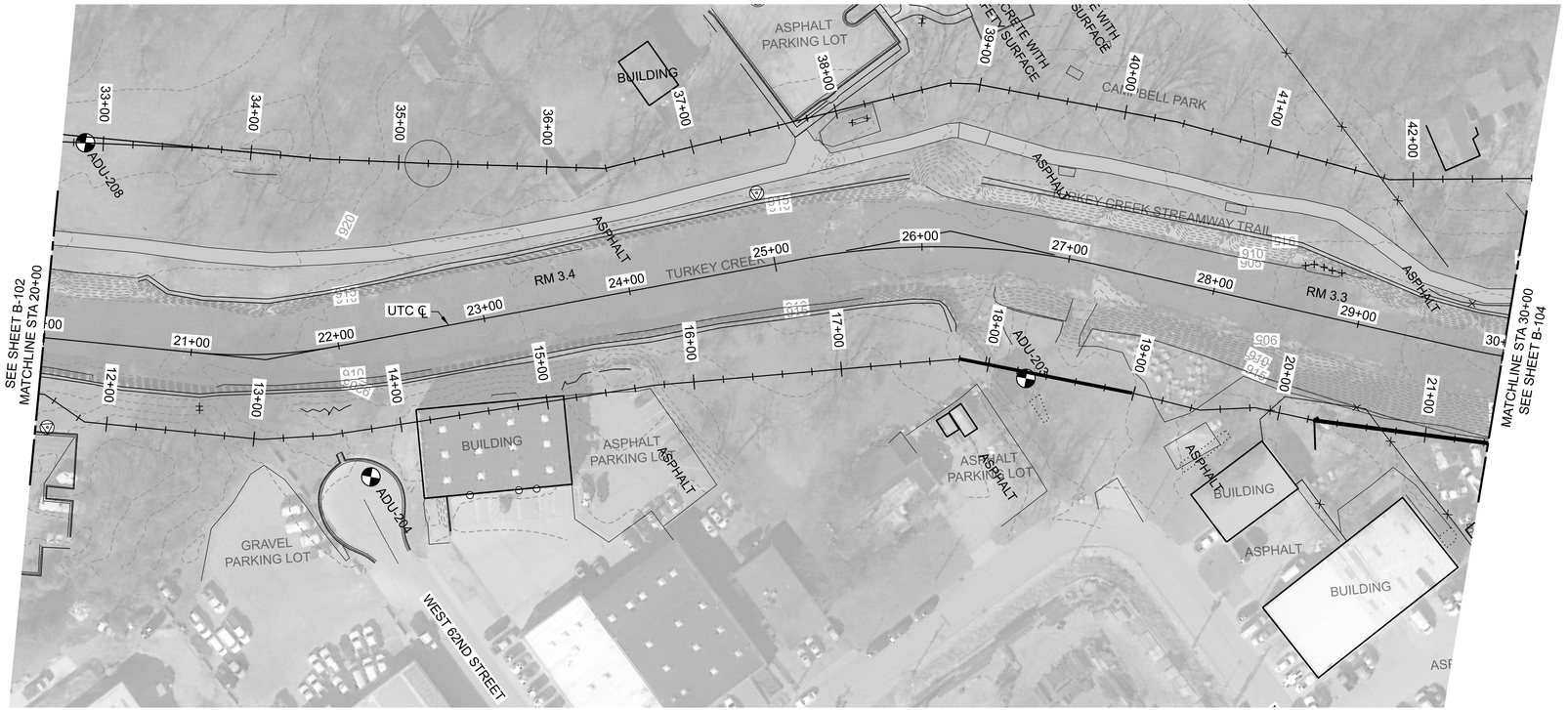
BORING PROFILE
STA 10+00 TO STA 20+00

SHEET ID
UTC
B-102

pw:\\PWINT-NPC.EIS.DG.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CW\TurkeyCr\505229_Upper_Turkey_Creek\6.0 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-102.dgn
11:24:49 AM 10/14/2025 PD: DS:

35%



D1 BORING LOCATIONS UTC STA 20+00 TO STA 30+00

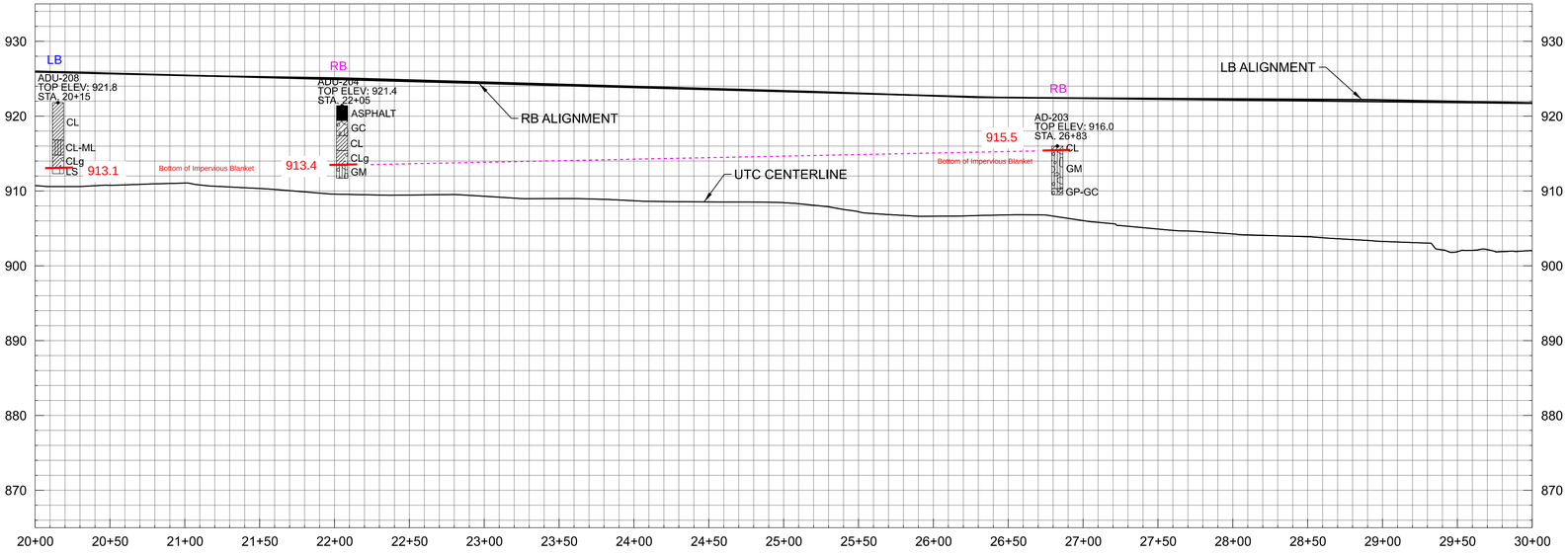
SCALE: 1"=50'

0 50' 100'

A1 BORING PROFILE UTC STA 20+00 TO STA 30+00

SCALE: 1"=50'

0 50' 100'



US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-103.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

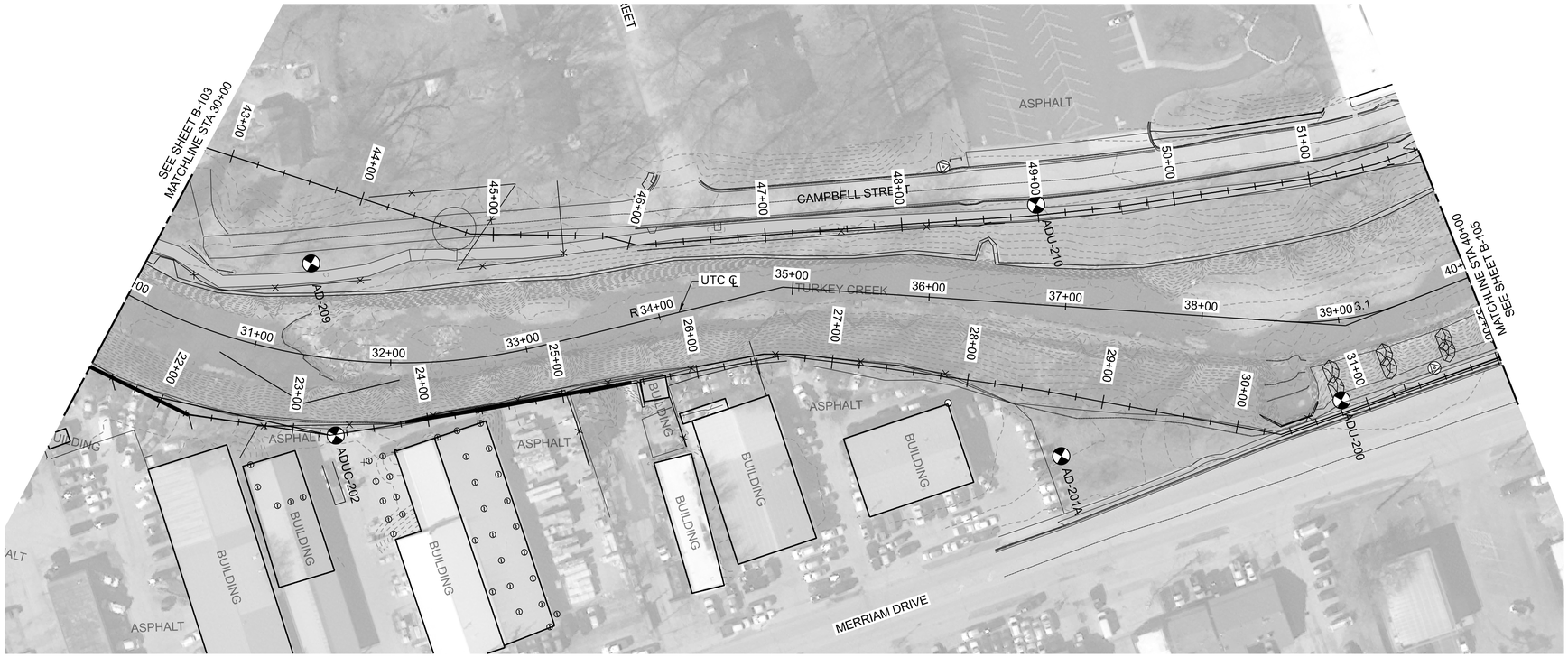
BORING PROFILE
STA 20+00 TO STA 30+00

SHEET ID
UTC
B-103

pw:\\PWINT-NPC.EIS.DG.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CW\TurkeyCr\505229_Upper_Turkey_Creek\6.0 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-103.dgn
11:25:28 AM 10/14/2025
PD:
DS:

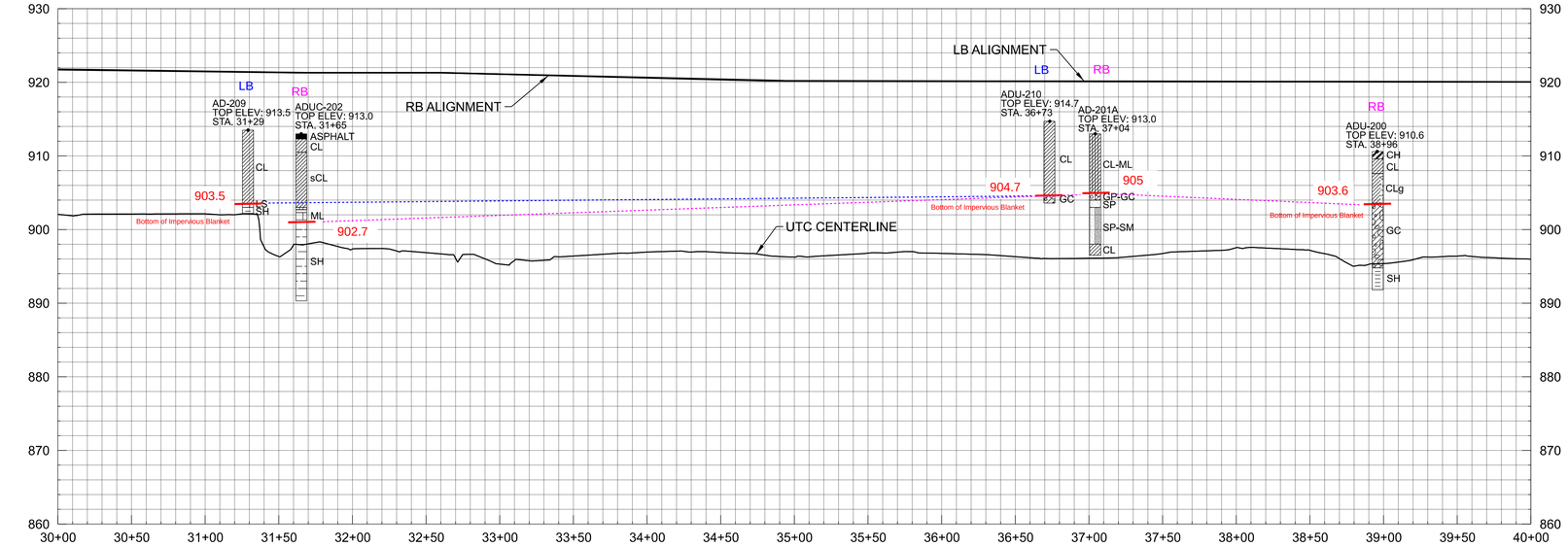
35%



D1 BORING LOCATIONS UTC STA 30+00 TO STA 40+00

SCALE: 1"=50'

0 50' 100'



A1 BORING PROFILE UTC STA 30+00 TO STA 40+00

SCALE: 1"=50'

0 50' 100'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-104.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

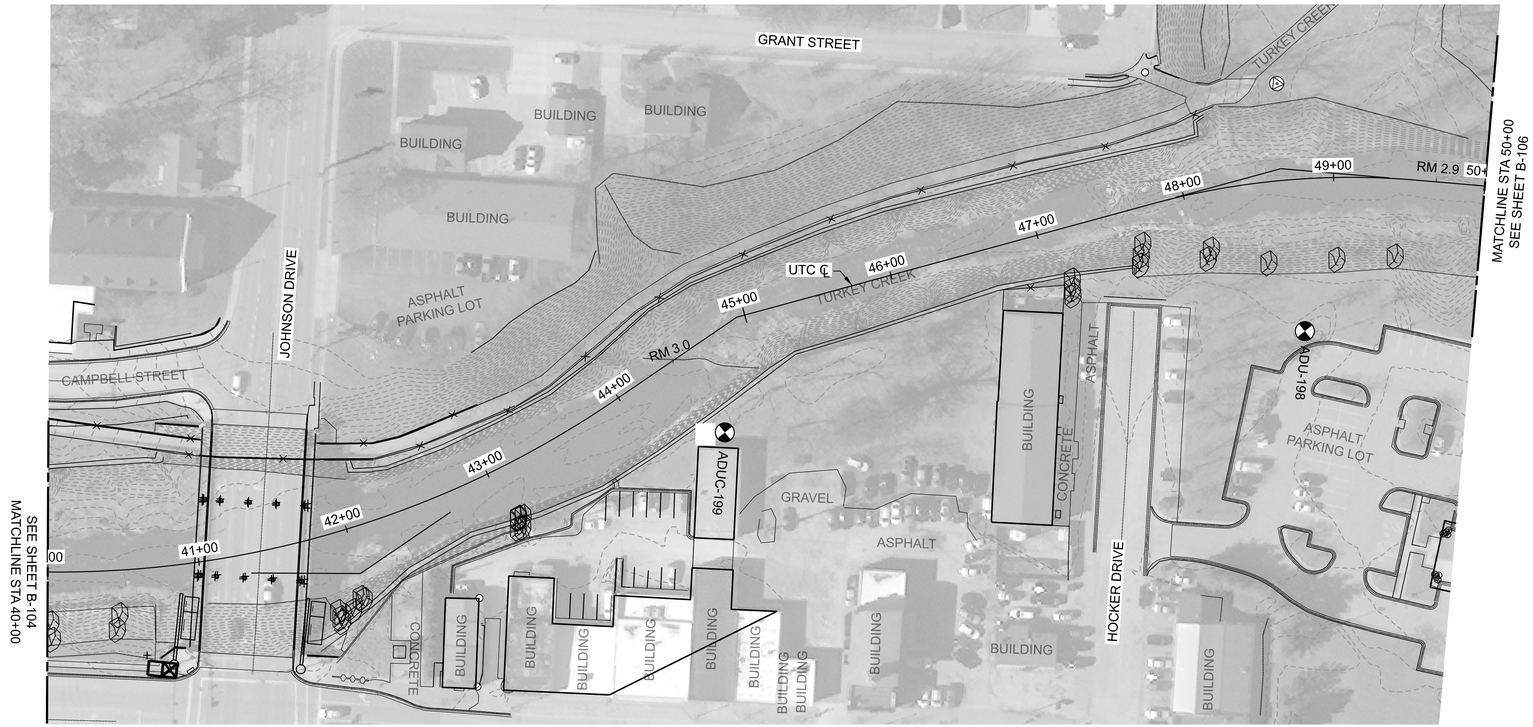
BORING PROFILE
STA 30+00 TO STA 40+00

SHEET ID
UTC
B-104

pw:\\PWINT-MPC.EIS.DG.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CW\TurkeyCr\505229_Upper_Turkey_Creek\6 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-104.dgn
11:25:54 AM 10/14/2025
PD:
DS:

35%

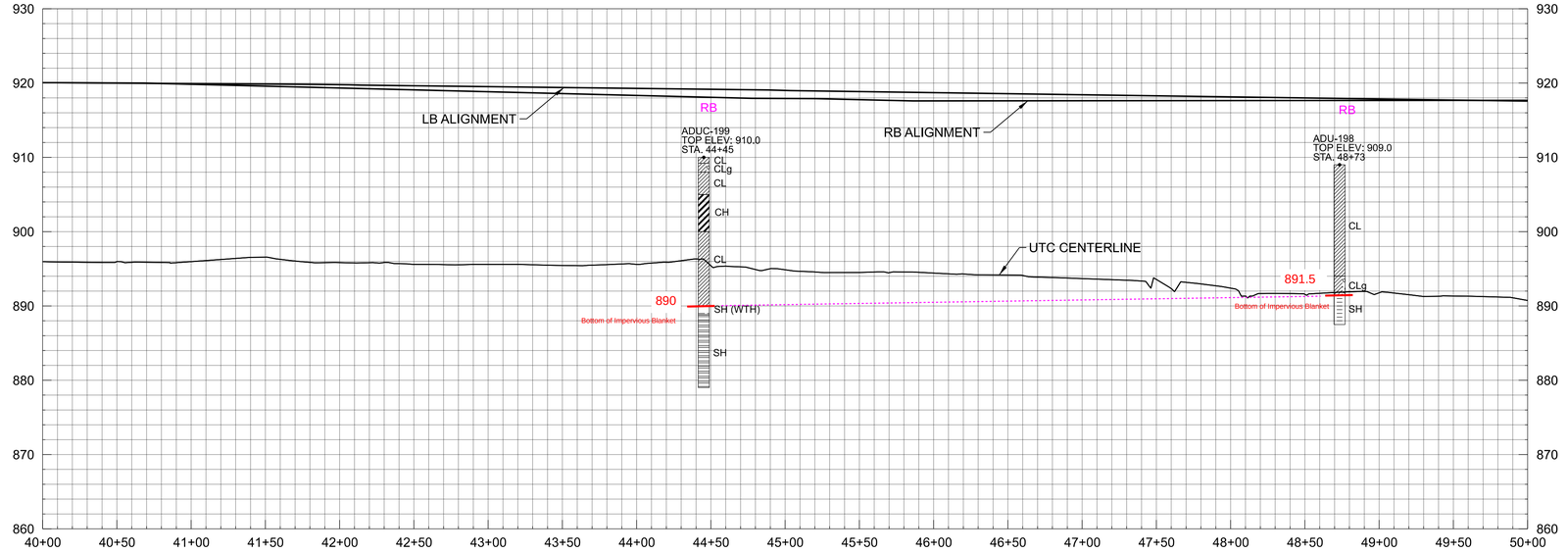


N

D1 BORING LOCATIONS UTC STA 40+00 TO STA 50+00

SCALE: 1"=50'

0 50' 100'



A1 EXISTING CONDITIONS UTC STA 40+00 TO STA 50+00

SCALE: 1"=50'

0 50' 100'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-105.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

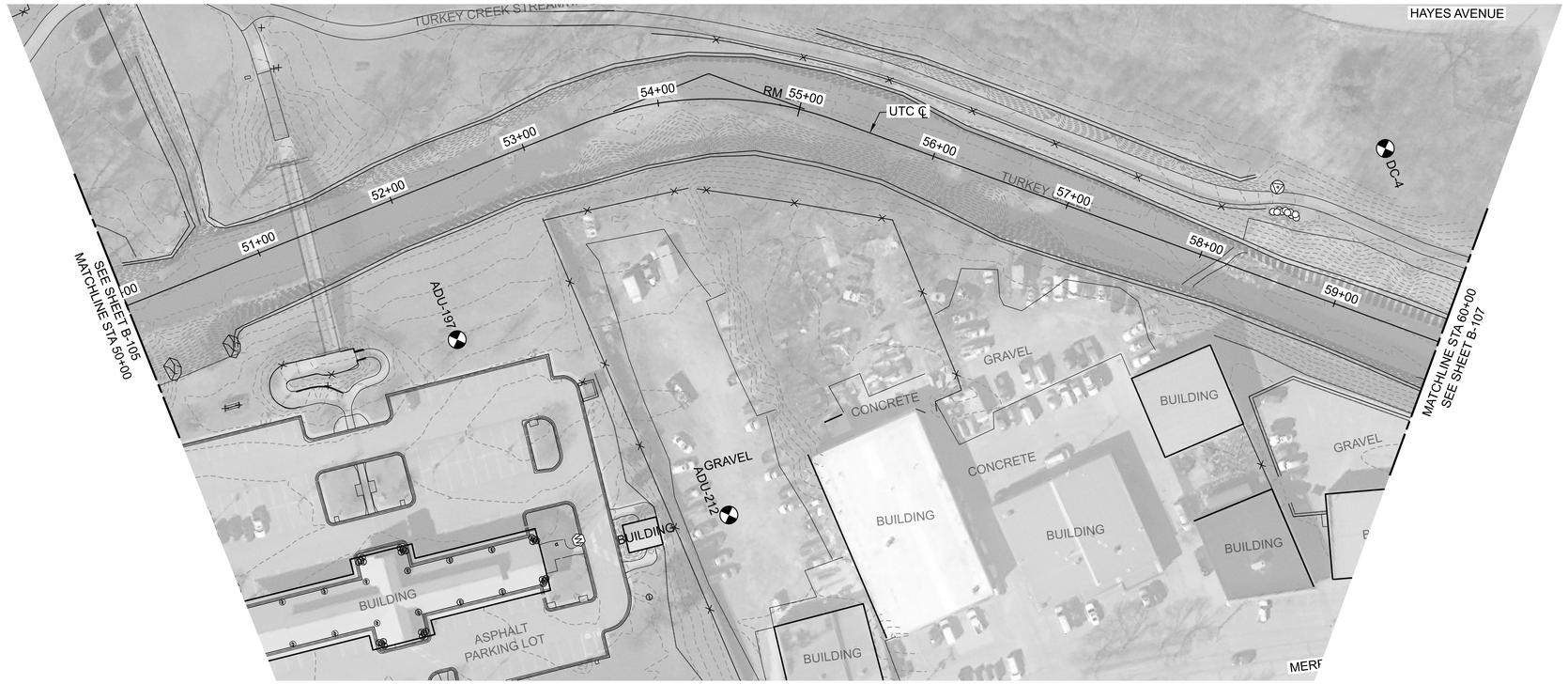
BORING PROFILE
STA 40+00 TO STA 50+00

SHEET ID
UTC
B-105

pw:\\PWINT-MPC.EIS.DS.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CWA\TurkeyCr\505229_Upper_Turkey_Creek\6.0 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-105.dgn
11:26:21 AM 10/14/2025 PD: DS:

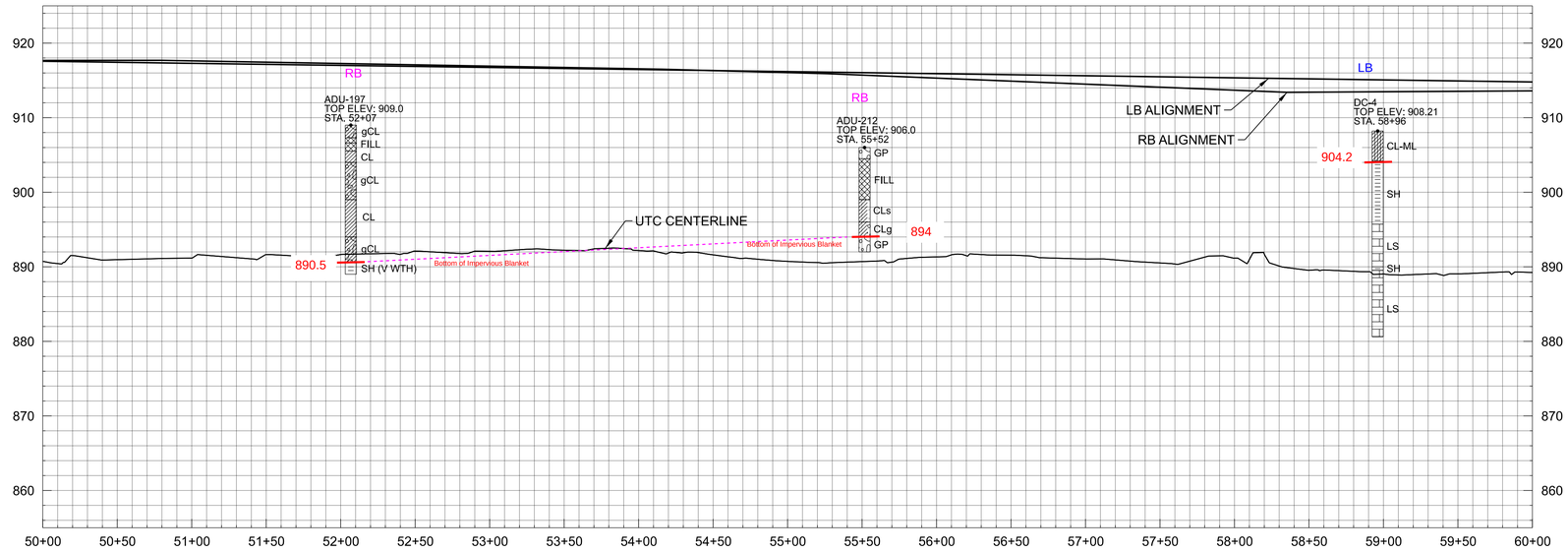
35%



D1 BORING LOCATIONS UTC STA 50+00 TO STA 60+00

SCALE: 1"=50'

0 50' 100'



A1 EXISTING CONDITIONS UTC STA 50+00 TO STA 60+00

SCALE: 1"=50'

0 50' 100'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-106.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

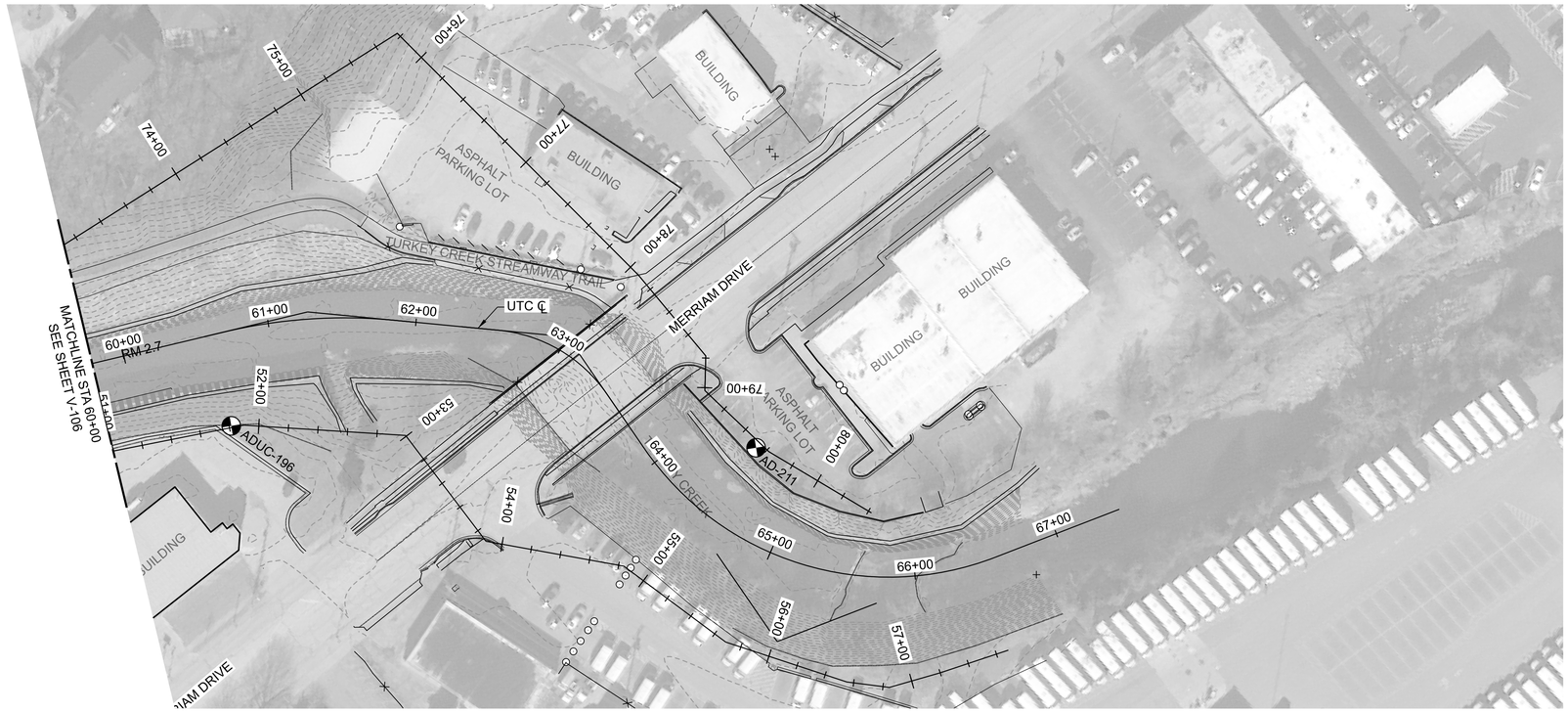
BORING PROFILE
STA 50+00 TO STA 60+00

SHEET ID
UTC
B-106

pw:\\PWINT-NPC.EIS.DG.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CW\TurkeyCr\505229_Upper_Turkey_Creek\6.0 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-106.dgn
11:26:47 AM 10/14/2025 PD: DS:

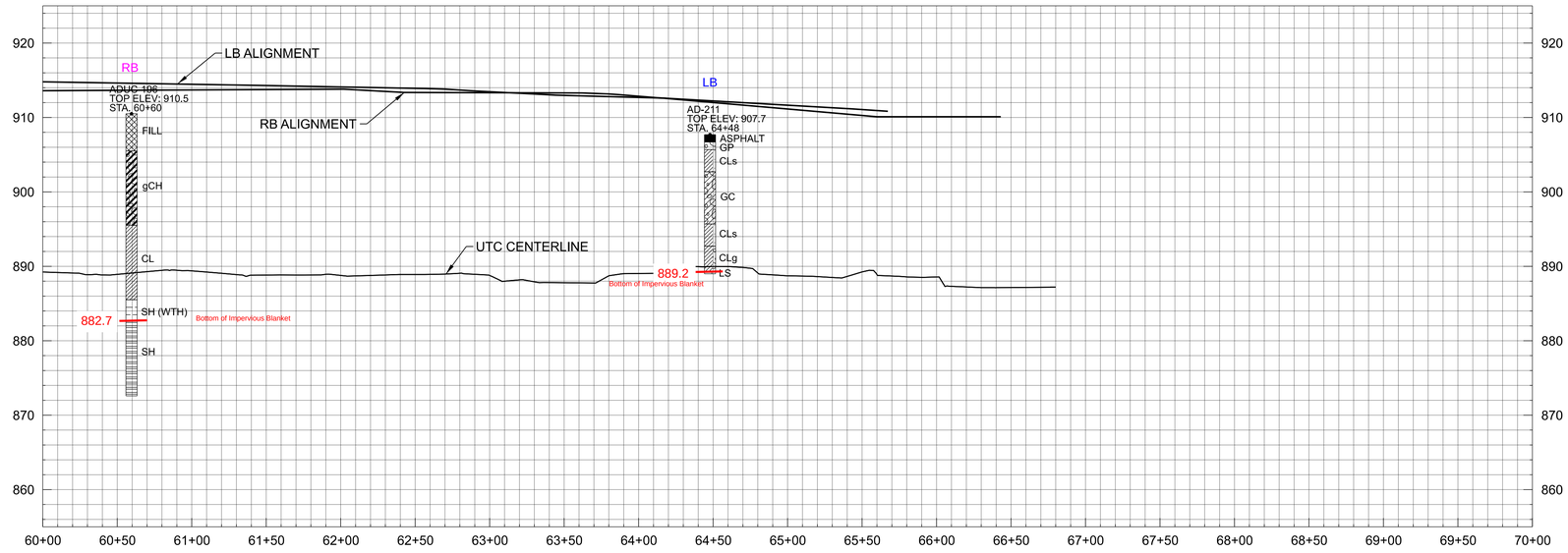
35%



D1 BORING LOCATIONS UTC STA 60+00 TO STA 67+45.54

SCALE: 1"=50'

0 50' 100'



A1 EXISTING CONDITIONS UTC STA 60+00 TO 67+45.54

SCALE: 1"=50'

0 50' 100'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY: M. VANLANDINGHAM
DRAWN BY: M. VANLANDINGHAM
CHECKED BY: D. NEWMAN
SUBMITTED BY: H. MILDENBERGER
SIZE: ANSI D
ISSUE DATE: OCTOBER 2024
SOLICITATION NO.: SOLICIT_NO
CONTRACT NO.:
UTC_B-107.dgn

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

DRAFT

MERRIAM, KANSAS
FLOOD RISK MANAGEMENT PROJECT
UPPER TURKEY CREEK
P2 - 505229, FY2023

BORING PROFILE
STA 60+00 TO END

SHEET ID
UTC
B-107

pw:\\PWINT-NPC.EIS.DG.USACE.ARMY.MIL:CENWK - Kansas City District\Documents\CW\TurkeyCr\505229_Upper_Turkey_Creek\6.0 Engineering-Design\6.1 Drawings\CAD_BIM\

UTC_B-107.dgn
11:27:16 AM 10/14/2025 PD: DS:

35%

Levee Unit Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

Case Analyzed - Water at Levee Top

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYSIS TAB AND ASSOCIATED PLOTS ONLY REFERENCE THE TOP ROW OF DATA.

Blanket Unit Weight = 115.0 pcf (saturated)

																									calibration				
	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Soil Boring	Land Blanket Bottom	Modified Top of Bedrock*	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)			Factor		Effective Seepage Length (ft)		Head at	Head at	Computed Hydraulic	Critical Hydraulic	Factor of Safety for	average horizontal gradient between entrance and levee toe
	Elevation	Elevation	Elevation	Elevation	Elevation	Reference	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	Riverside	Levee	Land Side			River Side	Land Side	Toe (elevation msl)	Toe (elevation msl)	Gradient	Gradient	Effective Stress Uplift / Blanket Rupture	
Station	(msl)	(msl)	(msl)	(msl)	(msl)			(msl)	H	k_f/k_{br}	k_f/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3	c_r	c_l	x_1	x_3	h_o	h_o	i_o	i_c	(i_c/i_o)	(i_h)
LB R1 F 13+20/UTC 0+50	939.7	928.6	928.6	928.6	921	ADU-207	921	910	11.1	400	400	7.6	7.6	7.6	11	8	9	infinity	0.005468	0.005468	7.99	182.87	10.16	938.76	1.34	0.84	0.63	0.65
LB R1 L 26+80/UTC 14+50	932.3	927.5	927.5	927.5	915	DC-5	915	864	4.8	400	400	12.5	12.5	12.5	51	55	40	infinity	0.001980	0.001980	54.78	504.98	4.04	931.54	0.32	0.84	2.61	0.05
LB R2 L 41+40/UTC 28+50	922.2	915.7	915.7	915.7	904.7	interp 208 & 209	904.7	863	6.5	400	400	11	11	11	41.7	45	50	infinity	0.002335	0.002335	44.84	428.35	5.32	921.02	0.48	0.84	1.74	0.07
LB R3 L 44+40/UTC 31+80	921.3	913.9	913.9	913.9	902.2	AD-209	902.2	871	7.4	400	400	11.7	11.7	11.7	31.2	40	54	infinity	0.002617	0.002617	39.85	382.12	5.94	919.84	0.51	0.84	1.66	0.08
LB R3 F 44+80/UTC 32+40	921.3	913.8	913.8	913.8	902.2	AD-209	902.2	881	7.5	400	400	11.6	11.6	11.6	21.2	45	14	infinity	0.003188	0.003188	44.69	313.64	6.32	920.12	0.54	0.84	1.55	0.13
LB R4 F 51+80/UTC 40+00	920.1	914.3	914.3	914.3	892	avg of RB in area	892	851	5.8	400	400	22.3	22.3	22.3	41	75	10	infinity	0.001654	0.001654	74.62	604.75	5.09	919.39	0.23	0.84	3.69	0.07
LB R5 L 76+60/UTC ~61+00	914.0	908.9	908.9	908.9	896	DC-4	896	835	5.1	400	400	12.9	12.9	12.9	61	100	40	infinity	0.001782	0.001782	98.95	561.03	4.09	912.99	0.32	0.84	2.66	0.04
LB R5 F 78+20/UTC 63+10	913.2	907.8	907.8	907.8	889.2	AD-211	889.2	874	5.4	400	400	18.6	18.6	18.6	15.2	35	10	infinity	0.002974	0.002974	34.87	336.29	4.76	912.56	0.26	0.84	3.29	0.12
RB R1 F 6+80/UTC 15+25	930.8	923.4	923.4	923.4	916	ADUC-205	916	905	7.4	400	400	7.4	7.4	7.4	11	15	9	infinity	0.005542	0.005542	14.97	180.44	6.53	929.93	0.88	0.84	0.95	0.31
RB R2 L 18+20/UTC 26+80	922.4	914.5	914.5	914.5	910	AD-203	910	879	7.9	400	400	4.5	4.5	4.5	31	30	58	infinity	0.004233	0.004233	29.84	236.22	5.76	920.26	1.28	0.84	0.66	0.09
RB R2 F 19+00/UTC 27+60	922.3	916	916	916	908.1	interp 203 & 202	908.1	887	6.3	400	400	7.9	7.9	7.9	21.1	50	10	infinity	0.003873	0.003873	49.38	258.22	5.12	921.12	0.65	0.84	1.30	0.11
RB R3 F 28+40/UTC 36+75	920.2	910.9	910.9	910.9	905	201A & LB 210	905	884	9.3	400	400	5.9	5.9	5.9	21	45	14	infinity	0.004492	0.004492	44.40	222.62	7.37	918.27	1.25	0.84	0.68	0.16
RB R4 F 33+80/UTC 41+75	919.5	909.4	909.4	909.4	891.3	interp 200 & 199	891.3	870	10.1	400	400	18.1	18.1	18.1	21.3	40	10	infinity	0.002546	0.002546	39.86	392.70	8.96	918.36	0.50	0.84	1.70	0.20
RB R4 L 41+40/UTC 49+50	917.7	907.6	907.6	907.6	891	avg 198 & 197	891	850	10.1	400	400	16.6	16.6	16.6	41	60	70	infinity	0.001917	0.001917	59.74	521.77	8.09	915.69	0.49	0.84	1.73	0.08
RB R5 F 52+60/UTC 61+60	913.8	904.3	904.3	904.3	884	ADUC-196	884	863	9.5	400	400	20.3	20.3	20.3	21	40	10	infinity	0.002422	0.002422	39.88	412.94	8.48	912.78	0.42	0.84	2.02	0.19

Regional Underseepage Criteria

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, NORTHWESTERN DIVISION
PO BOX 2870
PORTLAND OR 97208-2870

CENWD-RBT

MEMORANDUM FOR

CENWK-ED (David Mathews)
CENWO-ED (John Bertino)
CENWP-ED (Lance Helwig)
CENWS-ED (JoAnn Walls)
CENWW-ED (Brian Miller)

SUBJECT: HQUSACE Concurrence of the Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED) Underseepage Criteria

1. Reference memorandum, CECW-CE, dated 08 May 2015, subject same as above, (encl).

2. I am pleased to transmit the referenced memorandum that documents HQUSACE concurrence with the MRLS R-471-460 & L-455 PED Underseepage Criteria.

3. This criteria should be considered "Regional Criteria" to be applied to design of underseepage berms for other projects in NWD Districts until revisions to EM 1110-2-1913 (including Appendix C – Design of Underseepage Berms) are completed. The "Regional Criteria" intent has been worked through the regional geotechnical engineering community of practice and input from that group incorporated into the criteria document.

4. My Points of Contact for this memorandum are Dr. Yong Rhee, Senior Geotechnical Engineer, (503) 808-3831 or Yong.H.Rhee@usace.army.mil, or Mr. Scott Vollink, Levee Safety Program Manager and Acting Levee Safety Officer, (816) 389-3600, or Scott.D.Vollink@usace.army.mil.

KELLETT.JOSEPH.P.1231299269

Digitally signed by KELLETT.JOSEPH.P.1231299269
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=KELLETT.JOSEPH.P.1231299269
Date: 2015.05.11 08:43:54 -07'00'

Encl

JOSEPH P. KELLETT, P.E.
Chief, Regional Business Technical

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
441 G STREET, NW
WASHINGTON, DC 20314-1000

REPLY TO
ATTENTION OF

MAY 8 2015

CECW-CE

MEMORANDUM FOR Commander, Northwestern Division, U.S. Army Corps of Engineers, CENWD-DE, 1125 NW Couch St., Portland, OR 97209

SUBJECT: HQUSACE Concurrence of the Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED) Underseepage Criteria

1. References:

a. Memorandum, CENWD-RBT, dated 22 April 2015, subject: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED) Underseepage Criteria, (encl).

b. Memorandum, CENWK-ED-GD, 08 April 2015, subject: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria; Revised per USACE-HQ Comments.

2. This memorandum documents HQUSACE concurrence of the proposed underseepage criteria for use on the Missouri River Levee System (MRLS) R 471-460 & L-455 levee projects.

3. The point of contact for this action is Mr. Steven Kopecky, Deputy Chief, Northwestern Division-Regional Integration Team, who can be reached at (202) 761-4527.

Encl

JAMES C. DALTON, P.E., SES
Chief, Engineering and Construction
Directorate of Civil Works

CF:
CECW-CE (Dalton, Halpin, Conforti)
CECW-NWD/POD RIT (Kopecky)
CENWD-DE (Kem)
CENWD-RBT (Kellett, Vollink)
CENWK-ED (Mathews)

Printed on Recycled Paper

DEPARTMENT OF THE ARMY
KANSAS CITY DISTRICT, CORPS OF ENGINEERS
635 FEDERAL BUILDING
601 E. 12TH STREET
KANSAS CITY, MISSOURI 64106-2824

CENWK-ED-G

MEMORANDUM FOR Deputy Commander, Northwestern Division, ATTN; CENWD-RBT (Douglas Putman), 1125 NW Couch, Suite 500, Portland, Oregon 97209

SUBJECT: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED) Revised Underseepage Criteria; Document Transmittal for Major Subordinate Commander Approval

1. Purpose. This memorandum transmits for approval the third version, second revision, of MRLS R 471-460 & L-455 PED Underseepage Criteria and official responses to comments provided by USACE-HQ. Three final comments were received by CENWK via an email from CENWD dated 30 October 2014. The comments regarded an incorrect statement on consideration of bridge effects on the design water surface, clarification in Table 2 that the design water surface is the same as the NED plan, and an additional reference to historical practice in TM 3-424 (USACE 1956).

2. The following document is enclosed: Memorandum For Record, Subject: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria; Revised per USACE-HQ Comments; dated 30 October 2014.

3. The Engineering Division point-of-contact is Mr. Jacob W. Owen, 816-389-3314. An indication of concurrence or non-concurrence is requested.

11/5/14

Encl

DAVID L. MATHEWS, P.E.
Chief, Engineering Division

CF:
PM-PF (Lynn)

ENCL 1

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, NORTHWESTERN DIVISION
PO BOX 2870
PORTLAND OR 97208-2870

CENWD-RBT

MEMORANDUM THRU CECW-NWD, Steven Kopecky

FOR Commander, US Army Corps of Engineers (CECW-CE\Robert Bank), 441 G Street NW, Washington, DC 20314-1000

SUBJECT: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED) Underseepage Criteria

1. This memorandum transmits for HQUSACE review and endorsement the final version of the proposed underseepage criteria for use on the Missouri River Levee System (MRLS) R471-460 & L455 levee projects.

2. We have had significant vertical teaming on the development of this criteria between NWK, NWD, HQUSACE and RMC personnel. The criteria incorporates comments included in a resolution document received from HQUSACE (Conforti) via email on 09 March 2015. Revisions to the criteria based on the resolution document were concurred with by RMC (Shewbridge) via email on 19 March 2015.

3. My Point of Contact for this memorandum is Mr. Scott Vollink, Levee Safety Program Manager and Acting Levee Safety Officer, (816) 389-3600, or Scott.D.Vollink@usace.army.mil.

KELLETT.JOSEPH.P.1231299269

Digitally signed by KELLETT.JOSEPH.P.1231299269
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=KELLETT.JOSEPH.P.1231299269
Date: 2015.04.22 09:37:55 -07'00'

JOSEPH P. KELLETT, P.E.
Chief, Regional Business Technical

Encl

CF:
CENWK-ED

DEPARTMENT OF THE ARMY
KANSAS CITY DISTRICT, CORPS OF ENGINEERS
635 FEDERAL BUILDING
601 E. 12TH STREET
KANSAS CITY, MISSOURI 64106-2824

CENWK-ED-G

MEMORANDUM FOR Deputy Commander, Northwestern Division, ATTN; CENWD-RBT (Joseph Kellett), 1125 NW Couch, Suite 500, Portland, Oregon 97209

SUBJECT: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED) Revised Underseepage Criteria; Document Transmittal for Major Subordinate Commander Approval

1. Purpose. This memorandum transmits for approval the final version of MRLS R 471-460 & L-455 PED Underseepage Criteria, revised per comments provided by USACE-HQ.

2. The following document is enclosed: Memorandum For Record, Subject: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria; Revised per USACE-HQ Comments; dated 08 April 2015.

3. The Engineering Division point-of-contact is Mr. Jacob W. Owen, 816-389-3314.

4. An indication of concurrence or non-concurrence is requested.

4/17/15

Encl

DAVID L. MATHEWS, P.E.
Chief, Engineering Division

CF:
PM-PF (Lynn)

CENWK-ED-GD
SUBJECT: Missouri River Levee System (MRLS) L-385; Documentation of Project Deficiencies

K:\MissionProjects\civ\Levee Safety Program\2.0 Federal Levee Projects\2.45 R-471-460 KS MO\2. Correspondence\2.1 Official Correspondence\Draft-In Process\ 10_30_2014_ST J DIV TRNSM MEMO CRIT – revised per HQx2.doc

for OWEN
ED-G

MATHEWS
ED

CENWK-ED-GD 08 April 2015

MEMORANDUM FOR RECORD

SUBJECT: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria; Revised per USACE-HQ Comments

1. PURPOSE: The purpose of this memorandum is to document evolutions in levee underseepage criteria for the subject project and make recommendations for levee underseepage criteria for use in PED. This memorandum is considered an addendum to the 28 February 2011 Memorandum for Record, "Kansas City District Levee Underseepage Guidelines," that laid the groundwork for developing underseepage criteria for Kansas City District levees and floodwalls currently subject to study or PED. This memorandum is an update and replacement for the 14 February 2014 Memorandum for Record, "Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria," and has been revised per USACE-HQ comments transmitted via email to CENWD on 21 July 2014 and subsequently transmitted via email to CENWK on 22 July 2014. The comments received were in draft memorandum format and included information regarding the forthcoming revised EM 1110-2-1913 Design, Construction, and Evaluation of Levees. CENWK acknowledges the information provided in the memorandum, generally concurs with the comments, and has updated the proposed criteria accordingly. Additional comments were received via email to CENWK on 30 January 2015. CENWK generally concurs with the comments, and has updated the proposed criteria accordingly. Although this addendum specifically applies to the subject project, the principles applied within this memorandum should be considered during the development of underseepage criteria for future projects within the Kansas City District pending finalization of the updated EM 1110-2-1913.

2. REFERENCES:

a. U.S. Army Corps of Engineers, Kansas City District (NWK) (28 February 2011). *Kansas City District Levee Underseepage Guidelines*, Memorandum for Record

b. U.S. Army Corps of Engineers (30 April 2000). *Design and Construction of Levees*, Engineer Manual (EM) 1110-2-1913

c. U.S. Army Corps of Engineers (1 May 2005). *Design Guidance for Levee Underseepage*, Engineer Technical Letter (ETL) 1110-2-569

d. U.S. Army Corps of Engineers (31 August 2010). *USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation*, Engineer Circular (EC) 1110-2-6067

e. U.S. Army Corps of Engineers (June 2012). *Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines*. New Orleans District

f. Bligh, W.G. (1910). *Dams Barrages and Weirs on Porous Foundations, Engineering News*

g. Schmertmann, J.H. (2000). *The No-Filter Factor of Safety Against Piping Through Sands. American Society of Civil Engineers, GeoInstitute Special Technical Publications No. 111*, pp. 65-132

h. U.S. Army Engineer Waterways Experiment Station (WES) (1956). *Investigation of Underseepage and its Control, Lower Mississippi River Levees,* Technical Memorandum No. TM-3-424, 2 Vols, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS

i. U.S. Army Corps of Engineers (28 May 1999). *Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies*, Engineer Technical Letter (ETL) 1110-2-556

j. U.S. Army Corps of Engineers and U.S. Department of the Interior – Bureau of Reclamation (December 2013). *Best Practices in Dam and Levee Risk Analysis.*

3. BACKGROUND: Underseepage criteria used to develop recommendations in the 2006 feasibility report for the subject project applied the "Kansas City District" underseepage criteria commonly in use at the time of the study is included within Table 1:

TABLE 1—UNDERSEEPAGE CRITERIA—2006 FEASIBILITY REPORT

CONDITION	FLOOD STAGE	MINIMUM GRADIENT SAFETY FACTORS	
		LEVEE TOE	UNDERSEEPAGE BERM TOE
LEVEE WITHOUT AN UNDERSEEPAGE BERM	TOP OF LEVEE	1.1	BERM NOT REQUIRED
LEVEE WITH AN UNDERSEEPAGE BERM	TOP OF LEVEE MINUS 3'	1.5	1.1

Since the 2006 feasibility report has been completed underseepage criteria within USACE has been in flux. Current USACE guidelines are contained in Engineer Manual (EM) 1110-2-1913 *Design and Construction of Levees*, 30 April 2000. New underseepage guidelines superseding many of the recommendations in EM 1110-2-1913 were published in Engineer Technical Letter (ETL) 1110-2-569 *Design Guidance for Levee Underseepage*, 1 May 2005. ETL 1110-2-569 stated that it would be rescinded when EM 1110-2-1913 was revised. EM 1110-2-1913 has not been revised, but the ETL officially expired in May 2010. A draft version

of revised EM 1110-2-1913, Appendix C - *Design of Seepage Berms* was issued for USACE internal review in October 2006 but was never finalized. EC 1110-2-6067 *USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation*, 31 August 2010, states expired ETL 1110-2-569 should be used as a guide to evaluate levee structures and is the current USACE policy instead of the requirements in the current EM 1110-2-1913.

NWK, in the absence of published updated levee underseepage criteria, developed the 28 February 2011, CENWK-ED-GD Memorandum for Record, Subject: "Kansas City District Levee Underseepage Guidelines," that outlined the levee underseepage criteria shown in Table 2. NWK has used these criteria since signing of the memorandum.

Currently, a USACE team is working on a revision to EM 1110-2-1913 that is expected for release in the near future. NWK had representation at the EM revision kick off meeting, and has been communicating with the revision team on underseepage criteria over the last couple years in an attempt to ensure NWK underseepage criteria will be in line with future USACE recommendations.

TABLE 2—UNDERSEEPAGE CRITERIA—28 FEBRUARY 2011 NWK MEMORANDUM

CONDITION	MINIMUM GRADIENT SAFETY FACTORS (FLOOD STAGES ALL CASES WILL BE AT THE NED LEVEE CREST)	
	LEVEE TOE	UNDERSEEPAGE BERM TOE[1]
LEVEE WITHOUT AN UNDERSEEPAGE BERM	1.6	BERM NOT REQUIRED
LEVEE WITH AN UNDERSEEPAGE BERM	1.6	GRADUATED[2] SAFETY FACTORS OF 1.5 AT A DISTANCE OF 100' FROM THE LEVEE TOE TO 1.1 AT A DISTANCE OF 500' OR FURTHER FROM THE LEVEE TOE

1. The minimum underseepage berm width will be 100 feet.
2. The graduated factor of safety with berm width was an attempt to recognize that boil development at increasing distance from the levee toe is lower risk.

4. PED UNDERSEEPAGE CONTROL REVISITED: The NWK 2011 underseepage criteria were used to revise the feasibility study underseepage control for the subject project as part of PED. From the feasibility study to PED, average underseepage berm widths increased from 215 feet to 380 feet. The primary contributors to the underseepage berm width increases included: elevating the design flood stage to the levee crest, revisions to the Missouri River hydrology and hydraulic analysis completed during PED which raised the levee top profile approximately 0.5 feet, and the increased required berm toe safety factor

based on the NWK 2011 criteria. Berm thicknesses were less of an impact because a minimum berm thickness at the levee toe of 5 feet was applied during the feasibility study. Because of the large increases in the underseepage control requirements from feasibility to PED, and because these systems have performance histories during recent significant flood events, the Northwest Division (NWD) requested during a project briefing on 31 October 2013 that NWK revisit underseepage criteria for the subject project. NWD specifically requested the New Orleans District Hurricane and Storm Damage Risk Reduction (HSDRRS) underseepage criteria be considered. HSDRRS criteria are included within Table 3.

TABLE 3 —HSDRRS UNDERSEEPAGE CRITERIA

UNDERSEEPAGE BERM WIDTH DIVIDED BY LEVEE HEIGHT	MINIMUM GRADIENT SAFETY FACTORS			
	LEVEE TOE		UNDERSEEPAGE BERM TOE	
	DESIGN WATER SURFACE ELEVATION [1]	PROJECT GRADE WATER SURFACE ELEVATION [2]	DESIGN WATER SURFACE ELEVATION [1]	PROJECT GRADE WATER SURFACE ELEVATION [2]
LEVEE WITHOUT AN UNDERSEEPAGE BERM	1.6	1.3	BERM NOT REQUIRED	BERM NOT REQUIRED
4	1.6	1.3	1.5	1.2
8	1.6	1.3	1.3	1.1
12	1.6	1.3	1.1	1.0
16 OR MORE	1.6	1.3	1.0	1.0

1. Design water surface elevation is the project design water surface with a 90% confidence level.
2. Project grade is the top of levee and is higher than the design water surface elevation to account for wave action/runup minus the overbuild that is provided for primary consolidation

5. LEVEE PERFORMANCE DURING THE 2011 FLOOD EVENT: MRLS R 471-460 is 13.8 miles in length. Of the total length, 14.4% is earthen levee with no designed underseepage control, 4.8% is earthen levee with a combination of designed underseepage berm and pressure relief wells, and 80.8% is earthen levee with designed underseepage berms.

MRLS R 471-460 was loaded to 90% of the levee height during the 2011 flood event, which is approximately 1.5 feet of levee freeboard. During the flood event, 10% of the levee unit experienced sand boils. Of the portion of the unit that experienced sand boils, 13.7% occurred where there was no designed underseepage control and 86.3% occurred where there were designed underseepage berms. Approximately 17% of the sand boil areas were within low areas not accounted for during project design and landward ditches that eliminated

or thinned the blanket and increased the head between the river level and the landside ground surface. Generally, boils that occurred in reaches with underseepage berms occurred at berm toes.

Outside the MRLS 471-460 levee reaches exposed to sand boil activity, there were no functioning piezometers to determine underseepage pressures landward of the levee that could be used to assess performance or establish criteria for underseepage control design. However, observations of no underseepage distress can be used to assess reasonableness of underseepage analysis input parameters.

MRLS L-455 is 15.6 miles in length. Of the total length, 50.9% is earthen levee (except for about 100 feet of floodwall) with no designed underseepage control, 4.7% is earthen levee with a combination of a designed underseepage berm and pressure relief wells, and 44.4% is earthen levee with designed underseepage berms.

MRLS L-455 was loaded to 71% of the levee height during the 2011 flood event, which is approximately 4 feet of levee freeboard. During the flood event, 6.5% of the levee unit experienced sand boils. Of the portion of the unit that experienced sand boils, 7.0% occurred within reaches with no designed underseepage control, 3.7% occurred within reaches with combined underseepage berms and pressure relief wells, and 89.3% occurred within reaches with the designed underseepage berms. Approximately 54% of the observed sand boils occurred within landward ditches or a lake beach area that eliminated or thinned the blanket and increased the head because of a lower landside ground surface. Generally, boils that occurred in reaches with underseepage berms occurred at berm toes.

MRLS L-455 piezometers are located in the vicinity of a pressure relief well field and at locations downstream of the pressure relief well field. Generally, the piezometers indicated the levee performed as designed. However, in some areas the piezometric data and original blanket thickness did not correlate with the observed performance during the flood; i.e. boils were observed where they would not be expected based on the original design.

At locations of underseepage distress observed during the 2011 flood, underseepage berms were constructed as part of the post flood rehabilitation under PL 84-99.

The original design criteria for the subject project are shown in Table 4. Performance of the subject project during the 2011 flood is within reasonable expectations for a levee founded on alluvial deposits, loaded near the crest, with the shown underseepage criteria. Some boil activity would be expected at underseepage berm toes based on a minimum safety factor of less than 1.0.

TABLE 4—UNDERSEEPAGE CRITERIA—MRLS R 471-460 & L-455 ORIGINAL DESIGN CRITERIA		
CONDITION	MINIMUM GRADIENT SAFETY FACTORS (FLOOD STAGES ALL CASES WILL BE AT THE LEVEE CREST[1])	
	LEVEE TOE	UNDERSEEPAGE BERM TOE
LEVEE WITHOUT AN UNDERSEEPAGE BERM	1.5	BERM NOT REQUIRED
LEVEE WITH AN UNDERSEEPAGE BERM	1.5	0.8

[1]Levee Crest is the same as the Design Water Surface as defined in paragraph 6.c

6. UNDERSEEPAGE TELECONFERENCE: On 9 October 2013 a teleconference to discuss underseepage design and safety factors occurred with the following participants:

Dr. Scott Shewbridge, PE	Risk Management Center (RMC)
Jacob Owen, PE	Chief, ED-G
Glen Bellew, PE	Chief, ED-GD
Charles Detrick	ED-GD

Dr. Shewbridge was consulted because he is a contributor to the ongoing revisions of EM 1110-2-1913. The purpose of the teleconference was to get input on underseepage criteria being developed for the EM 1110-2-1913 revision. Below is a summary of the teleconference and the information discussed:

a. The history of the subject project (as described above in this memorandum) was presented to Dr. Shewbridge.

b. Dr. Shewbridge noted it appeared NWK was following an acceptable process and using criteria that are consistent with the current draft revisions of EM 1110-2-1913 for underseepage safety factors at the levee toe and using a design water surface at the levee crest.

c. In support of the flood stage placed at the levee crest for all analysis and design cases, Dr. Shewbridge stated the flood stage used in analysis and design should correspond to the stage that coincides with the National Economic Development (NED) Recommended Plan. This design stage will help ensure the levee performs consistent with the performance

assumed in economic analysis used to justify the project and meet performance expectations of stakeholders. This corresponds with the levee crest for the subject project. Dr. Shewbridge stated the levee crest elevation associated with estimated settlement and hydraulic superiority should not be included as part of the underseepage analysis.

d. New Orleans District is applying underseepage criteria to the HSDRRS that requires a gradient safety factor at the levee toe of 1.3 with water at the levee crest (project grade) and 1.6 with water at the "design water surface" for riverine systems. The difference between the design water surface and levee crest includes wave freeboard and overbuild for settlement. The case for a parallel application to the subject levee systems may be to apply a gradient safety factor of 1.3 with the flood stage at the levee crest and 1.6 with the flood stage at the "nominal" design flood event elevation (generally, approximately 3 feet below the levee crest for the subject project).

e. Dr. Shewbridge indicated deviating from recommended underseepage safety factors could be justified with such considerations as risk and uncertainty, consequences of failure, and life safety. It was noted the justification process may result in the safety factors remaining unchanged, increased, or decreased.

f. Because it is not practical to have high vertical gradient safety factors at underseepage berm toes, berm widths could be determined by addressing sand boil activity landward of the levee to ensure that seepage will not result in a progressive piping failure towards the levee embankment. This can be done by considering the Creep Ratio, or horizontal gradient, concept.

7. CREEP RATIO CONCEPT: The critical creep ratio concept is discussed in Bligh (1910) and Schmertmann (2000). Creep ratio is the ratio of seepage path length to total head. The seepage path length is the "length of creep." The total head is assumed to be dissipated along the length of creep. Horizontal gradient is the inverse of creep ratio. For the creep ratio concept, it is assumed there is an unfiltered seepage exit, such as would be the case through a sand boil. If an unfiltered exit is present and the creep ratio is less than the critical creep ratio (i.e., a seepage path too short for the applied head), backward erosion piping has a high likelihood of occurrence. The aforementioned references indicate critical creep ratios for silts and fine sands as shown in Table 5. The creep ratio concept can be applied to the base of a confining cohesive blanket overlying an artesian coarse grained aquifer, as is the typical levee foundation conditions in the lower Missouri River basin.

TABLE 5—CRITICAL CREEP RATIO BY REFERENCE		
REFERENCE	MATERIAL	HORIZONAL SEEPAGE CRITICAL GRADIENT, i_{h-c}
BLIGH (1910)	RIVER BEDS OF LIGHT SILT OR SAND OF WHICH 60% PASSESS THE 100-MESH SIEVE, AS THOSE OF THE NILE OR MISSISSIPPI RIVERS	1/18[1]
SCHMERTMANN (2000)[2]	SAND, $C_u = 1$	1/18
	SAND, $C_u = 2$	1/5
	SAND, $C_u = 3$	1/2.5
	SAND, $C_u = 4$	1/1.7

[1]value is considered "safe" according to literature and does not necessarily represent a limit state or critical value.
[2]See reference g. for data and equation relation between C_u and i_{h-c}, estimates are provided here for general reference.

8. UNDERSEEPAGE TOLERABLE RISK AND ECONOMICALLY OPTIMIZED CRITERIA: Subsequent to the 9 October teleconference, an exercise addressing Tolerable Risk Guidelines (obtained from the Best Practices in Dam and Levee Risk Analysis Manual, 2013) and construction cost was completed. The exercise compared expected performance and failure consequences to Tolerable Risk Guidelines and investigated underseepage control quantities and reliability for various underseepage criteria. The underseepage criteria assessed were 1) 2006 Original Feasibility, 2) 2011 NWK, and 3) HSDRRS.

Expected performance for the assessed underseepage criteria was evaluated based on annual initiation of underseepage failure (see section 9 for full failure description). A typical reach from the subject project was used for this exercise. The nominal hydraulic annual chance of exceedance (ACE) was multiplied by the probability of blanket rupture (failure initiation) for various water levels to determine the annual probability of failure initiation. The annual probability of underseepage failure initiation was determined using the methods described in ETL 1110-2-556. It should be understood that the annual probability of underseepage piping failure initiation will be higher than the annual probability of levee failure due to underseepage piping in nearly all situations. However, initiation is readily calculated with some degree of certainty and the other failure components are not (see section 9). Failure initiation at the levee toe is thought to be a reasonable, albeit high, estimate of the annual risk and a comparison can be made of the risk associated with various underseepage criteria for typical NWK levees. ETL 1110-2-556 considers initiation of underseepage piping failure and actual failure to be synonymous in the examples contained within. Life loss was

estimated using the Levee Screening Tool to determine the annualized life loss for levee failure based on various hydraulic loading scenarios. The annual probability of failure initiation was plotted versus the average incremental annual loss of life and compared to current Tolerable Risk Guidelines for dams. This information is shown in Figure 1. The 2011 NWK criteria plotted below Tolerable Risk Guidelines for failure initiation, and the other criteria plotted above Tolerable Risk Guidelines. This indicates a vertical gradient safety factor of 1.6 at the levee toe is a reasonable minimum for water at the levee crest, and a lower safety factor may not provide acceptable reliability for typical NWK levees. It is worth noting that riverine levees (Mississippi River and Tributary Levees – MR&T) within Mississippi Valley Division (MVD) where HSDRSS criteria is commonly used are significantly taller than NWK levees, and HSDRSS criteria may not have unacceptable risk when applied to MR&T levees and loading conditions. This is because the distance between the levee top and design water surface (typically 3 feet) may be 10% or less of the total levee height for a MR&T levee system. For a typical MR&T situation where the design water surface criteria control the underseepage design, there can only be a 10% higher load before overtopping. Consequently, the safety factor at the levee top will likely be higher than the allowable minimum and the risk of failure lower. For a typical NWK levee, that same 3 feet typically represents 15-30% of the total levee height. When the HSDRSS criteria is applied to NWK levees, the safety factor with water at the levee top is often near the allowable minimum.

Underseepage control quantities and probability of failure were compared for the assessed underseepage criteria on a typical levee section from the subject project to determine where diminishing returns are realized on construction cost vs. reliability. The annualized probability of failure was calculated as described in the preceding paragraph. The underseepage control quantities were based on designed underseepage berms for the various criteria. Figure 2 shows the results of the assessment: probability of failure versus construction quantities. The point on the curve where the slope begins to flatten would be equivalent to "optimized" criteria. From Figure 2, this location would be between the 2011 NWK and HSDRRS criteria; closer to HSDRRS than to 2011 NWK. This indicates that, only considering construction costs (quantities), the 2011 NWK criteria may be slightly "over" optimized. However, the 2011 NWK criteria do not appear to provide economically wasteful reliability when applied to the subject project.

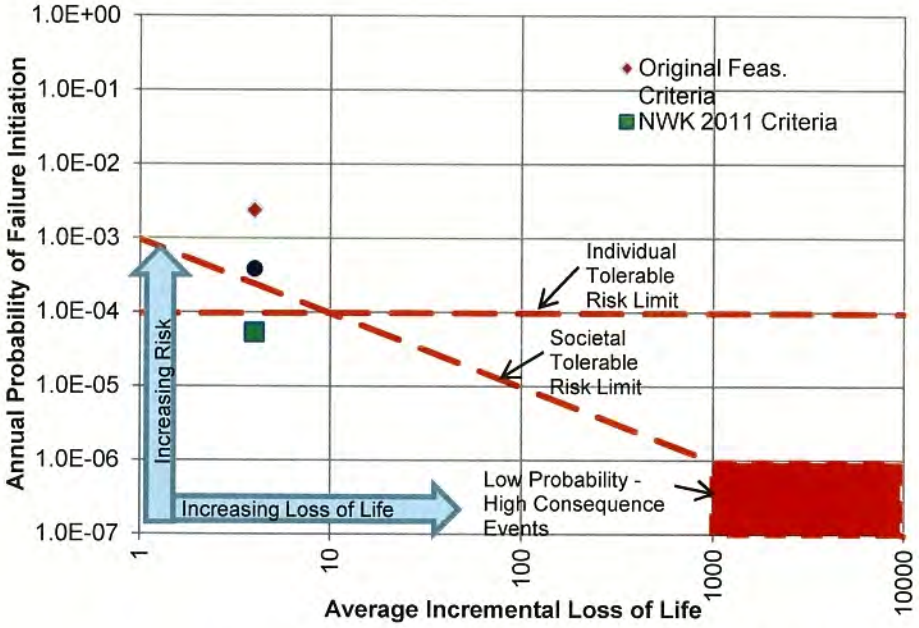


FIGURE 1—UNDERSEEPAGE CRITERIA TOLERABLE RISK GUIDELINES COMPARISON

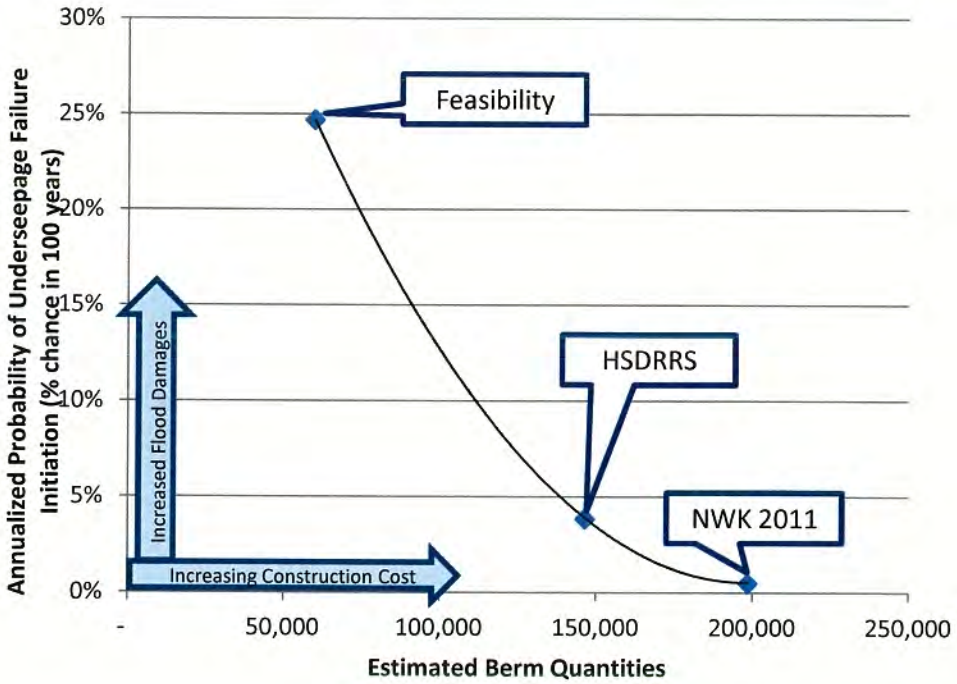


FIGURE 2—UNDERSEEPAGE CRITERIA QUANTITIES COMPARISON

9. LEVEE FAILURE BY UNDERSEEPAGE AND PIPING: The general understanding of levee failure by underseepage and piping when there is a fine grained blanket overlying a coarse grained aquifer is as follows:

a. Water elevations riverside of the levee increase, applying a differential head across the embankment and foundation, surcharging the coarse grained aquifer (artesian condition). (LOAD)

b. The artesian head in the aquifer is large enough to rupture/heave the fine grained blanket (critical vertical gradient is exceeded), creating a sand boil (unfiltered exit) in the fine grained blanket that allows sand to exit. (INITIATION)

c. The ratio of seepage path to total head (creep ratio) is such that movement of soil particles occurs through the unfiltered exit from along the interface between the fine grained blanket and coarse grained aquifer material. (PROGRESSION)

d. Movement of material continues along the blanket/aquifer interface and an erosion pipe is supported by the cohesive blanket material. (PROGRESSION)

e. There is no erosion pipe collapse or self healing to prevent progression of the pipe towards the levee riverside, and the pipe makes a direct connection with the river at the riverside toe. (CONTINUATION)

f. Successful intervention, consisting of initial observation of distress and subsequent flood fighting actions such as sand bag rings or emergency underseepage berms, does not occur to prevent failure. (INTERVENTION)

g. Gross enlargement of the pipe occurs, the levee collapses into the pipe resulting in a loss of freeboard, and the levee overtops and breaches. (BREACH)

The two critical conditions, early in the failure mechanism, are needed to result in levee failure. The first condition is the formation of an unfiltered exit by heave, or blow out, of the fine grained blanket. This condition is typically designed against by including a safety factor on vertical gradient through the blanket at the landside levee toe. The second condition is the presence of a horizontal gradient that will move foundation material along the seepage path and form an internal erosion pipe. Although, the requirement for a minimum total horizontal seepage length is a consideration in EM 1110-2-1901 Seepage Analysis and Control for Dams for the case of a pervious foundation with no downstream blanket (i.e., an unfiltered exit at the toe), and TM 3-424 (USACE 1956) discusses this concept for levee underseepage, horizontal gradients have not been a recent design consideration for levees except that long underseepage berms (> 400 ft) have been truncated because it was

recognized that long underseepage paths seldom lead to underseepage failure. However, underseepage berm widths could be determined with the expectation of sand boil development as long as there is reasonable assurance against foundation material movement. Furthermore, underseepage berms provide a longer seepage path for internal erosion and increase the likelihood of successful intervention. Intervention has been frequently used to prevent progression of seepage related failures in NWK throughout history. However, relying heavily on intervention during project design is not considered to be prudent given that intervention first requires observation. Early observation can be inhibited by landward vegetation, nightfall, and ponded water. Underseepage designs should provide a balance between expected levee performance, construction cost, and reasonable flood fight responses that are appropriate for the failure consequences.

10. NWK 2013 RECOMMENDED UNDERSEEPAGE CRITERIA: Sand boil development over a limited area landward of a levee should not be unexpected because of the nature of floodplain formation. Generally, the soil blanket is not uniform in alluvial deposits. By the nature of floodplain deposition, it is reasonable to expect variability, both in dimension and composition that cannot be fully defined with subsurface investigations. The empirical methods based on engineering principles adopted by the U.S. Army Corps of Engineers (current EM 1110-2-1913) acknowledge and account for blanket variability within the levee underseepage equations. The challenge is to discern from the available subsurface information the underseepage equation parameters that realistically represent the controlling floodplain features. It is neither practical nor economically feasible to account for each isolated change in the blanket, nor is it practical or economical to have large vertical gradient safety factors aimed at preventing any sand boil development landward of levees. Identifying isolated areas that require additional underseepage control beyond that established at the time of design and construction should occur as part of the project's performance via the "observational method" first described by Terzaghi and Peck. The emphasis of levee design should be to ensure stability near the levee toe while acknowledging manageable risk will permit reduced safety factors and limiting underseepage berm widths some distance away from the levee embankment. The recommended underseepage criteria in this memorandum were established with this philosophy.

As an addendum to Kansas City District Levee Underseepage Guidelines, 28 February 2011 Memorandum for Record, the recommended underseepage analysis and design safety factors for the MRLS R 471-460 and L-455 PED, and similar future projects, will consist of the following:

a. Flood Stage: The flood stage for all cases will be the elevation of the levee's physical crest elevation at the top of the impervious cap, excluding overbuild for settlement, transition structures, etc as discussed in paragraph 6.c.

b. Vertical Gradient and Creep Ratio: Both vertical gradients and creep ratios should be considered in design. Design objectives for levee toes are to have comparatively high vertical gradient safety factors such that an unfiltered exit is unlikely to occur at the levee toe. As such, the creep ratio is not a consideration for the levee toe. Conversely, design objectives for underseepage berm toes are to have comparatively low vertical gradient safety factors such that an unfiltered exit is more likely, or even expected, to occur. The creep ratio will be considered in determining minimum underseepage berm widths such that progression of the seepage condition beyond sand boil formation is not likely to occur. However, creep ratio should not be solely relied upon to determine reliable levee performance.

The critical vertical gradient is equal to the soil effective unit weight divided by the unit weight of water. The current guideline for $i_{h\text{-}c}$ is located in Schmertmann (2000). The fine sands located directly beneath the overlying fine grained blanket in the Missouri River floodplain have a typical C_u greater than 2.0. As such, initial designs without available laboratory grain size testing should assume a critical horizontal gradient of 1/5 until laboratory data can be obtained. The horizontal seepage length is defined as the distance from the effective seepage entrance to the first exit point for seepage at the landward toe of the underseepage berm, or $X_1 + L_2 + dxW$. The total head, H, should be measured from the water loading elevation to the landside ground surface.

The required vertical gradient safety factor ($FS_{i\text{-}v}$) and ratio of critical horizontal gradient ($i_{h\text{-}c}$) to in situ horizontal gradient (i_h) are included within Table 6. Underseepage berms should be designed to meet the vertical gradient safety factor with a generally recommended minimum berm width of 150 feet for new levee projects. A minimum berm width of 150 feet has been long standing practice in NWK to relocate the seepage concentration point away from the levee toe when berms are required. For modifications to levees with significant and meaningful performance data that increases the confidence in subsurface conditions, such as this project, reduction of the recommended minimum berm width could be considered. At any location with an $FSi_{\text{-}v}$ less than 1.6, $i_{h\text{-}c}/i_h$ and failure likelihood shall be considered in the risk assessment framework (Section 9) to ensure acceptable risk. A graphical representation of the underseepage criteria is included in Exhibit 1.

The underseepage stability considerations at features such as landward ditches, landward free exits such as oxbow lakes, and other features should be considered separately based on vertical seepage gradient safety factors and creep ratio concepts as appropriate to ensure satisfactory performance. These considerations and analyzed features should be fully documented in the design documentation for the project using similar criteria as recommended for underseepage berms. These considerations and analyzed features should be fully documented in the design documentation for the project using similar criteria as recommended for underseepage berms.

Additional discussion in support of the recommendations in Table 6 follows:

(1) There is considered sufficient reliability at the levee toe with the minimum required vertical seepage gradient safety factor to not require consideration of horizontal seepage gradient concepts. A vertical seepage gradient safety factor of 1.6 is approximately a gradient of 0.5. This was shown to be the lower bound of sand boil formation based on flood observations documented in WES TM 3-424, further indicating this is generally a reliable condition.

TABLE 6 —MRLS R 471-460 & L-455 PED UNDERSEEPAGE CRITERIA

CONDITION	MINIMUM GRADIENT REQUIREMENTS			
	LEVEE TOE		UNDERSEEPAGE BERM TOE[1]	
	FS_{i-v}	i_{h-c}/i_h	FS_{i-v}	i_{h-c}/i_h
LEVEE WITHOUT UNDERSEEPAGE BERM	1.6	N/A	N/A	N/A
LEVEE WITH UNDERSEEPAGE BERM	1.6[1]	N/A	1.0	1.6

1. The minimum underseepage berm thicknesses at the levee toe and the underseepage berm toe will be 3 feet and 1 foot, respectively. Thicknesses will be adjusted sufficiently to account for settlement and post construction activity. Berms shall be graded for positive drainage away from the levee

(2) The recommended minimum vertical seepage gradient safety factor requirement through the natural blanket at the underseepage berm toe (FS = 1.0) is reasonable and based on the NWK design and performance experience on the subject and other levee performance throughout the Lower Missouri River. Vertical seepage gradient safety factors of 1.0, and as low as 0.8, have been used at the toe of long underseepage berms successfully with the general acknowledgement that there is manageable risk as the distance increases landward of the levee. However, it should not be common engineering practice to

set general criteria guidelines below a "failure" limit state. As such, a value below 1.0 was not considered as a general design guide.

(3) Horizontal seepage concepts are not easily analyzed by analytical models. Horizontal gradients that may lead to continued internal erosion are generally based on observed field performance and laboratory experiments. However, horizontal gradient concepts are still considered important in establishing minimum underseepage berm widths based on total horizontal seepage length.

(5) Typically, levees within NWK are 10-20 feet in height. Minimum berm widths, thicknesses, and required safety factors at berm toes recommended in this memorandum are considered appropriate for NWK and may not apply to other USACE districts with generally larger levees or different foundation conditions.

(6) For future projects, a risk assessment to determine the adequacy of design criteria reliability for the consequences and uncertainty in subsurface conditions should be performed. The risk assessment should consider the full failure event tree. Intervention effectiveness should not be considered in establishing design critiera without careful consideration of future resources. If intervention effectiveness is largely used to justify a design variance, the anticipated intervention shall be well documented in the DDR and OMRR&R Manual.

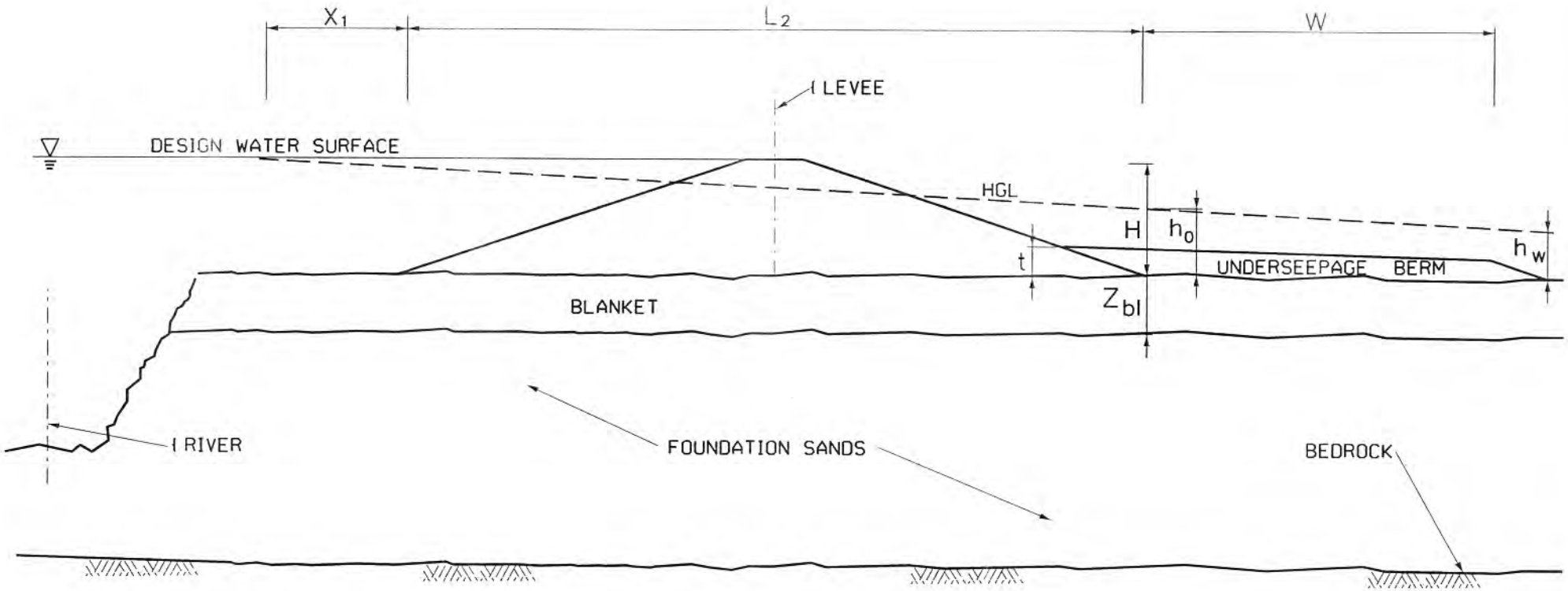
11. CONCLUSIONS: The recommended design approach and safety factors outlined within this memorandum will be applied to the MRLS R 471-460 & L-455 PED underseepage analysis and design. The design analysis, safety factors, and required underseepage control will be documented within the project's Engineering Documentation Report (EDR) and the Design Documentation Report (DDR). Additionally, the underseepage design recommendations for the subject project are recommended for initial designs of similar NWK projects in the future, pending consideration of a revised EM 1110-2-1913, at such time it is issued. The adequacy of all future new levee designs and levee modification designs should be subject to a full risk analysis based on expected life safety consequences, for all potential failure modes and water levels in accordance with the forthcoming update to EM 1110-2-1913 and finalization of EC 1110-2-6072 Levee Safety Program – Policy and Procedures. The use of typical failure initiation and progression concepts to develop criteria should better ensure proposed solutions are not over/under conservative or based on arbitrary limits. Any deviation from the recommended criteria should be well documented in project design documentation, based on sound engineering reasoning and should consider the general failure mode described in this memorandum. All performance expectations should be well documented in design, operations, and maintenance documents.

12. Point of contact for this memorandum is Glen Bellew (816-389-3553).

JACOB W. OWEN, P.E.
Chief, Geotechnical Branch

Encl

CF:
PM-PF (Lynn)



NOMENCLATURE:

H = NET HYDROSTATIC HEAD (RIVER SURFACE ELEVATION - TAILWATER ELEVATION)
L_2 = LEVEE BASE WIDTH
w = UNDERSEEPAGE BERM WIDTH
t = UNDERSEEPAGE BERM THICKNESS AT THE LEVEE TOE
z_{bl} = BLANKET THICKNESS (SILT, CLAY, OR COMBINATION; LESS PERVIOUS THAN FOUNDATION SANDS)
x_1 = DISTANCE FROM EFFECTIVE SEEPAGE ENTRY TO RIVERSIDE LEVEE TOE
HGL = HYDRAULIC GRADE LINE (AT MIDDEPTH OF FOUNDATION SANDS) BENEATH THE LEVEE
γ' = BLANKET EFFECTIVE UNIT WEIGHT
γ_w = WATER UNIT WEIGHT
h_o = EXCESS HYDROSTATIC HEAD AT THE LEVEE TOE
h_w = EXCESS HYDROSTATIC HEAD AT THE UNDERSEEPAGE BERM TOE
FS_{iv} = VERTICAL GRADIENT SAFETY FACTOR
i_v = VERTICAL GRADIENT
i_h = HORIZONTAL GRADIENT
$i_{h\text{-}c}$ = HORIZONTAL GRADIENT

NOTES:

MINIMUM UNDERSEEPAGE BERM THICKNESS AT LEVEE TOE IS 3 FEET AND AT BERM CROWN IS 1 FOOT
BERMS SHALL BE GRADED WITH POSITIVE DRAINAGE AWAY FROM THE LEVEE
MINIMUM BERM WIDTH FOR NEW LEVEES SHALL BE 150 FEET
EXCESS HEAD CALCULATIONS TO BE PERFORMED IN ACCORDANCE WITH EM 1110-2-1913

REQUIRED SAFETY FACTORS:

LEVEE TOE:

$$FS_{iv} = \frac{\gamma'/\gamma_w}{(h_o - t)/(z_{bl} + t)} \geq 1.6$$

UNDERSEEPAGE BERM TOE:

$$FS_{iv} = \frac{z_{bl} * \gamma'}{h_w * \gamma_w} \geq 1.0$$

$$i_h = \frac{H}{X1 + L2 + W}$$

$i_{h\text{-}c}$ BASED ON COEFFICIENT OF UNIFORMITY OF PIPING LAYER FROM SCHMERTMANN (2000)

$$\frac{i_{h\text{-}c}}{i_h} \geq 1.6$$

EXHIBIT 1

CENWK-ED-GD
SUBJECT: Missouri River Levee System (MRLS) R 471-460 & L-455 Pre-Construction, Engineering, and Design (PED); NWK 2013 Underseepage Criteria; Revised per USACE-HQ Comments

K:\MissionProjects\civ\Levee Safety Program\2.0 Federal Levee Projects\2.45 R-471-460 KS MO\2. Correspondence\2.1 Official Correspondence\Draft-In Process\ 081914 ST J PED SEEP SUMMIT MFR-revised per HQ

FOR
HENGGELER
ED-GD (LSPM)

FOR
BELLEW
ED-GD

OWEN
ED-G

District Underseepage Guidance

(Used for permeability ratios on page 4)

UNDERSEEPAGE

1. General. The need for underseepage control is determined from a study of the thickness and characteristics of the natural blanket material, the thickness and permeability of the foundation sands, the effective seepage entrance and exit, and the maximum differential head on the levee created by the design water surface. Seepage enters the foundation sands beneath the levee through the bed of the river and through the foreshore blanket. This results in a hydraulic gradient and an artesian head at the base of blanket as the water flows from the riverward to the landward side of the levee. The pressures that develop landward of the levee during sustained flood stages may, unless controlled, cause heaving or rupture of the blanket in thin blanket areas resulting in a concentration of seepage flow and the formation of sand boils and potential levee failure. Relief wells, underseepage berms or other measures should be considered to control the effect of underseepage pressures in the permeable sands below the impervious blanket.

2. Guidance. Where applicable the seepage modeling techniques addressed within the appendices of EM 1110-2-1913, “Design and Construction of Levees” may be used by the designer. The techniques addressed are commonly used to determine the hydrostatic pressures landward of the levee and may also be used in design of underseepage berms and in calculating pressures to be used in structural uplift.

3. District Specific Guidance. The Kansas City District method of estimating the underseepage gradient and the required factors of safety are slightly different than the method and factors of safety presented in the EM 1110-2-1913 and are based on the conclusions of a Corps of Engineers, Missouri River Division Conference, held in Omaha in 27 November, 1962. The underseepage analysis and design recommendation are based on experience on the Missouri River during the flood of 1952. The procedure addressed is applicable when the impervious blanket is considered infinite in extent landward of the levee toe.

4. Tools. Groundwater model programs may be used for calculating underseepage and gradients, but the designer should confirm the model is applicable to the condition being modeled. Another tool, which may be used when addressing seepage issues, is the traditional flow net.

5. Calculations of the underseepage factors of safety. The factors of safety to be used in underseepage analysis are as follows:

5.1. The gradient (piping) factor of safety is defined as:

$$F_s = \frac{\gamma_b * z}{h * \gamma_w} = \frac{i_c}{i}$$

where:

F_S = gradient factor of safety
γ_b = buoyant weight of the saturated soil
γ_w = unit weight of water
z = thickness of impervious blanket
h = excessive piezometer head above ground surface acting on base of blanket
i = actual gradient (h/z)
i_c = critical gradient when flotation occurs, determined as follows:

$$i_c = \frac{\gamma_b}{\gamma_w}$$

The following factors of safety should be used:

$F_S \geq 1.1$ at the toe of levee with the river at the top of levee when evaluating for need for underseepage control.

$F_S \geq 1.5$ at the toe of the levee with the river at the design water surface (generally 3 feet below top of levee for urban levees). Designer should confirm actual design with the U.S. Army Corps of Engineers, Local Protection Section when designing underseepage control.

$F_S \geq 1.1$ at the toe of the underseepage berm with the river at the design water surface when designing underseepage control, for urban levees.

$F_S \geq 0.8$ at the toe of the long underseepage berm with the river at the design water surface when designing underseepage control, for agricultural levees.

$F_S \geq 1.5$ at the bottom of excavations (permanent or temporary) for excavations landward of the flood control projects, considering the river at the design elevation.

$F_S \geq 1.5$ at the mid-point between pressure relief wells in the instances when the underseepage is controlled by relief wells, considering the river at the design elevation.

$F_S \geq 1.5$ at the levee toe if the underseepage is not controlled by an underseepage berm, considering the river at the design elevation (a factor of safety of 1.1 should be obtainable approximately 200 feet landward of the levee).

If the geotechnical information obtained for the proposed project indicates different field conditions than the initial conditions used for the levee design, a back calculation will be necessary to determine the actual underseepage factors of safety. Any proposed construction should not worsen the existing condition of the levee and should be calculated based on the actual factors of safety obtained in back calculations. Different field conditions can occur if the blanket was eroded and the thickness reduced after flood events, if the distance from the riverside toe of the levee was reduced due to erosion of the riverbank, or the material in the blanket has been eroded and sand deposits replaced the natural impervious blanket after flood events.

5.2. Uplift Factor of Safety. In the special condition where the landward blanket is continuous, impervious, and relatively thick (greater than 10 feet), the designer should also check for uplift

of the blanket in accordance with structural guidance for uplift of a structure or pipe. The gradient factor of safety should also be checked when checking for uplift.

5.3 Heave Factor of Safety in Excavations. At the bottom of temporary or permanent excavations, heave occurs under conditions of high hydraulic gradient, when the seepage force due to differential head equals the overlying total weight of soil. The factor of safety against heave ≥ 1.5 for river stage at the design elevation is required and is defined as follows.

$$F_s = \frac{\Sigma(\gamma_t * D_t)}{\gamma_w * (h + D_t)}$$

where:

$F_{S.}$ = Factor of safety against heave at the bottom of the excavation

γ_t = Total unit weight of each soil layer below the excavation and above the pervious

D_t = Thickness of each soil layer below the excavation and above the sand

γ_w = Unit weight of water

h = Differential total head or net head at the analyzed location considering the river at the design elevation, above the existing top of ground in the excavation

6. Special Conditions. There is a special condition of basins and other permanent excavations constructed within the critical area landward of the levee. The seepage should be addressed both from river to basin or excavation and from basin or excavation to river.

6.1 Seepage from a permanent excavation landward of the flood protection structure (basin or ditch) should be considered in design. The factors of safety should be determined at the riverside toe of a levee considering the water elevation in the basin or ditch at the normal operation and the basin or ditch without impervious features. For this conditions, the following factors of safety are acceptable:

a. Piping (gradient) factor of safety: $F_S = 1.5$

b. Uplift factor of safety : Fs = 1.5

c. Heave factor of safety: $F_S = 1.5$

7. Kansas City District Method of Analysis. Figure UNDERSEEPAGE ANALYSIS shows a simplified cross section, with nomenclature and design data to be used in the underseepage analysis shown in the subsequent pages. The landward ground surface is generally assumed to be saturated and therefore used as the tailwater elevation. The permeability ratio represents the horizontal permeability of the pervious aquifer divided by the vertical permeability of the blanket material. The recommended basis for the selection of permeability ratios for use in the underseepage study is as follows:

Blanket Material	Assumed Permeability Ratio, K_f/K_b
SM	100
ML	200 - 400
ML-CL	400
CL	400 - 600
CH	800 - 1000

K_f = the horizontal permeability of the pervious substratum;
Kb = the vertical permeability of the blanket above the pervious substratum;

7.1 Seepage Quantities. The natural seepage per unit length of levee is determined using the equation:

$$Q = \$ * K_f * H$$

where:

Q = seepage per unit length of levee;
$\$$ = shape factor depending on the dimensions of the generalized cross section of the levee and foundation, the characteristics of the top stratum both riverward and landward of the levee, and the pervious substratum;

$$\$ = \frac{D_f}{L_1 + L_2 + L_e}$$

L_1 = effective length of the riverside blanket
L_2 = base width of the assumed impervious fill and natural blanket beneath it
L_e = distance from the landside toe of the assumed impervious section to the effective seepage exit
H = the net head on the levee.

8. Recommended references:

The following references contain details regarding COE requirements for the underseepage analysis within the critical area of a flood control project constructed by the COE.

8.1. EM 1110-2-1913, Design and Construction of Levees includes the following:

a. Necessary subsurface investigations and laboratory testing to obtain the geotechnical properties necessary for underseepage analyses.
b. Seepage control, including foundation underseepage and seepage through embankments.
c. Appendix B "Mathematical Analysis of Underseepage and substratum Pressure" establishes the method of underseepage analysis, the assumptions and the determination of factors involved in the analysis for different cases.

8.2. EM-1110-2-1901 “Seepage Analysis and Control for Dams”, includes the following:

a. Necessary subsurface investigation for determination of soil parameters for groundwater analysis.
b. Determination of permeability of soils in the foundation.
c. Seepage principles, including seepage forces, hydraulic gradients and uplift pressure.
d. Seepage conditions in the presence of relief wells, including well design.
e. Seepage control in earth foundation adjacent to structures, and remedial seepage control.

NOMENCLATURE AND EXPLANATIONS

(For Figure 1 UNDERSEEPAGE)

H = Total head on levee

D_{bR} = Thickness of natural blanket riverward of levee

D_{bL} = Thickness of natural blanket landward of levee

D_f = Thickness of natural pervious foundation

k_{bR} = Permeability coefficient in a vertical direction of the natural blanket riverward of levee

k_{bL} = Permeability coefficient in a vertical direction of the natural blanket landward of levee

k_f = Permeability coefficient of the pervious foundation

L_R = Actual length of the of the riverside natural blanket

L_L = Actual length of the of the landside natural blanket

C = Effective length of the pervious foundation of infinite length covered by a natural impervious blanket

$$C = \sqrt{D_f * D_b * \left(k_f / k_b\right)}$$

k_f/k_b = Permeability ratio between horizontal permeability of the permeable foundation and vertical permeability of the impervious natural blanket

L_1 = Effective length of the riverside blanket

$$L1 = C_r * \frac{e^{\left(2 * L_R / C_R\right)} - 1}{e^{\left(2 * L_R / C_R\right)} + 1}$$

L_2 = Base width of the assumed impervious fill and natural blanket beneath it.

L_e = Distance from the landside toe of the assumed impervious section to the effective seepage exit

$$Le = C \quad when \quad L_L = \infty \text{ and Le } = L_L \text{ when } L_L < C$$

γ_s = Saturated unit weight of the natural blanket

γ_w = Unit weight of water

i_c = Critical or maximum upward seepage gradient through natural blanket

$$i_c = \frac{\gamma_s - \gamma_w}{\gamma_w}$$

h_o = Head above tailwater at landside levee toe

$$h_o = \frac{H * L_e}{L_1 + L_2 + L_e}$$

i_o = Upward seepage gradient through natural blanket

$$i_o = \frac{h_o}{D_{bL}}$$

F_{Si0} = Gradient factor of safety at the toe of the levee

$$F_{Sio} = \frac{i_c}{i_o}$$

W_T = Trial berm width
$W_{T/2}$ = One half width of the berm
$H_{(WT/2)}$ = Head above tailwater at midpoint of underseepage berm

$$H_{(W_{T/2})} = \frac{H * L_e}{\left(\left(L_1 + L_2 + L_e\right) + W_{T/2}\right)}$$

x = Location where uplift has to be calculated
h_x = Head above tailwater at location where uplift has to be determined

$$h_x = \frac{h_o}{e^{\left(x / x_L\right)}} \quad where \quad x_L = L_e$$

$H_{(WT)}$ = Head above tailwater at end of underseepage berm

$$H_{(WT)} = \frac{H_{(W_{T/2})}}{e^{(W_{T/2}/c)}}$$

$i_{(wT)}$ = Upward seepage gradient through natural blanket at the end of berm

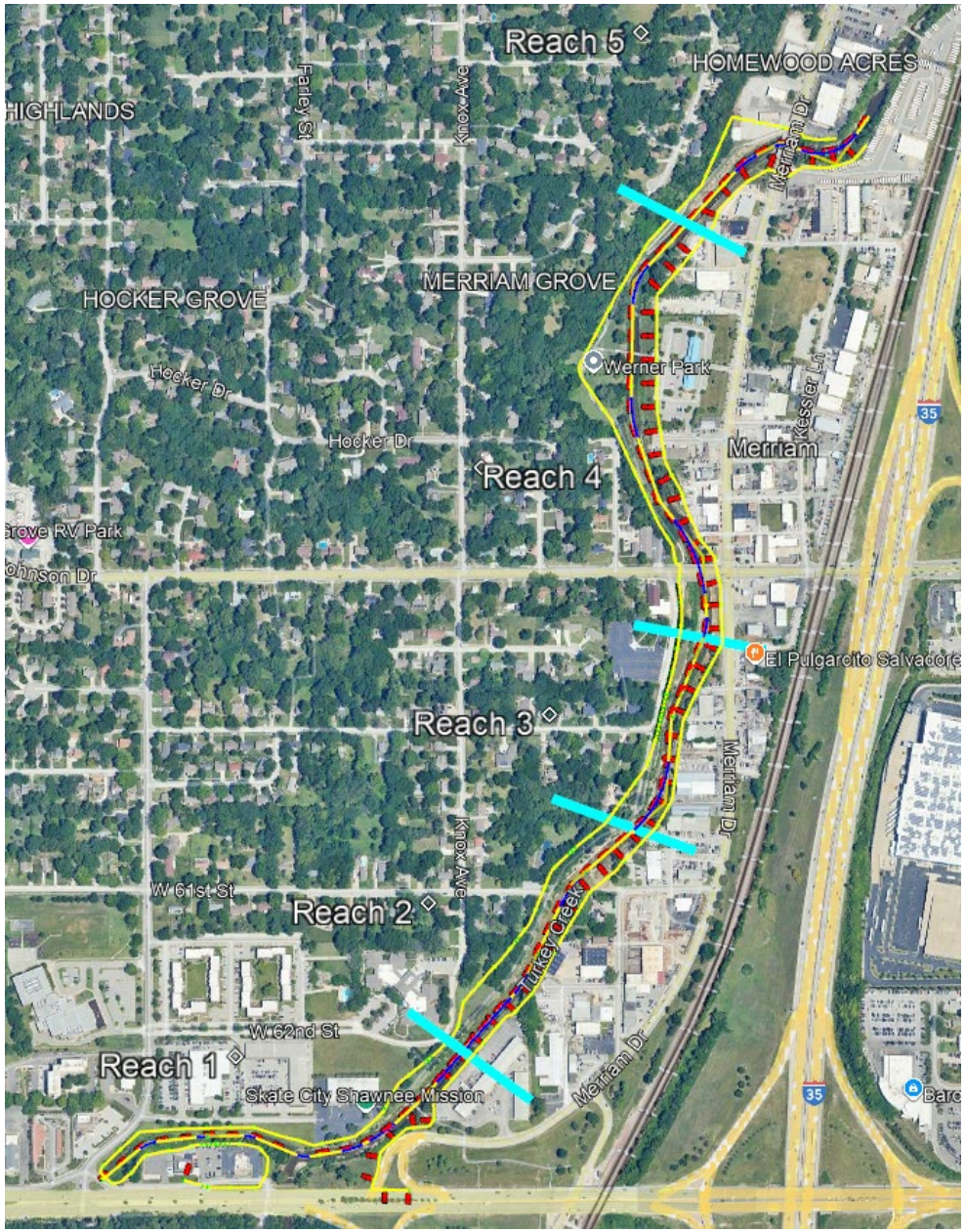
$$i_{(WT)} = \frac{H_{(WT)}}{D_{bL}}$$

$FSi_{(WT)}$ = Gradient Factor of safety at the end of the berm
L = Distance from effective seepage entry to effective exit.
M = Slope of hydraulic grade line beneath levee

APPENDIX B.2 – GLOBAL SLOPE STABILITY

Levee and Floodwall Slope Stability Calcs

General



The levee was divided into representative reaches for analysis based mainly on top of bedrock elevation. Soil stratigraphy and site geometry, including distance from the levee system alignment to the creek, were relatively consistent throughout the system while depth to bedrock increased from upstream to downstream (south to north). In general, the cross section with the tallest levee or floodwall was analyzed within a particular reach. Bottom of blanket elevations and top of bedrock elevations for design cross sections are based on the nearest borings or interpolated between borings.

Reach	UTC Centerline Stationing
1	0+00 to 17+60
2	17+60 to 29+50
3	29+50 to 38+60
4	38+60 to 57+80
5	57+80 to 66+00

General

Underseepage analysis performed for tallest levees and floodwalls within each reach based on August 2025 alignment.
Representative cross sections selected for stability analyses

Alignment is being refined but general levee/floodwall heights and soil conditions analyzed should be representative.

	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Land Blanket Bottom	Modified Top of Bedrock*	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	Riverside	Levee	Land Side
Station	(msl)	(msl)	(msl)	(msl)	(msl)		(msl)	H	k_f/k_{br}	k_f/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
LB R1 F 13+20/UTC 0+50	939.7	928.6	928.6	928.6	921	921	910	11.1	400	400	7.6	7.6	7.6	11	8	9	infinity
LB R1 L 26+80/UTC 14+50	932.3	927.5	927.5	927.5	915	915	864	4.8	400	400	12.5	12.5	12.5	51	55	40	infinity
LB R2 L 41+40/UTC 28+50	922.2	915.7	915.7	915.7	904.7	904.7	863	6.5	400	400	11	11	11	41.7	45	50	infinity
LB R3 L 44+40/UTC 31+80	921.3	913.9	913.9	913.9	902.2	902.2	871	7.4	400	400	11.7	11.7	11.7	31.2	40	54	infinity
LB R3 F 44+80/UTC 32+40	921.3	913.8	913.8	913.8	902.2	902.2	881	7.5	400	400	11.6	11.6	11.6	21.2	45	14	infinity
LB R4 F 51+80/UTC 40+00	920.1	914.3	914.3	914.3	892	892	851	5.8	400	400	22.3	22.3	22.3	41	75	10	infinity
LB R5 L 76+60/UTC ~61+00	914.0	908.9	908.9	908.9	896	896	835	5.1	400	400	12.9	12.9	12.9	61	100	40	infinity
LB R5 F 78+20/UTC 63+10	913.2	907.8	907.8	907.8	895	895	874	5.4	400	400	12.8	12.8	12.8	21	35	10	infinity
RB R1 F 6+80/UTC 15+25	930.8	923.4	923.4	923.4	916	916	905	7.4	400	400	7.4	7.4	7.4	11	15	9	infinity
RB R2 L 18+20/UTC 26+80	922.4	914.5	914.5	914.5	910	910	879	7.9	400	400	4.5	4.5	4.5	31	30	58	infinity
RB R2 F 19+00/UTC 27+60	922.3	916	916	916	908.1	908.1	887	6.3	400	400	7.9	7.9	7.9	21.1	50	10	infinity
RB R3 F 28+40/UTC 36+75	920.2	910.9	910.9	910.9	905	905	884	9.3	400	400	5.9	5.9	5.9	21	45	14	infinity
RB R4 F 33+80/UTC 41+75	919.5	909.4	909.4	909.4	891.3	891.3	870	10.1	400	400	18.1	18.1	18.1	21.3	40	10	infinity
RB R4 L 41+40/UTC 49+50	917.7	907.6	907.6	907.6	891	891	850	10.1	400	400	16.6	16.6	16.6	41	60	70	infinity
RB R5 F 52+60/UTC 61+60	913.8	904.3	904.3	904.3	884	884	863	9.5	400	400	20.3	20.3	20.3	21	40	10	infinity

highlighted cross sections
were analyzed for global slope stability

*top of bedrock modified as described in DDR

General

Underseepage analysis performed for tallest levees and floodwalls within each reach
Representative cross sections selected for stability analyses

Station	Head at Toe (ft) h_o	Head at Toe (elevation msl) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Effective Stress Uplift / Blanket Rupture (i_c/i_o)	average horizontal gradient between entrance and levee toe (i_h)
LB R1 F 13+20/UTC 0+50	10.16	938.76	1.34	0.84	0.63	0.65
LB R1 L 26+80/UTC 14+50	4.04	931.54	0.32	0.84	2.61	0.05
LB R2 L 41+40/UTC 28+50	5.32	921.02	0.48	0.84	1.74	0.07
LB R3 L 44+40/UTC 31+80	5.94	919.84	0.51	0.84	1.66	0.08
LB R3 F 44+80/UTC 32+40	6.32	920.12	0.54	0.84	1.55	0.13
LB R4 F 51+80/UTC 40+00	5.09	919.39	0.23	0.84	3.69	0.07
LB R5 L 76+60/UTC ~61+00	4.09	912.99	0.32	0.84	2.66	0.04
LB R5 F 78+20/UTC 63+10	4.75	912.55	0.37	0.84	2.27	0.12
RB R1 F 6+80/UTC 15+25	6.53	929.93	0.88	0.84	0.95	0.31
RB R2 L 18+20/UTC 26+80	5.76	920.26	1.28	0.84	0.66	0.09
RB R2 F 19+00/UTC 27+60	5.12	921.12	0.65	0.84	1.30	0.11
RB R3 F 28+40/UTC 36+75	7.37	918.27	1.25	0.84	0.68	0.16
RB R4 F 33+80/UTC 41+75	8.96	918.36	0.50	0.84	1.70	0.20
RB R4 L 41+40/UTC 49+50	8.09	915.69	0.49	0.84	1.73	0.08
RB R5 F 52+60/UTC 61+60	8.48	912.78	0.42	0.84	2.02	0.19

General

Existing Topography – Used Google Earth kmz file with topographic survey information prepared by EDG-C.

Levee Cross Section – Used 10-foot crest width and 1V:3H side slopes

Floodwall Cross Section – Used design footing widths

Levee Subsurface Profile – Developed representative soil profiles for each reach. As described in DDR text, developed a pervious zone thickness for a worst-case scenario, lowering top of the relatively impervious bedrock layer in the model to account for seepage through gravelly clays or joints or voids in bedrock.

SEEP/W Boundary Conditions – A water total head equal to the top of new levee is applied on the riverward slope, and a water total head equal to the landside grade is applied to the landside edge of the model. A potential seepage face is applied to the landward levee slope with water rate equal to 0 ft^3/sec, and a no-flow boundary is placed at bedrock with a water rate equal to 0 ft^3/sec.

Riverward Extent of Model – The model was extended at least halfway into Upper Turkey Creek.

Landward Extent of Model – Taken to 500 to 2000 feet landward of the new levee centerline. Extending the boundary farther appears to have no influence on the calculation results.

Hydraulic Conductivity of Sand – Used 0.003 ft/sec. This value is typically used for NWK levees such as the Argentine Levee Raise on the Kansas River (see Argentine DDR), the floodwall modification project at South Topeka on the Kansas River (see South Topeka DDR), and for design of Missouri River levee L-476 (see DM-1, p. I-15). This is also within the typical range used in Das Principles of Geotechnical Engineering, Table 5.1, p. 164, 1998.

Underseepage Calibration – A permeability ratio of K_f/K_b of 400 typical of ML-CL materials was used in the underseepage blanket theory spreadsheet. This equates to hydraulic conductivity of 7.5e-06 ft/sec for impervious blanket (K_b), using 0.003 ft/sec for sand (K_f). This was the starting point for saturated horizontal hydraulic conductivity. Each individual cross section was then calibrated to the blanket theory spreadsheet by adjusting the saturated horizontal hydraulic conductivity for impervious blanket and impervious fill in SEEP/W until the water total head at the landside toe matched the blanket theory spreadsheet.

Soil Strengths – See discussion in DDR

Levees

Performed calcs for four representative levee sections:
LB R1 L 26+80/UTC 14+50
LB R3 L 44+40/UTC 31+80
RB R2 L 18+20/UTC 26+80
RB RW L 41+40/UTC 49+50

Detailed example provided for LB R3 L 44+40/UTC 31+80
Major inputs and all results provided for other sections
(See Google Earth file, underseepage spreadsheet, and
GeoStudio files in project file for additional information)



L2 (levee width) = levee height from profile spreadsheet x 3 for 1V:3H slope x 2 for each side + 10' crest width

L2 = 7.387'*3*2 +10 = 54.322, say **54'**

Assume half of L2 on each side of centerline for showing landside and riverside levee toes

L1 (riverside seepage length) is distance from riverside levee toe to Turkey Creek (assuming no significant continuous pervious zone exposed on riverside of levee) = **40'**

Use survey topo and line distances in Google Earth to develop cross section for stability calcs

LB R3 L 44+40/UTC 31+80 Underseepage Calcs

UTC

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYSIS TAB AND ASSOCIATED PLOT

Case Analyzed - Water at Levee Top

Datum (NAVD88)

Add a Row

Blanket Unit Weight = 115.0 pcf (saturated)

	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Land Blanket Bottom	Top of Bedrock	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	River Side	Levee	Land Side
Station	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	H	k_f/k_{br}	k_f/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
LB R3 L 44+40/UTC 31+80	921.3	913.9	913.9	913.9	902.2	902.2	871	7.4	400	400	11.7	11.7	11.7	31.2	40	54	infinity

	Head at Toe (elevation msl)	Computed Hydraulic Gradient	Critical Hydraulic Gradient	Factor of Safety for Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe
Station	h_o	i_o	i_c	(i_c/i_o)	(i_h)
LB R3 L 44+40/UTC 31+80	919.84	0.51	0.84	1.66	0.08

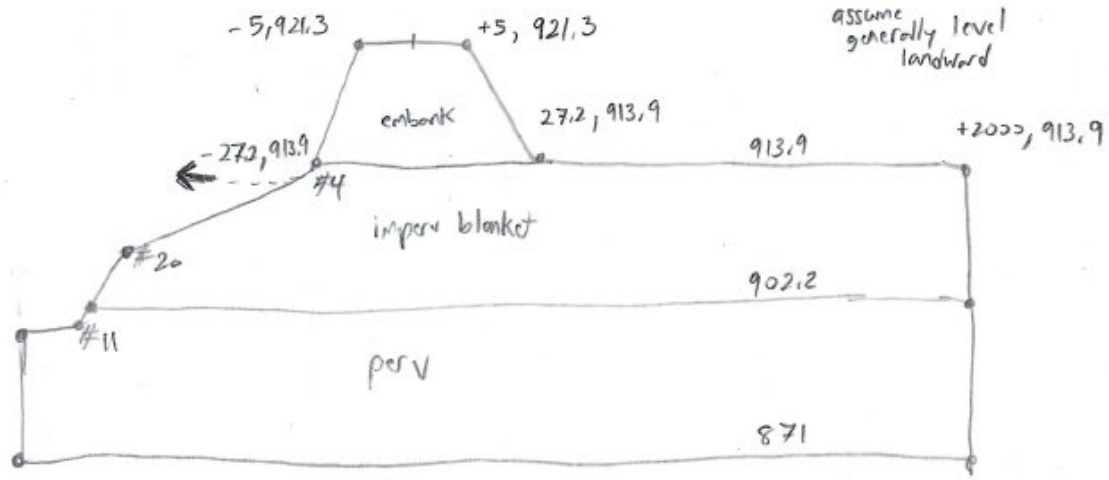
Use to calibrate SEEP/W

UTC LB R3 L 44+40 / UTC 31+80

Developing cross section

Google Earth ruler distance from RS toe	Gold 27.2' x GroStudio coord	EL
15'	-42.2'	913
30'	-57.2'	912
31'	-58.2	911
32'	-59.2'	910
37'	-64.2'	905
38'	-65.2	904
39'	-66.2'	903
39.5'	-66.7'	902.2
40'	-67.2'	901
41'	-68.2'	900
42'	-69.2'	899

consistent slope



channel width ~60'
go 30' w/ model

72' * -99.2' 899

point #20 -57.2 912
point #4 -27.2 913.9
current 30' offset from main bank slope top

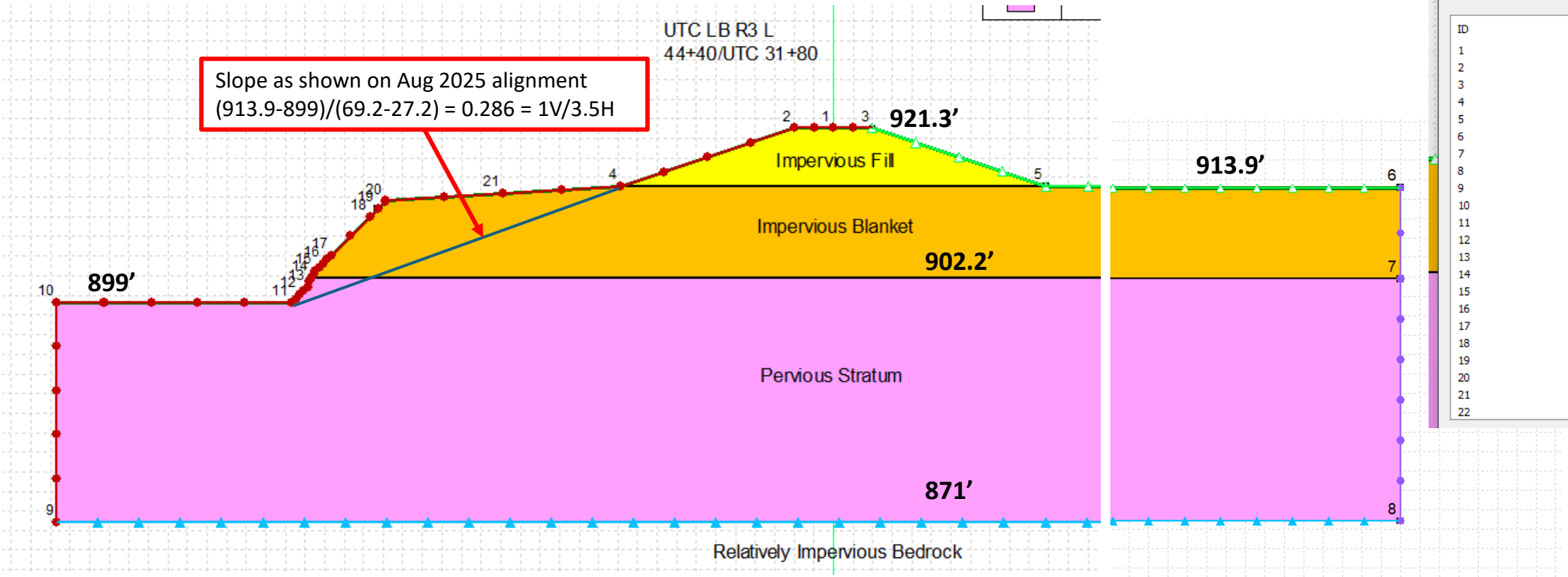
point #11 -69.2 899
point #4 -27.2 913.9

52' horiz 14.9' vert. $\frac{52'}{14.9'} = 3.5$
current roughly 1V:3.5H

★ try 1V:2H btwn #11 & #4
14.9' x 2 = 29.8'
move #4 to -69.2
+29.8
-39.4 x coord.
(so 178' offset btwn #20 + #4)

shift points #1-5 by 39.4 - 27.2 = 12.2'
(subtract 12.2')

Cross Section and SEEP/W



Define Points

ID	X (ft)	Y (ft)
1	0	921.3
2	-5	921.3
3	5	921.3
4	-27.2	913.9
5	27.2	913.9
6	2,000	913.9
7	2,000	902.2
8	2,000	871
9	-99.2	871
10	-99.2	899
11	-69.2	899
12	-68.2	900
13	-67.2	901
14	-66.7	902.2
15	-66.2	903
16	-65.2	904
17	-64.2	905
18	-59.2	910
19	-58.2	911
20	-57.2	912
21	-42.2	913
22	---	---

Define Boundary Conditions

BC Category: All

All Boundary Conditions

Name	Category	Hydraulic Boundary Type	Color
Bedrock - No Flow	Hydraulic	Water Rate	
Full Head	Hydraulic	Water Total Head	
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate	
Tailwater Head	Hydraulic	Water Total Head	

Water Rate = 0 ft³/sec

Water Total Head = 921.3 ft

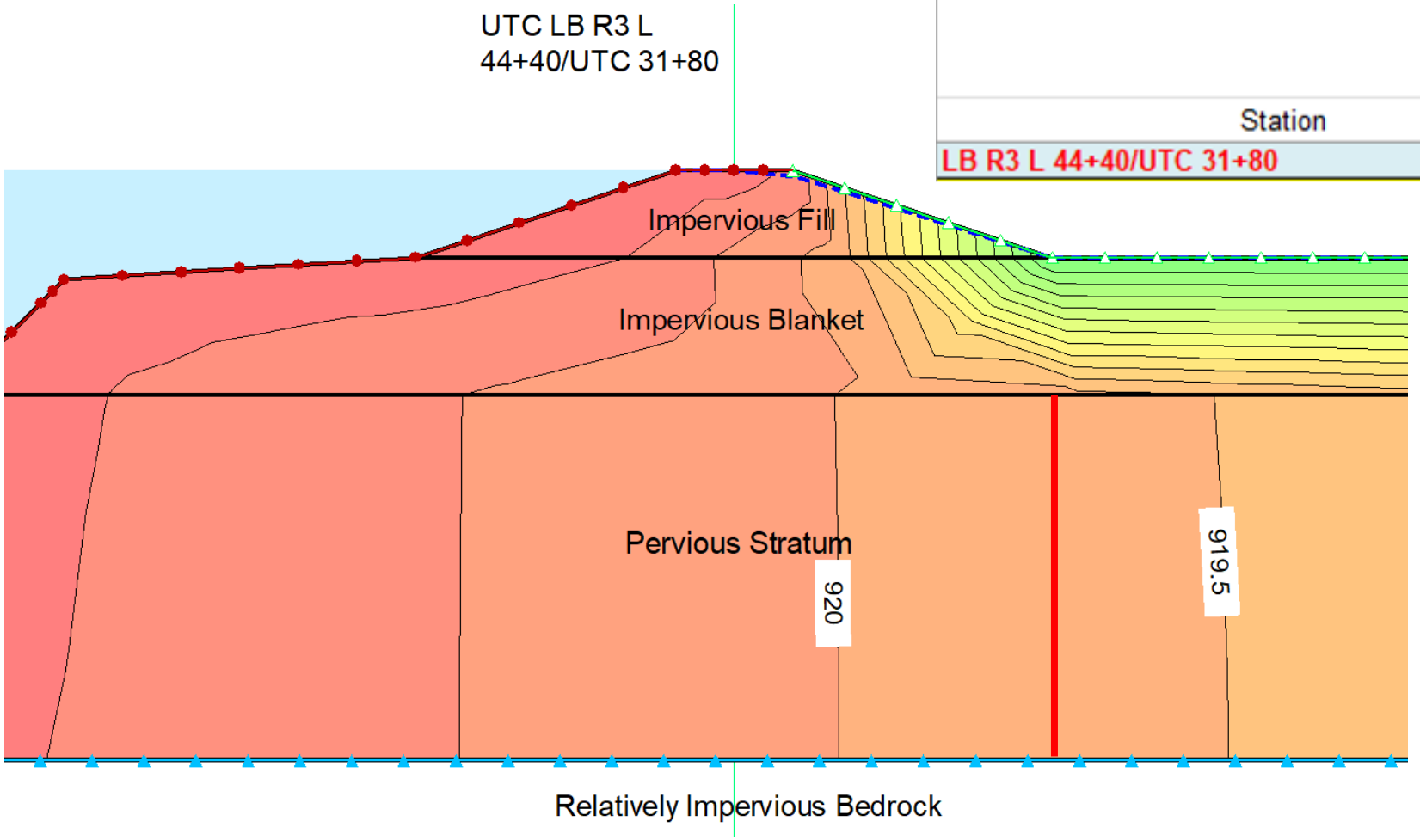
Water Rate = 0 ft³/sec

Potential Seepage Face Review

Water Total Head = 913.9 ft

Adjusted permeability of impervious embankment and blanket until water total head elevation at landside levee toe in SEEP/W matched blanket theory spreadsheet

Station	Head at Toe (elevation msl) h_o
LB R3 L 44+40/UTC 31+80	919.84

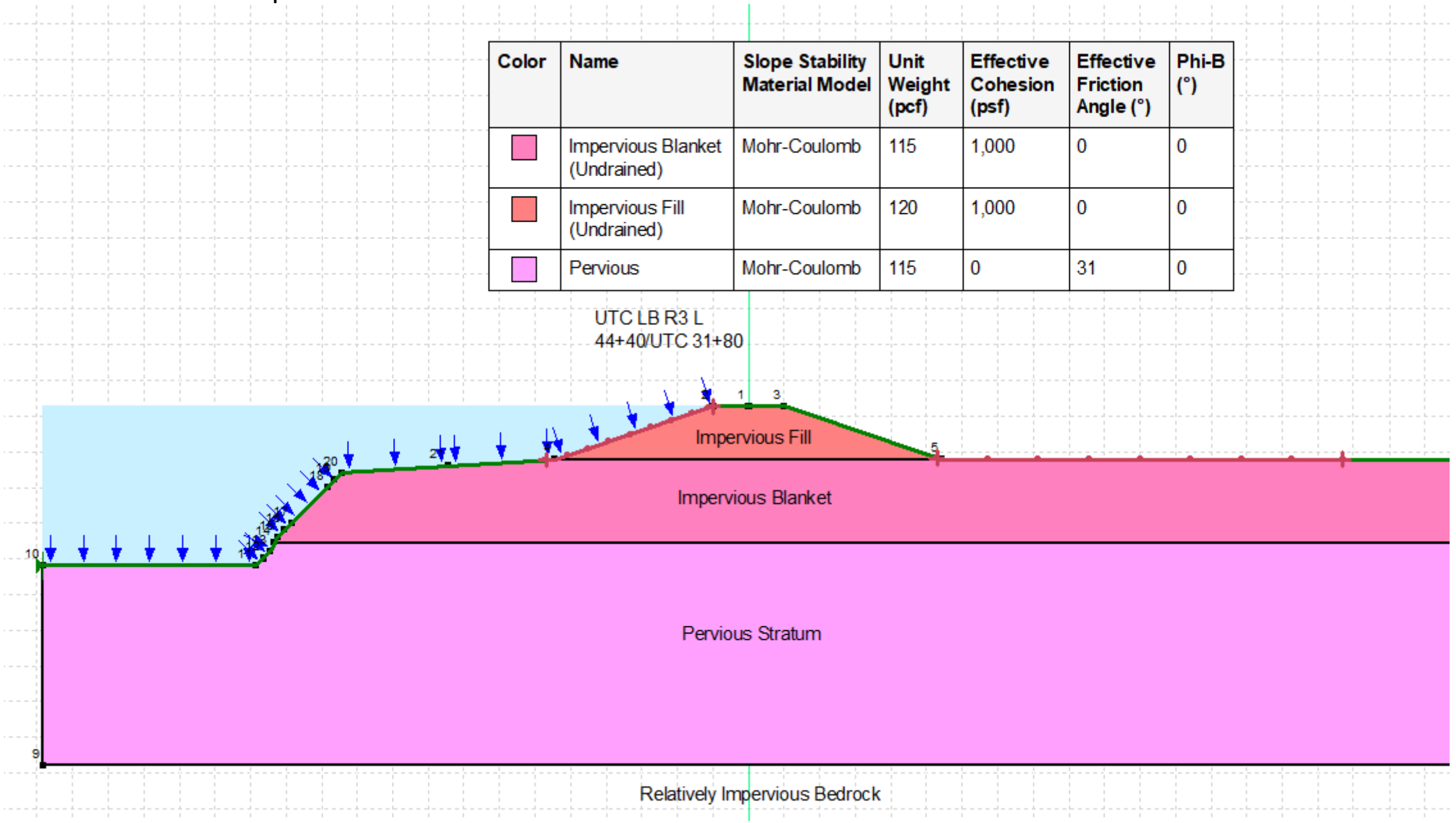


Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)
	Impervious Blanket (Drained)	Saturated Only	7.5e-06
	Impervious Fill (Drained)	Saturated Only	7.5e-06
	Pervious	Saturated Only	0.003

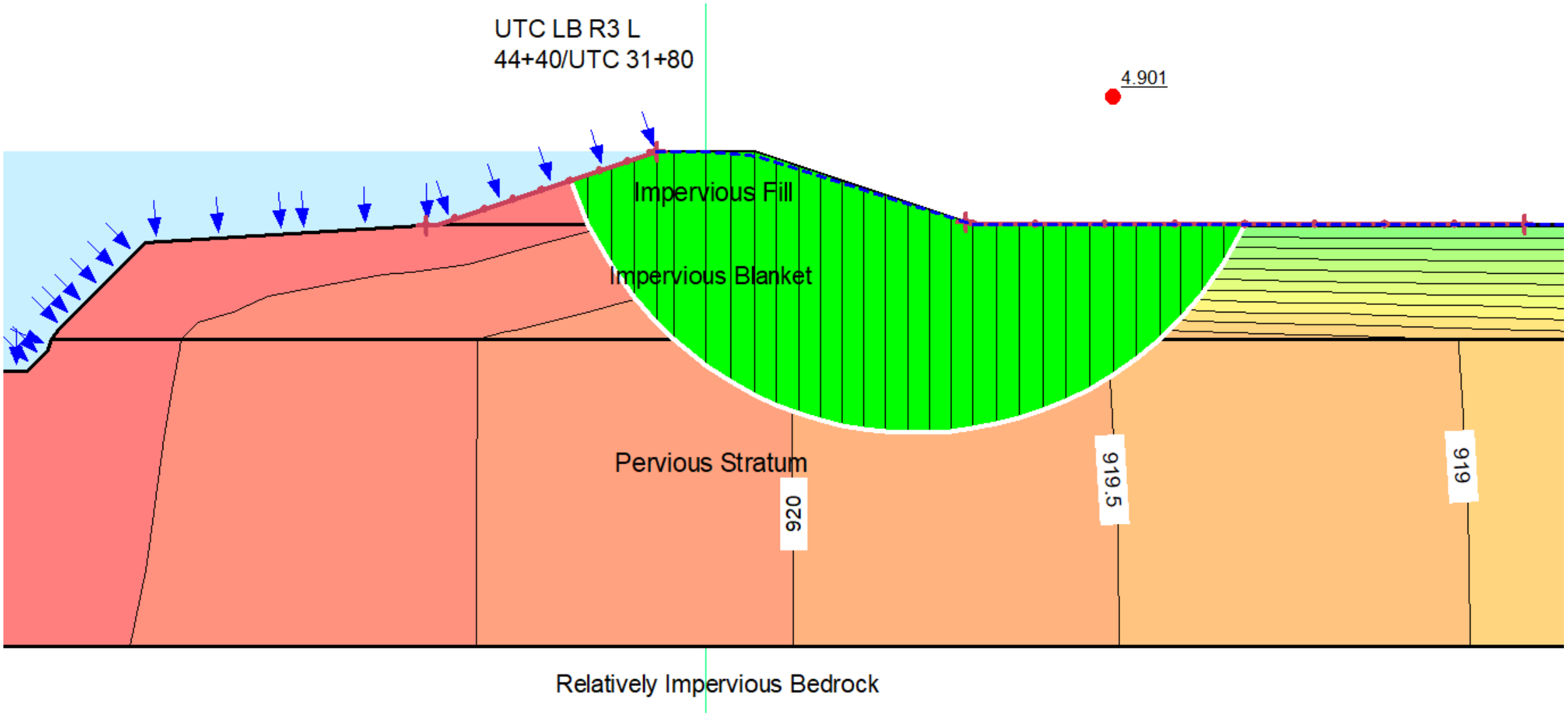
Adjust to calibrate

End of Construction Input

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Impervious Blanket (Undrained)	Mohr-Coulomb	115	1,000	0	0
	Impervious Fill (Undrained)	Mohr-Coulomb	120	1,000	0	0
	Pervious	Mohr-Coulomb	115	0	31	0

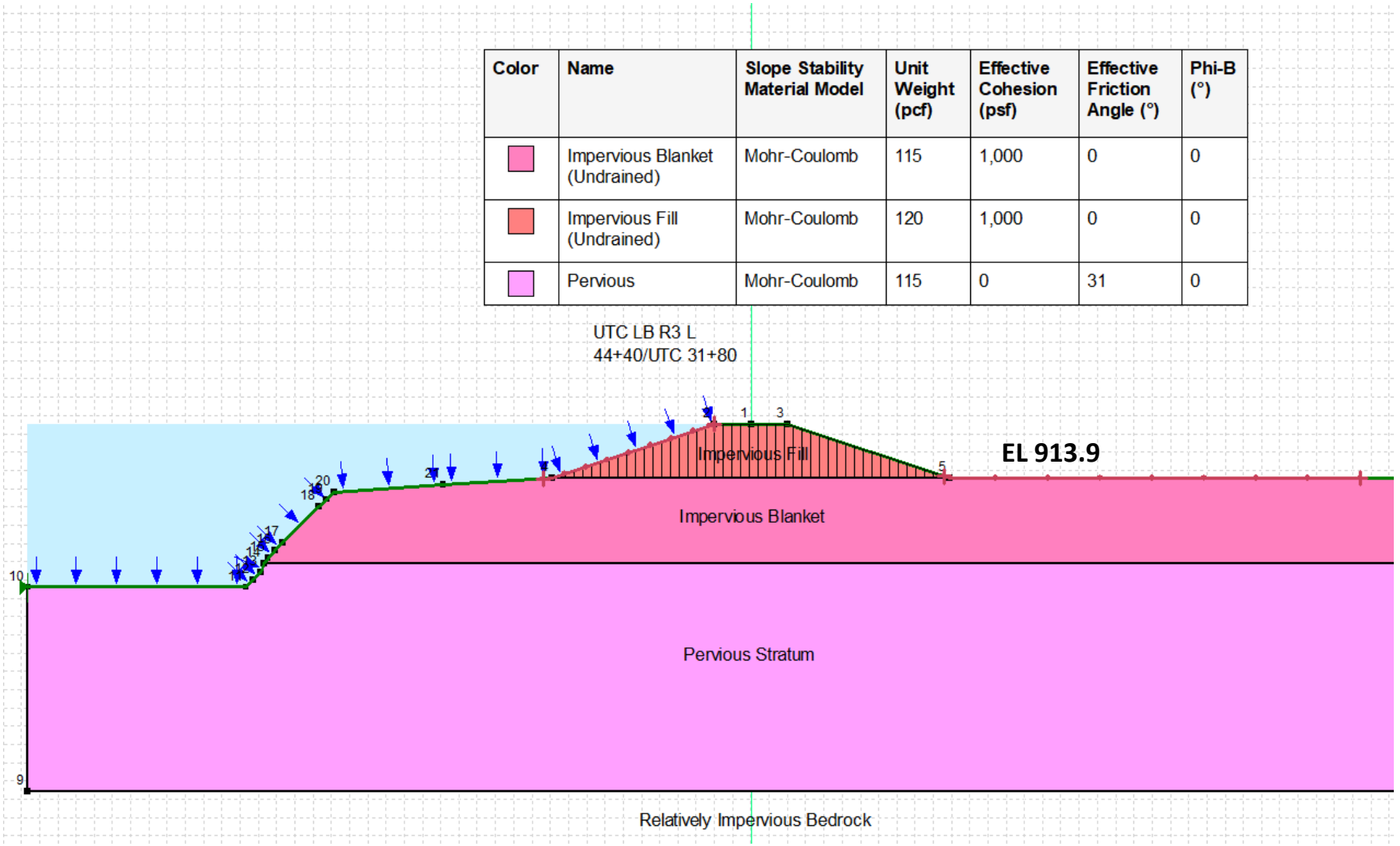


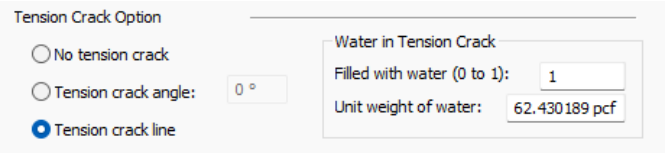
End of Construction Results

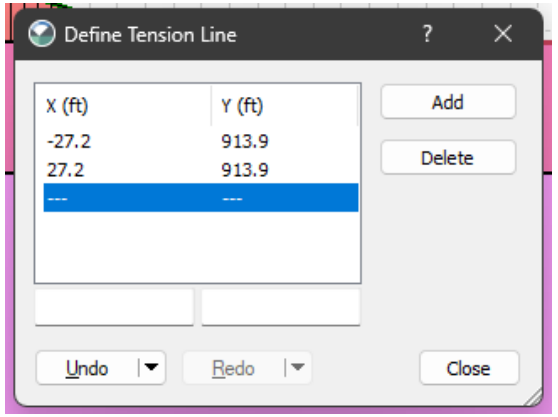


End of Construction Input with Tension Crack

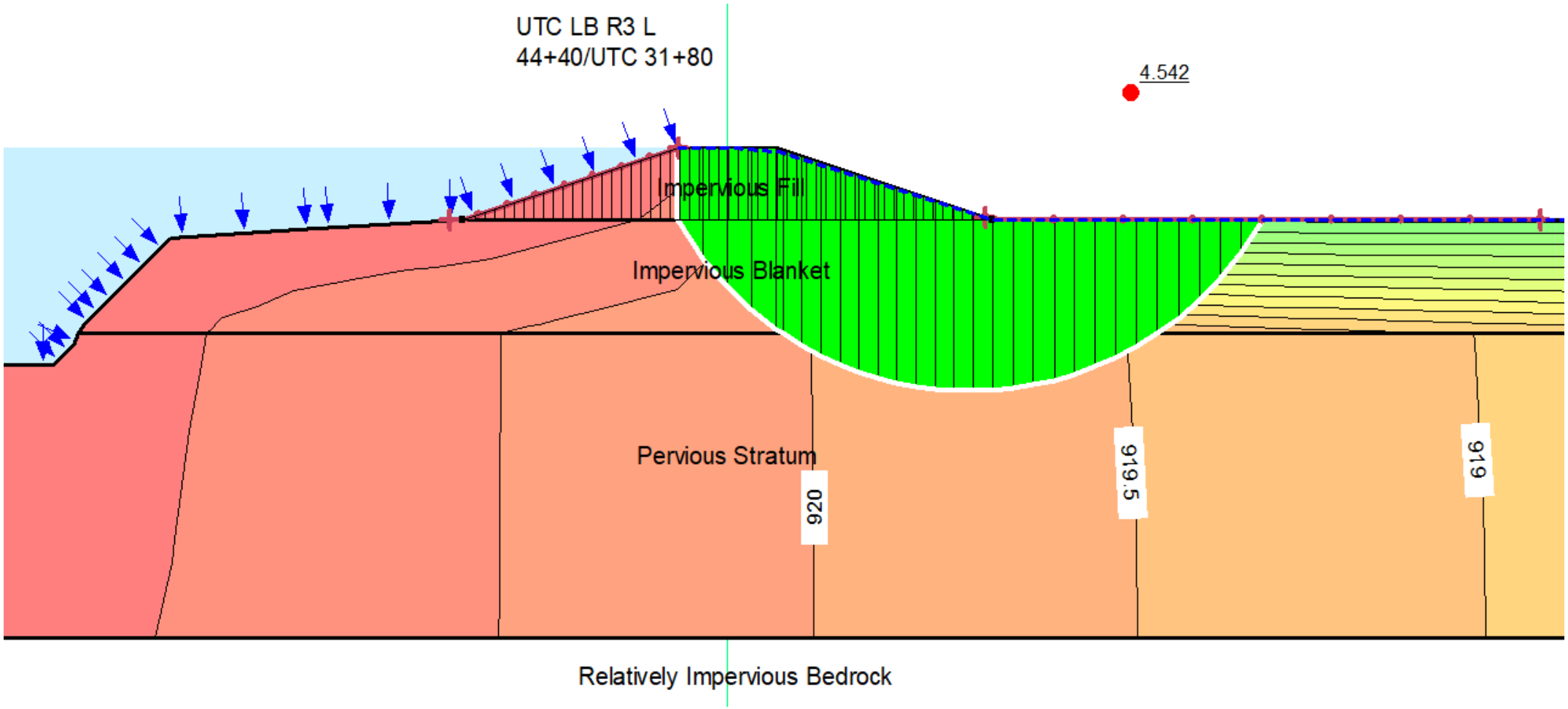
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Impervious Blanket (Undrained)	Mohr-Coulomb	115	1,000	0	0
	Impervious Fill (Undrained)	Mohr-Coulomb	120	1,000	0	0
	Pervious	Mohr-Coulomb	115	0	31	0





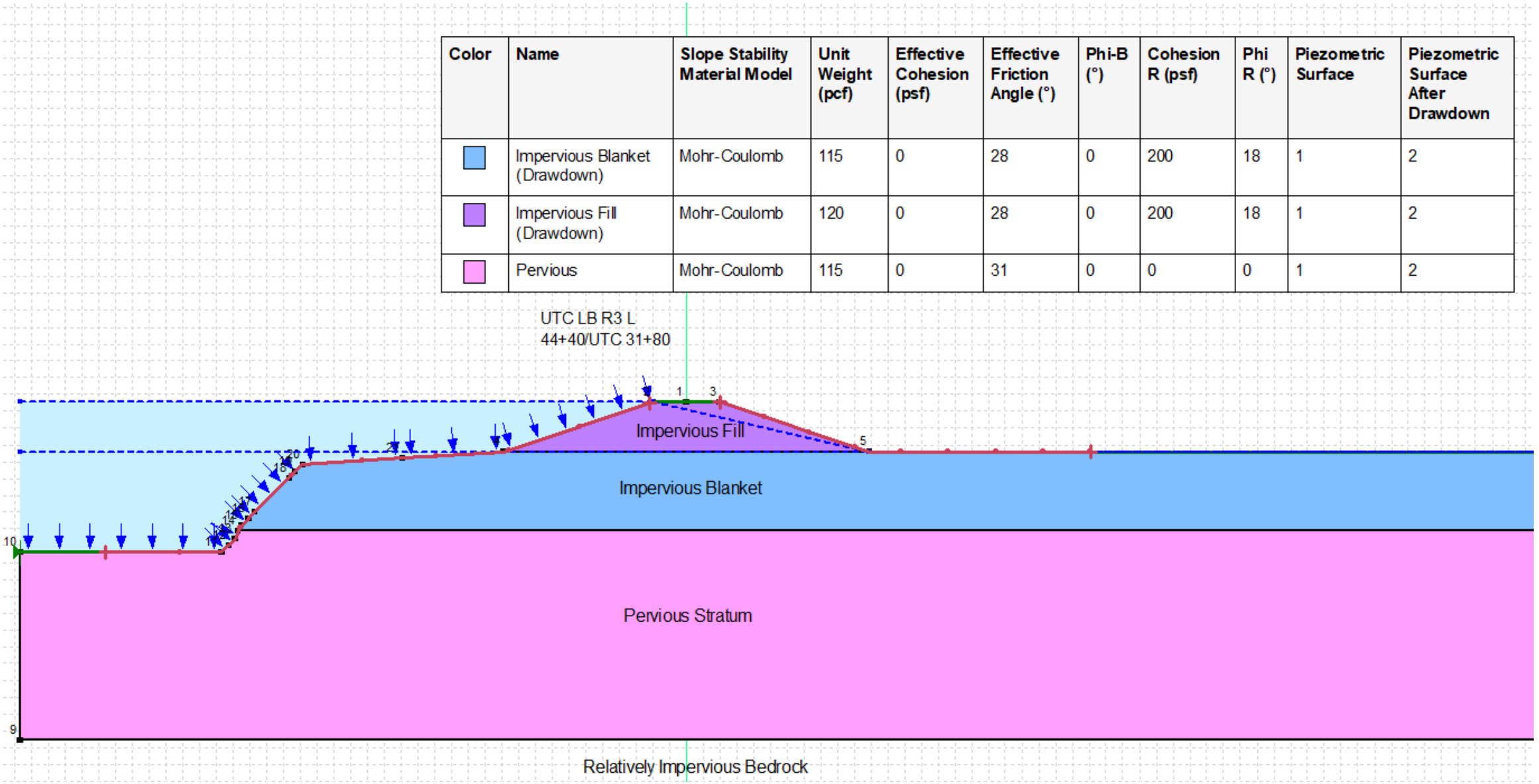


End of Construction Output with Tension Crack

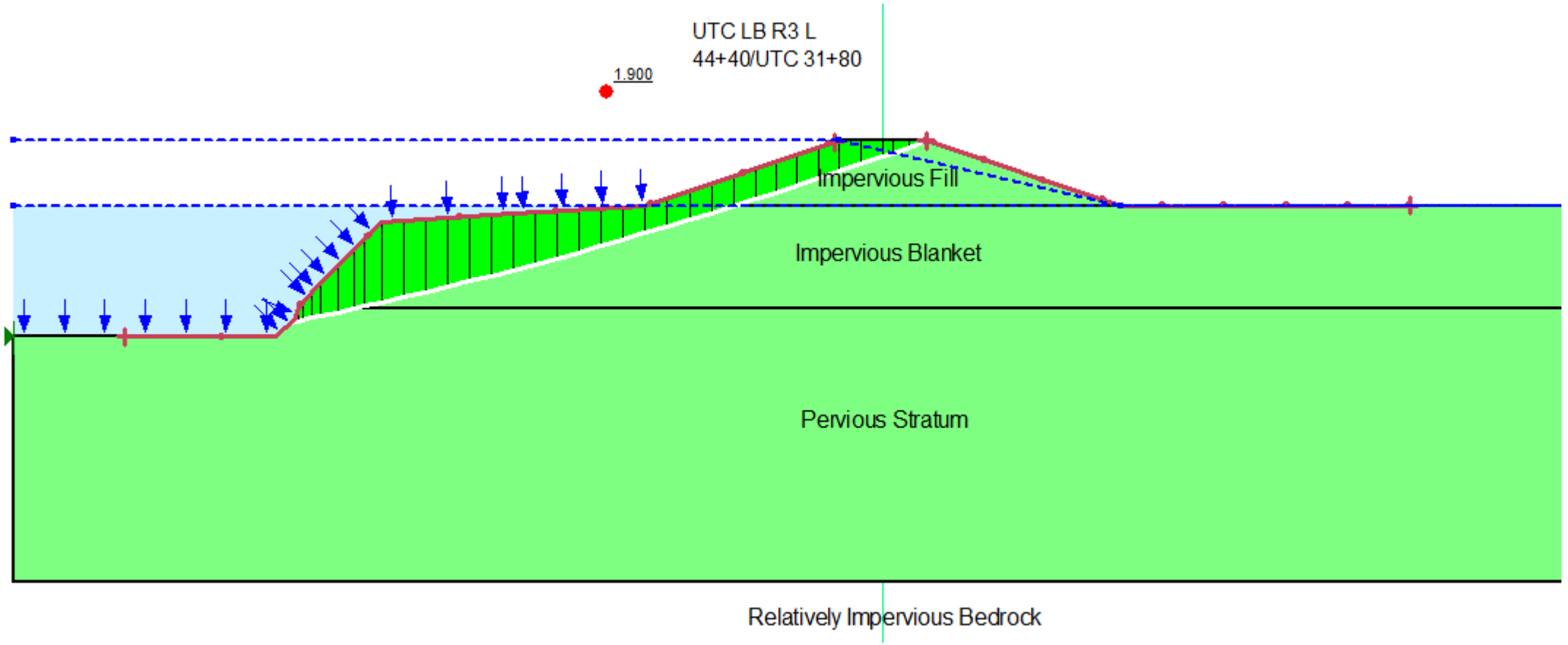


Rapid Drawdown Input

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)	Piezometric Surface	Piezometric Surface After Drawdown
	Impervious Blanket (Drawdown)	Mohr-Coulomb	115	0	28	0	200	18	1	2
	Impervious Fill (Drawdown)	Mohr-Coulomb	120	0	28	0	200	18	1	2
	Pervious	Mohr-Coulomb	115	0	31	0	0	0	1	2

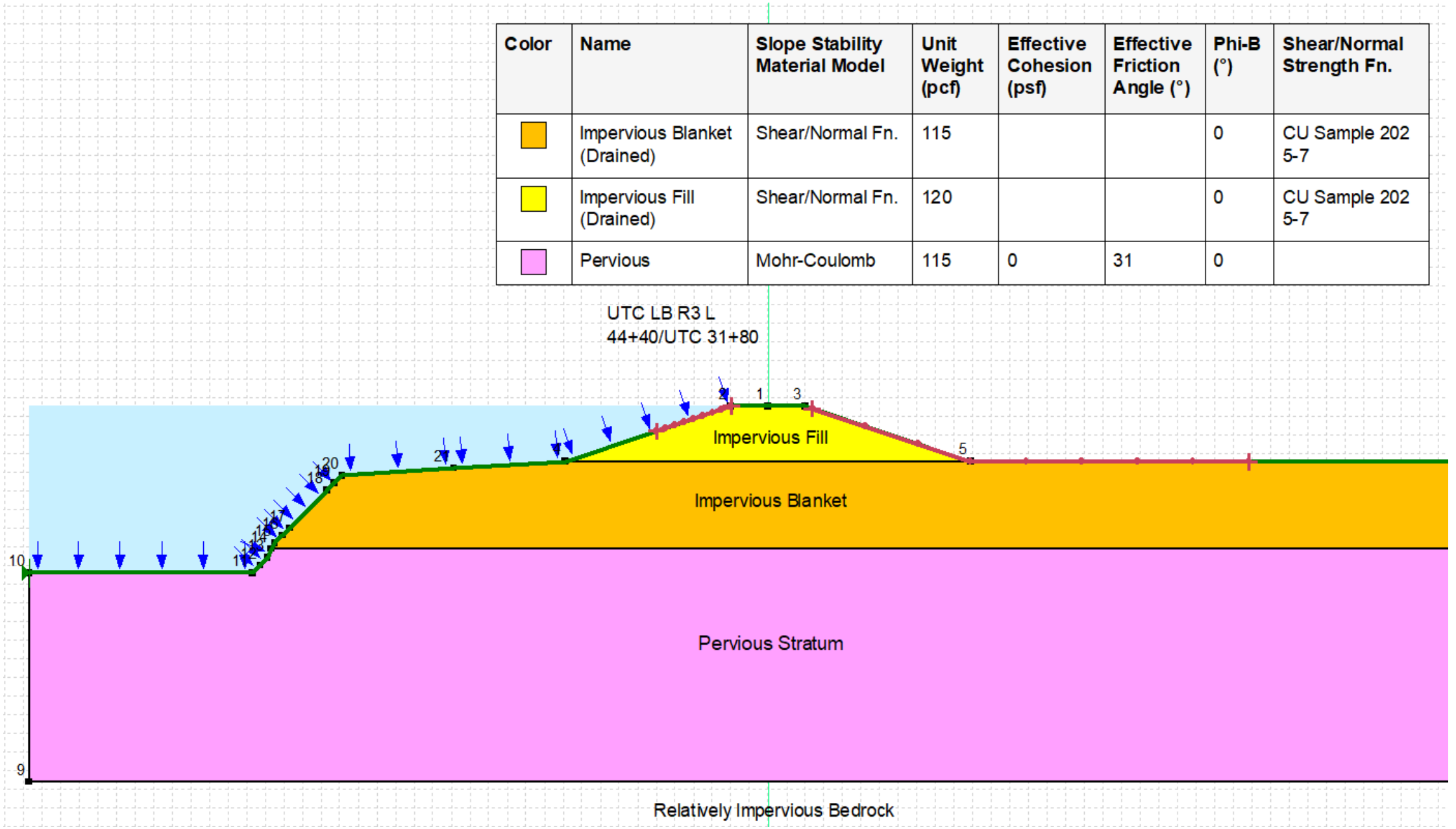


Rapid Drawdown Results

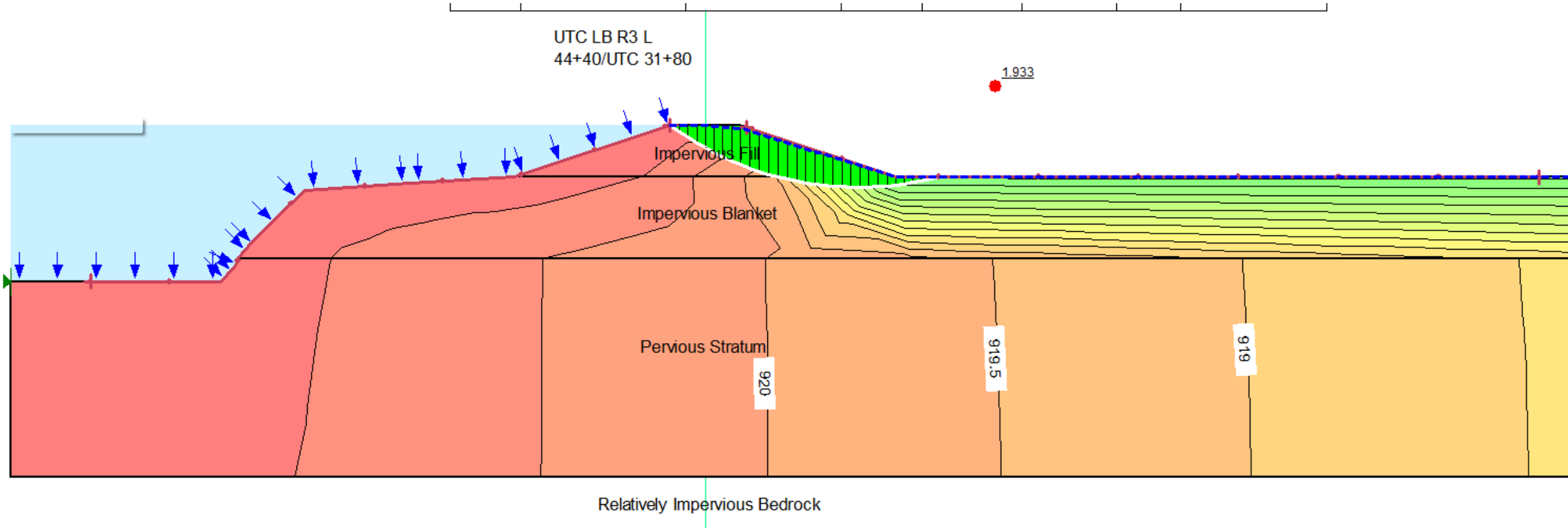


Steady State Input

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Shear/Normal Strength Fn.
	Impervious Blanket (Drained)	Shear/Normal Fn.	115			0	CU Sample 202 5-7
	Impervious Fill (Drained)	Shear/Normal Fn.	120			0	CU Sample 202 5-7
	Pervious	Mohr-Coulomb	115	0	31	0	



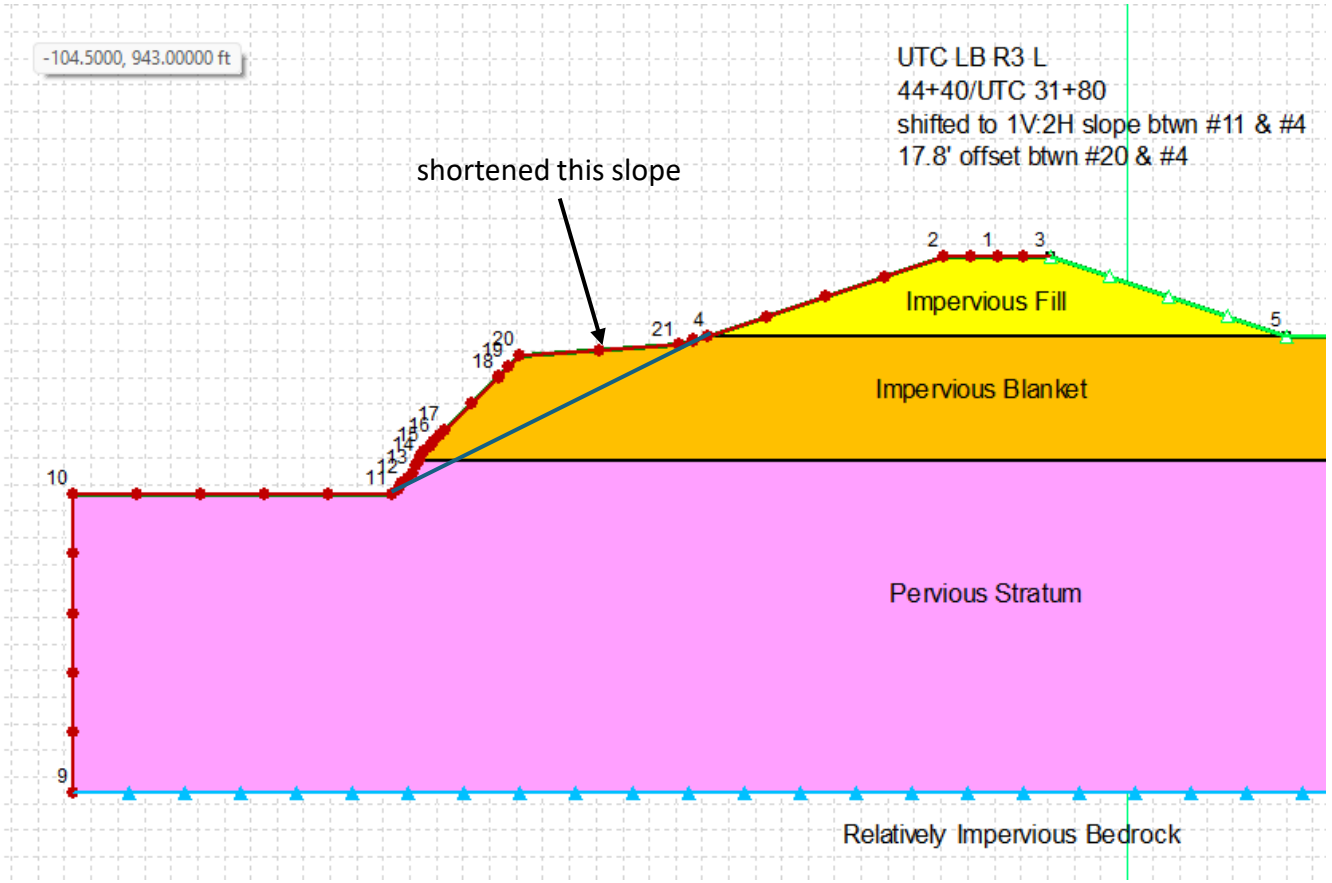
Steady State Results



Additional analyses performed for approx. 1V:2H slope

Shortened distance between riverside levee toe and bottom of bank slope

(913.9-899)/(69.2-39.4) = 14.9/29.8 = 0.5 = 1V:2H



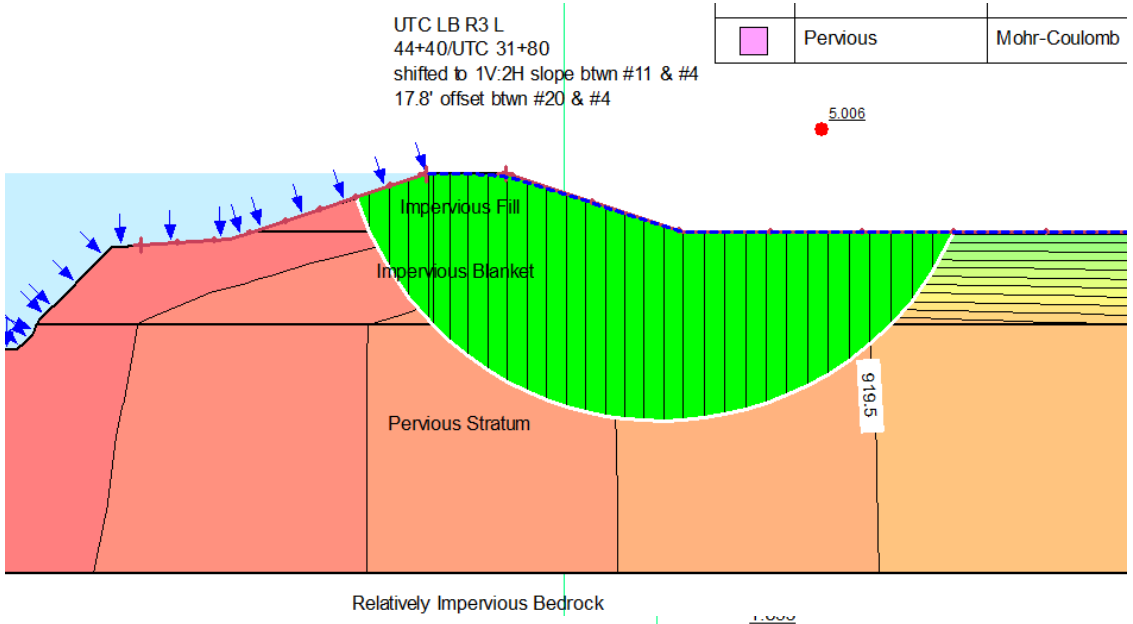
Define Points

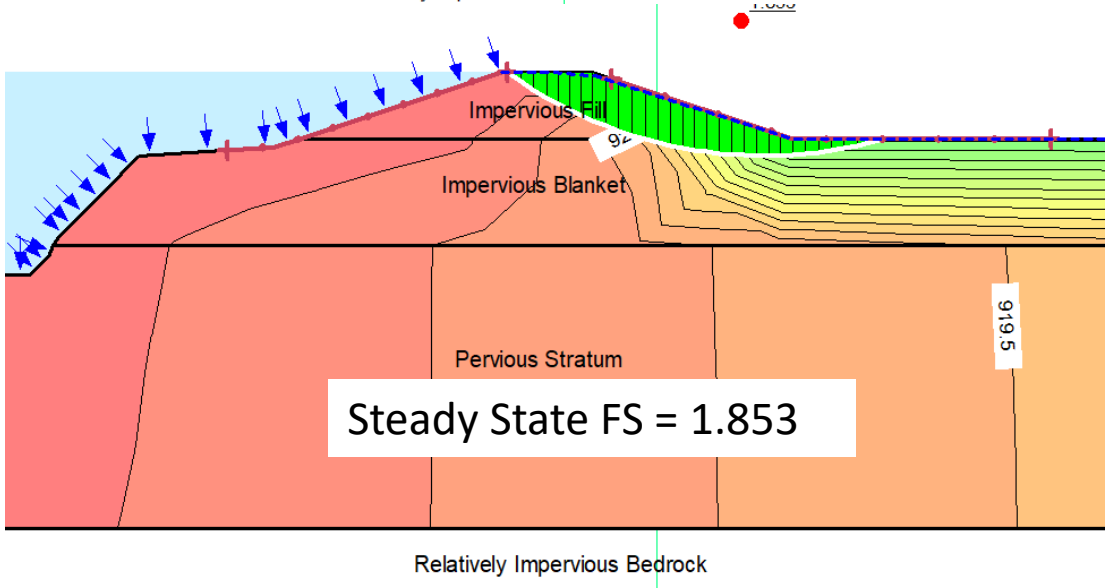
ID	X (ft)	Y (ft)
1	-12.2	921.3
2	-17.2	921.3
3	-7.2	921.3
4	-39.4	913.9
5	15	913.9
6	2,000	913.9
7	2,000	902.2
8	2,000	871
9	-99.2	871
10	-99.2	899
11	-69.2	899
12	-68.2	900
13	-67.2	901
14	-66.7	902.2
15	-66.2	903
16	-65.2	904
17	-64.2	905
18	-59.2	910
19	-58.2	911
20	-57.2	912
21	-42.2	913
22	---	---

Shortened the distance more for the 1V:1.5H slope

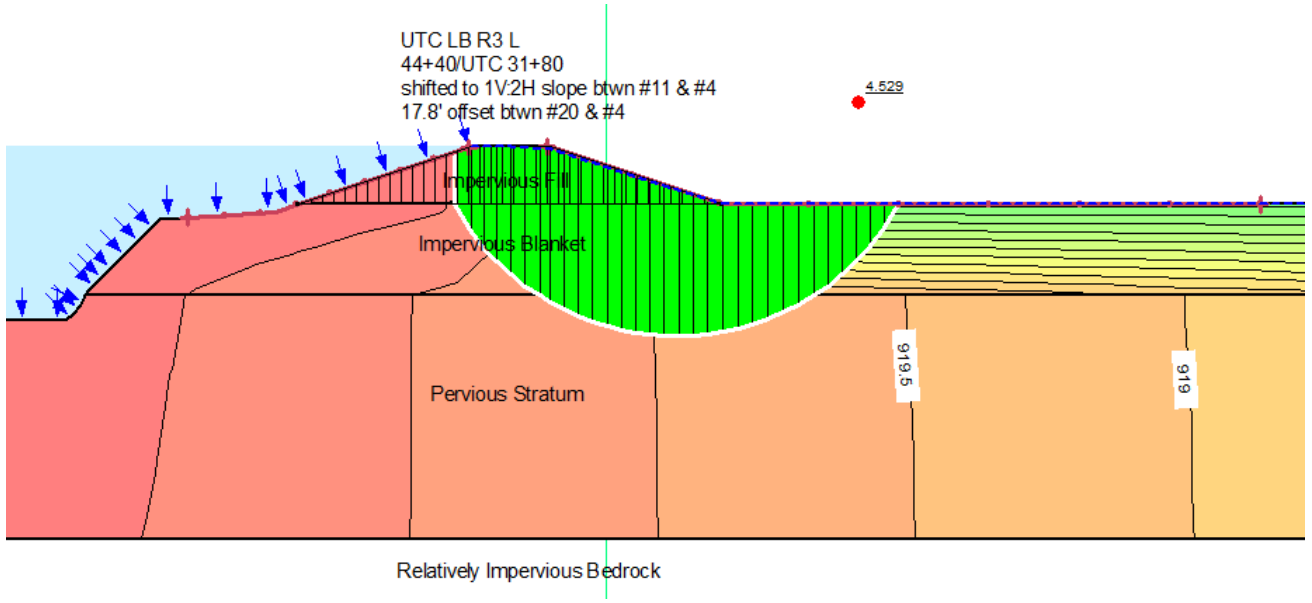
1V:2H Analysis Results

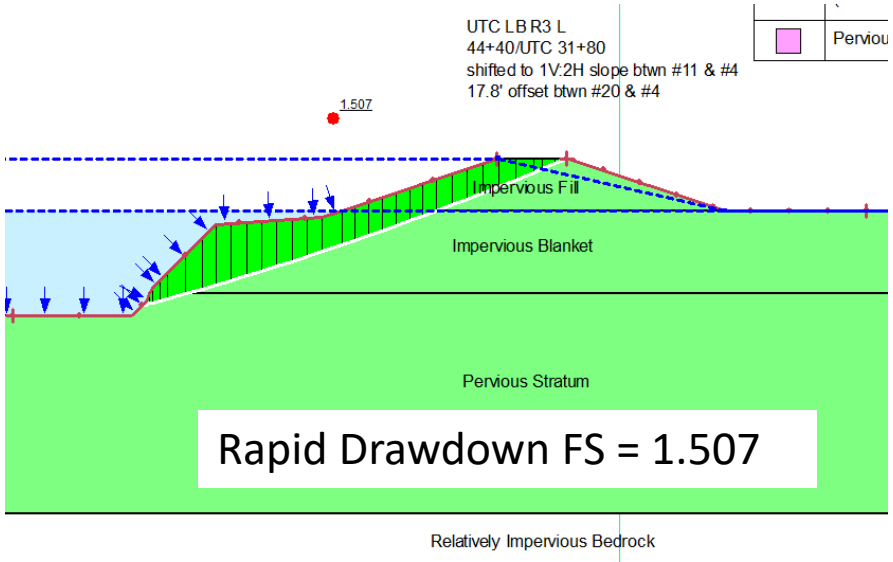
End of Construction FS = 5.006





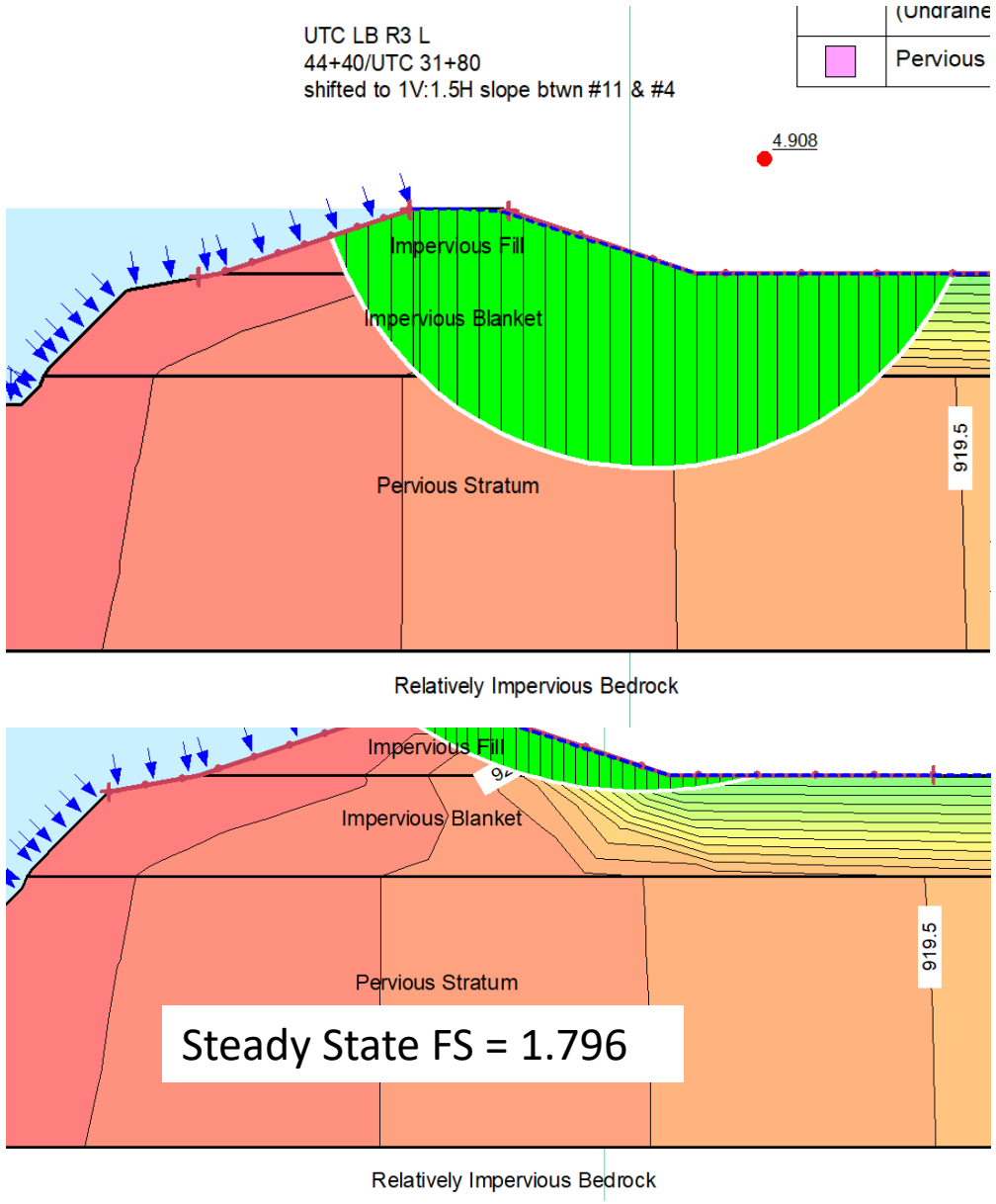
End of Construction w Tension Crack FS = 4.529



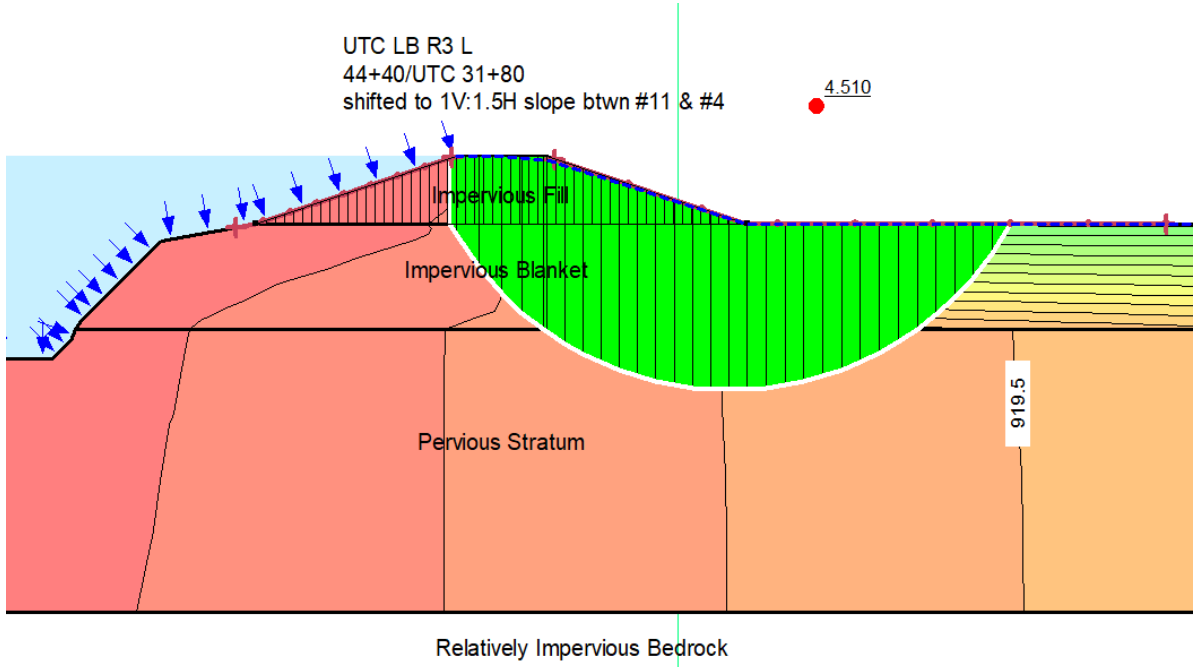


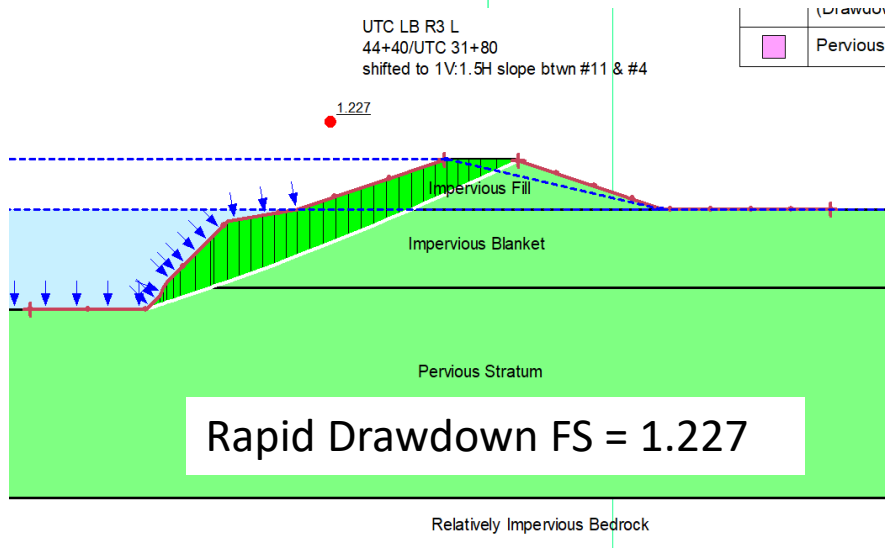
1V:1.5H Analysis Results

End of Construction FS = 4.908



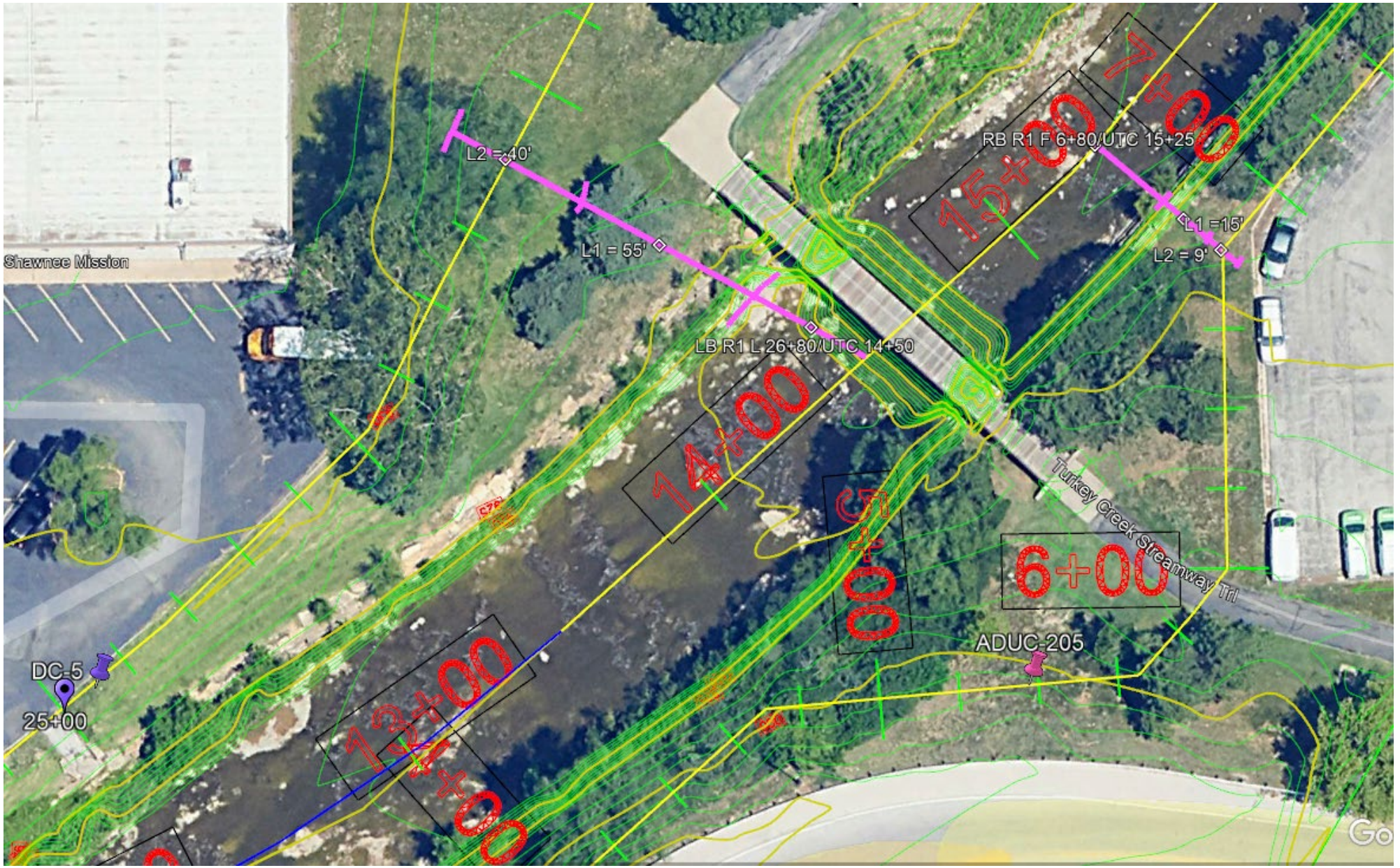
End of Construction w Tension Crack FS = 4.510





Results for Other Levee Cross Sections

LB R1 26+80-UTC 14+50



LB R1 26+80-UTC 14+50

Levee Unit Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYSIS TAB AND ASSOCIATED PL

Case Analyzed - Water at Levee Top

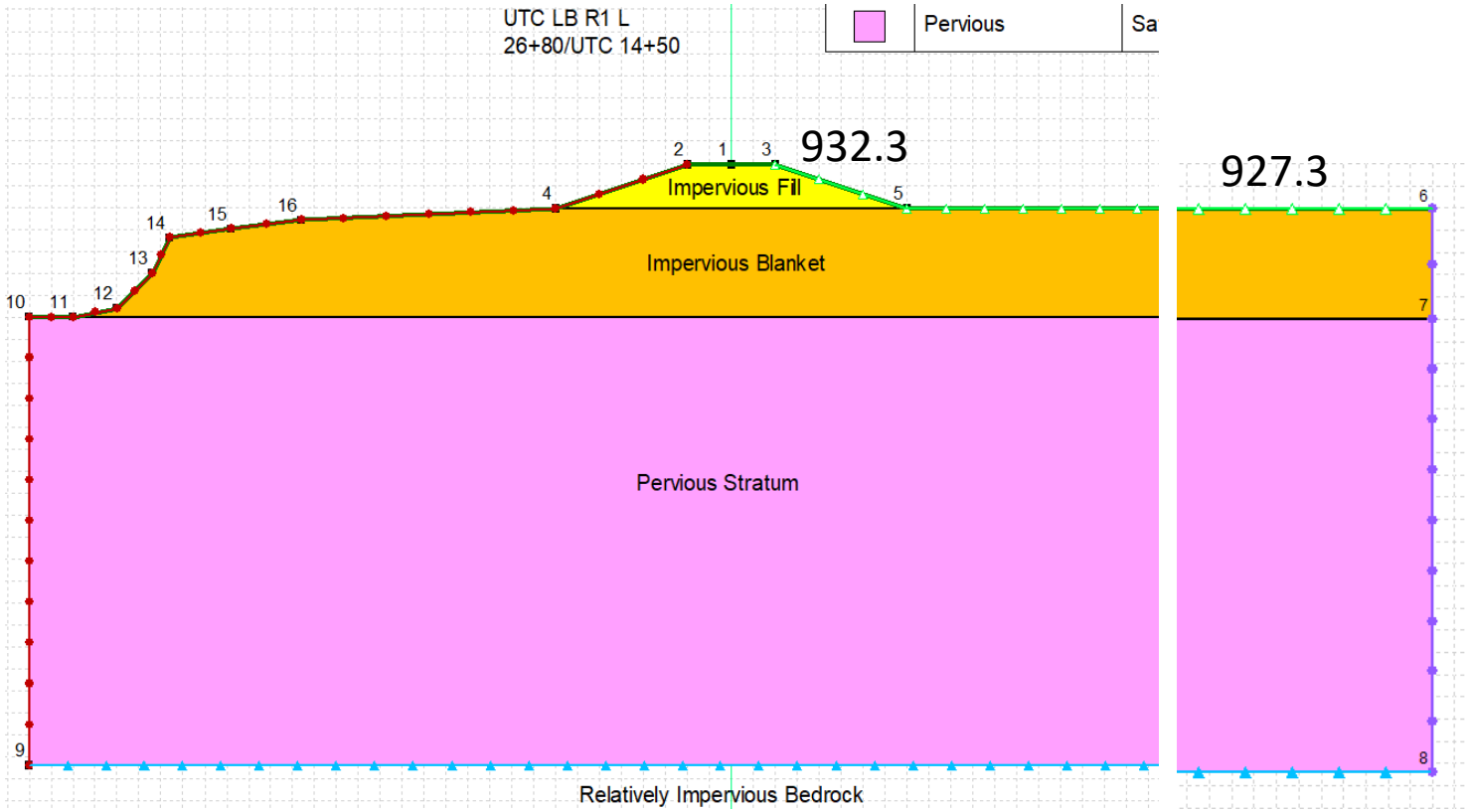
Blanket Unit Weight = 115.0 pcf (saturated)

	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Soil Boring	Land Blanket Bottom	Modified Top of Bedrock*	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Reference	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	Riverside	Levee	Land Side
Station	(msl)	(msl)	(msl)	(msl)	(msl)			(msl)	H	k_r/k_{br}	k_r/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
LB R1 L 26+80/UTC 14+50	932.3	927.5	927.5	927.5	915	DC-5	915	864	4.8	400	400	12.5	12.5	12.5	51	55	40	infinity

calibration

Factor		Effective Seepage Length (ft)		Head at Toe (ft)	Head at Toe (elevation msl)	Computed Hydraulic Gradient	Critical Hydraulic Gradient	Factor of Safety for Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe	FS horizontal gradient ($i_{h,c}$ = 1/5 = 0.2)
		River Side	Land Side	Toe (ft)	Toe (elevation msl)	Gradient	Gradient	Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe	FS horizontal gradient ($i_{h,c}$ = 1/5 = 0.2)
c_r	c_l	x_1	x_3	h_o	h_o	i_o	i_c	(i_c/i_o)	(i_h)	$(i_{h,c}/i_h)$
0.001980	0.001980	54.78	504.98	4.04	931.54	0.32	0.84	2.61	0.05	3.95

LB R1 26+80-UTC 14+50



Name: Bedrock - No Flow
Kind: Water Rate
Boundary Condition Type
Constant: 0 ft³/sec
Function:

Name: Tailwater Head
Kind: Water Total Head
Boundary Condition Type
Constant: 927.3 ft
Function:

Name: Full Head
Kind: Water Total Head
Boundary Condition Type
Constant: 932.3 ft
Function:

Name: Landside Slope - Potential Seepage Face
Kind: Water Rate
Boundary Condition Type
Constant: 0 ft³/sec
Function:
Potential Seepage Face Review

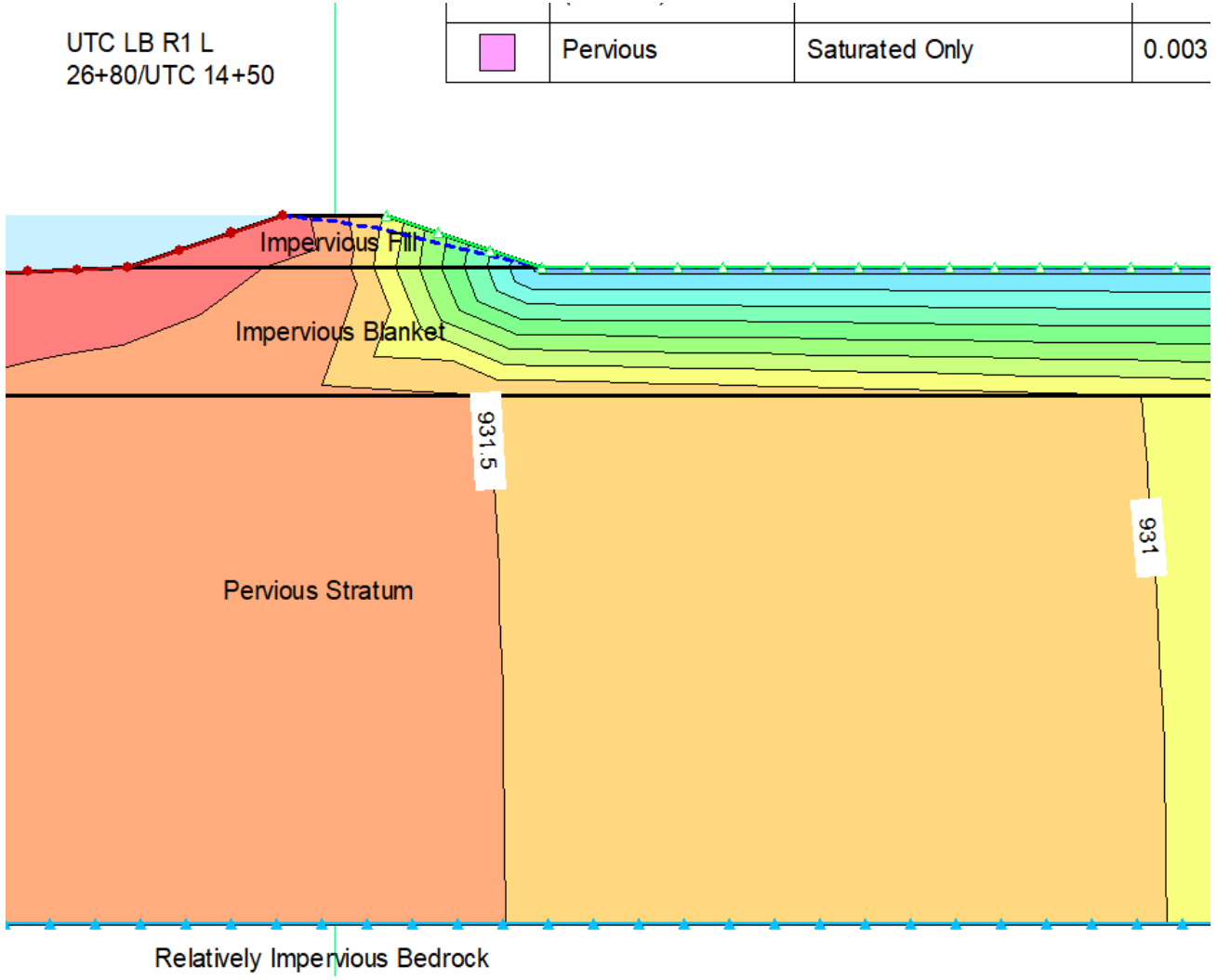
Define Boundary Conditions

BC Category: All

All Boundary Conditions

Name	Category	Hydraulic Boundary Type	Color
Bedrock - No Flow	Hydraulic	Water Rate	
Full Head	Hydraulic	Water Total Head	
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate	
Tailwater Head	Hydraulic	Water Total Head	

LB R1 26+80-UTC 14+50



calibration
Head at
Toe (elevation msl)
h_o
931.54

Name: Impervious Blanket (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	7.5e-06 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

Name: Pervious

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	0.003 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

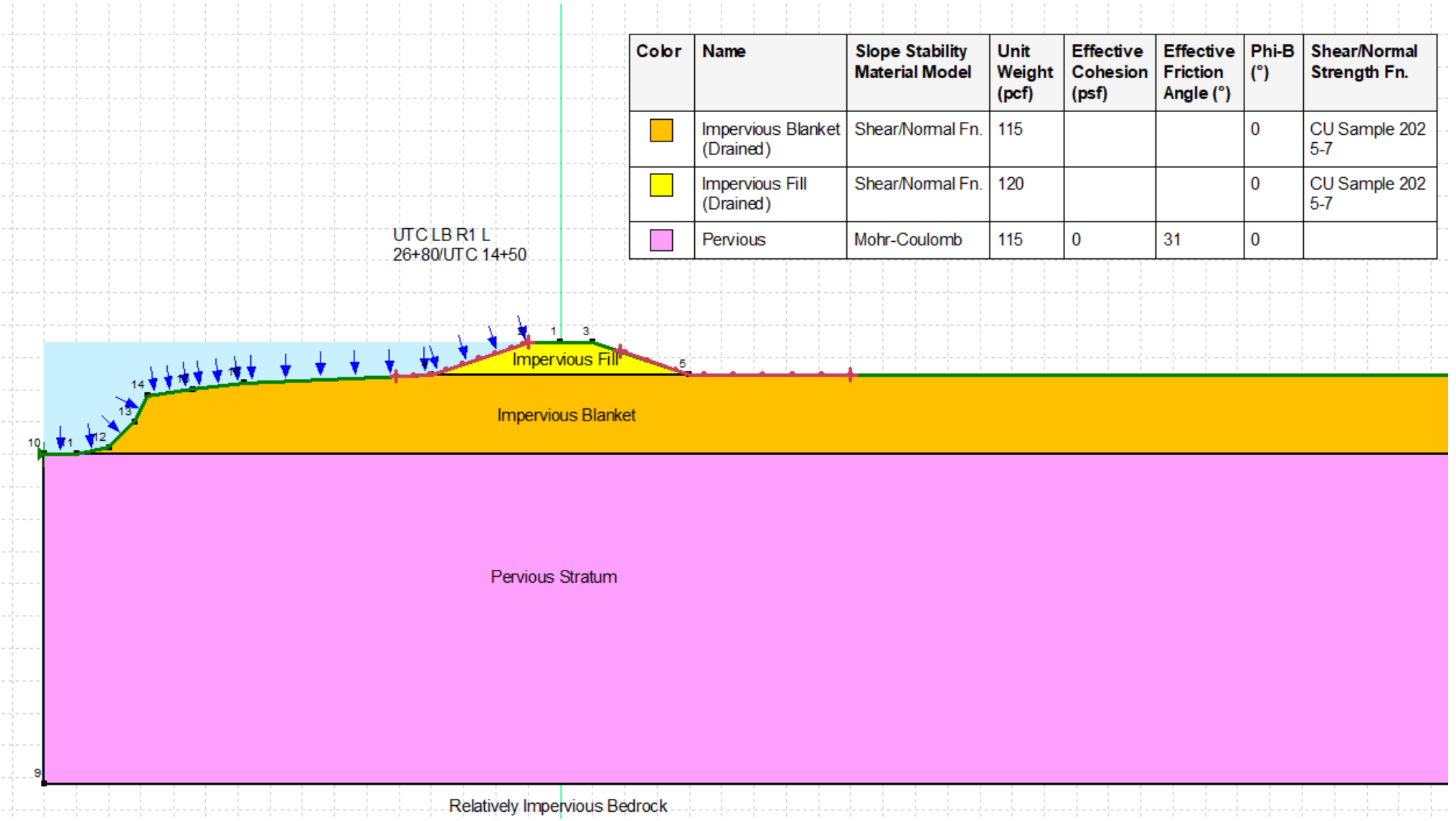
Name: Impervious Fill (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	7.5e-06 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

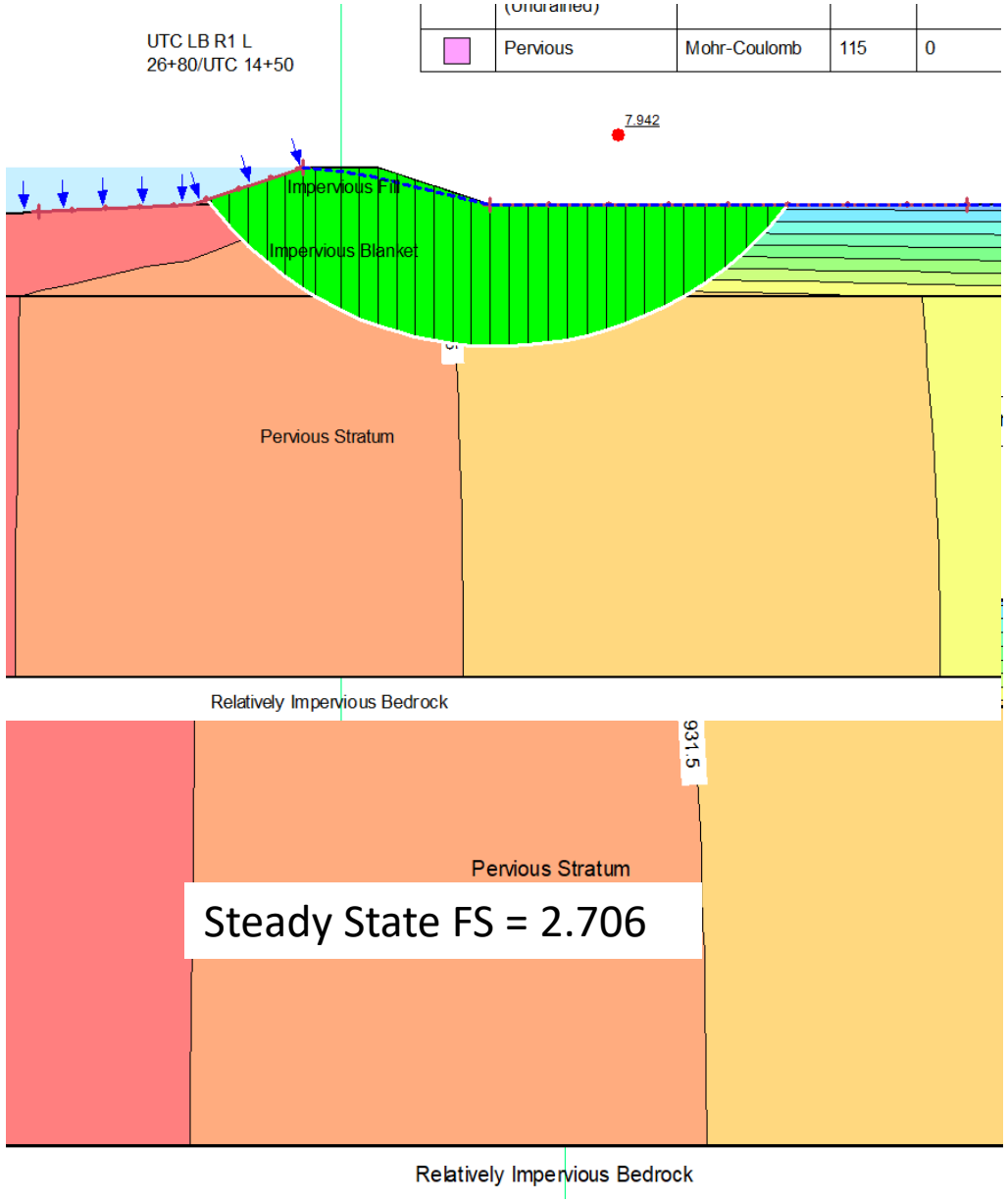
LB R1 26+80-UTC 14+50

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Shear/Normal Strength Fn.
	Impervious Blanket (Drained)	Shear/Normal Fn.	115			0	CU Sample 2025-7
	Impervious Fill (Drained)	Shear/Normal Fn.	120			0	CU Sample 2025-7
	Pervious	Mohr-Coulomb	115	0	31	0	

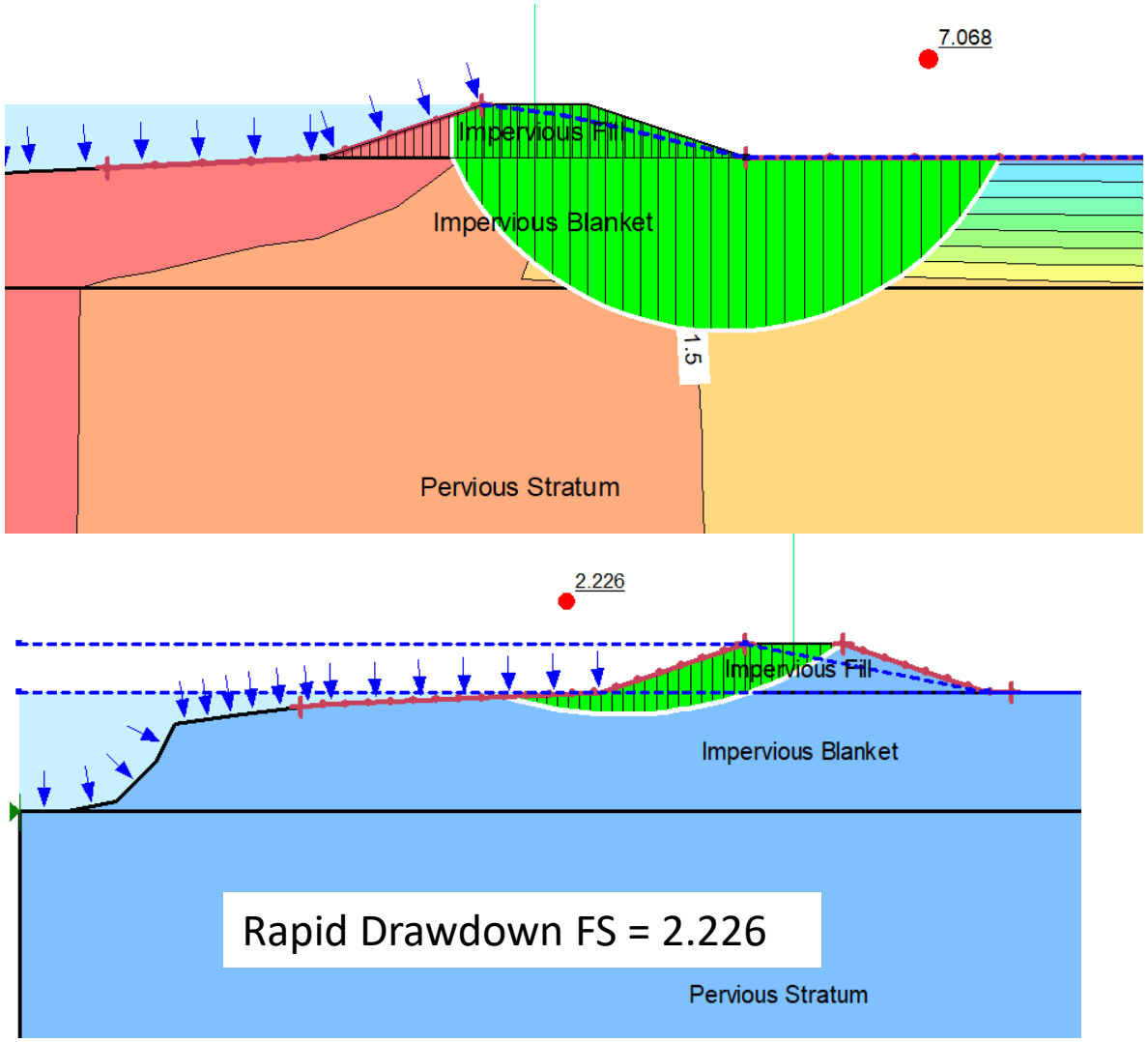


LB R1 26+80/UTC 14+50 ~1V:3H Analysis Results

End of Construction FS = 7.942

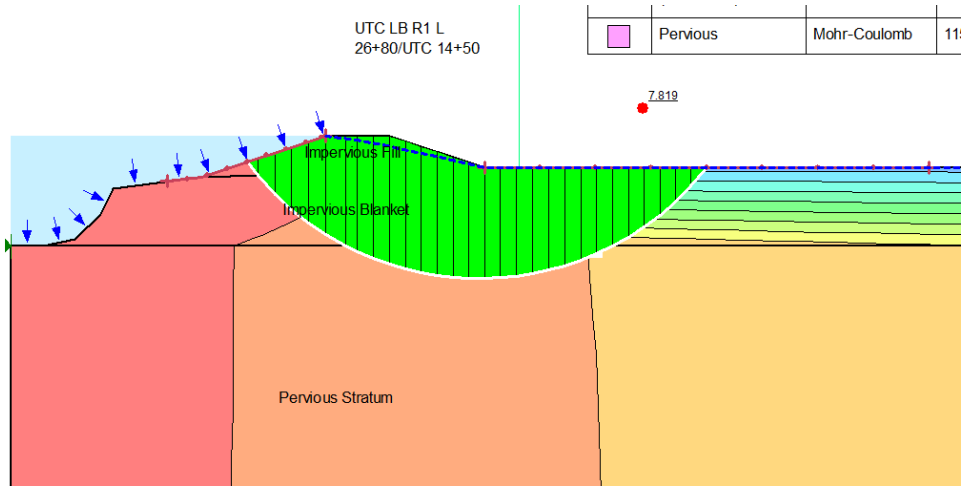


End of Construction w Tension Crack FS = 7.068

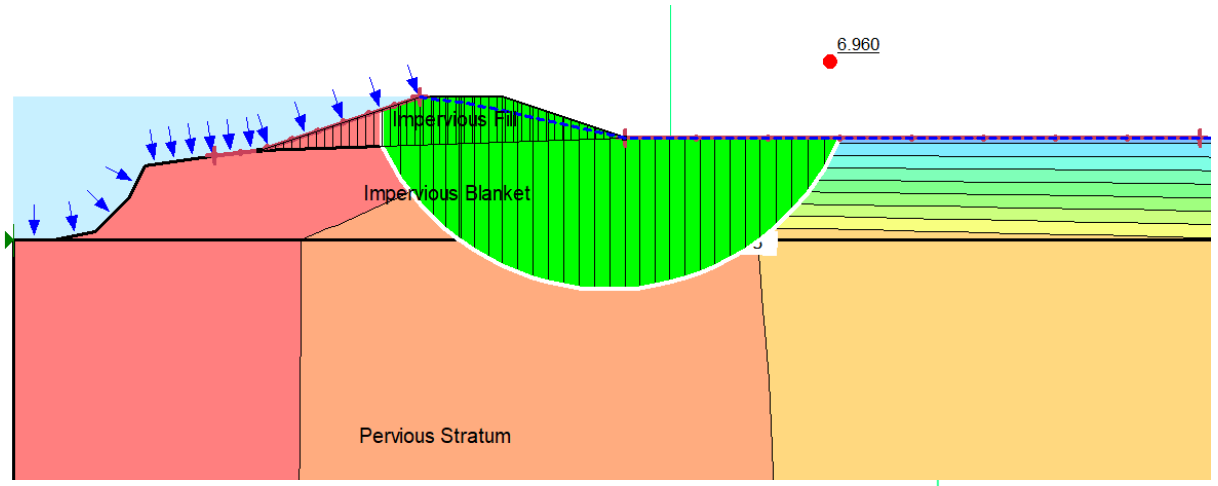


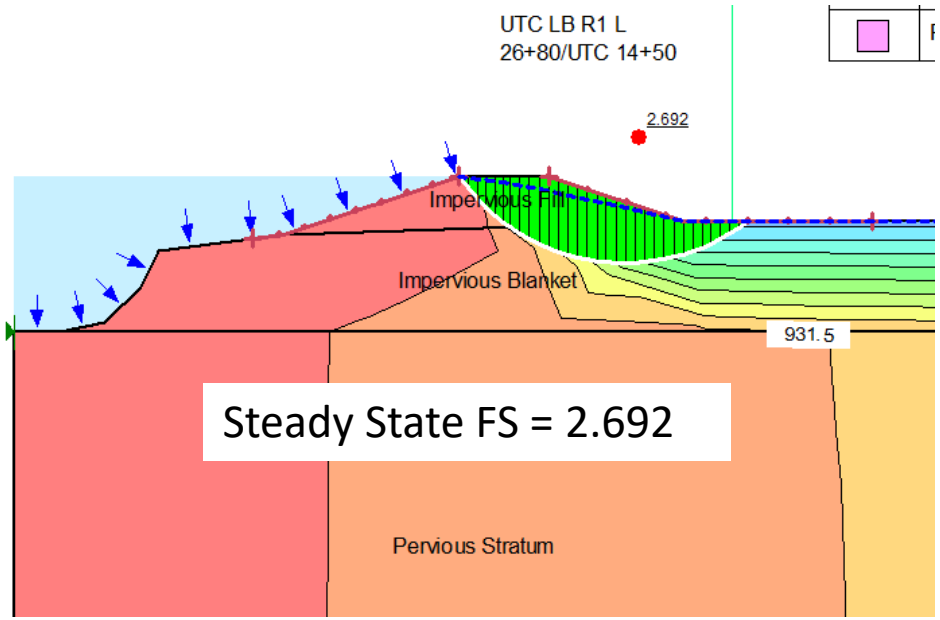
LB R1 26+80/UTC 14+50 ~1V:2H Analysis Results

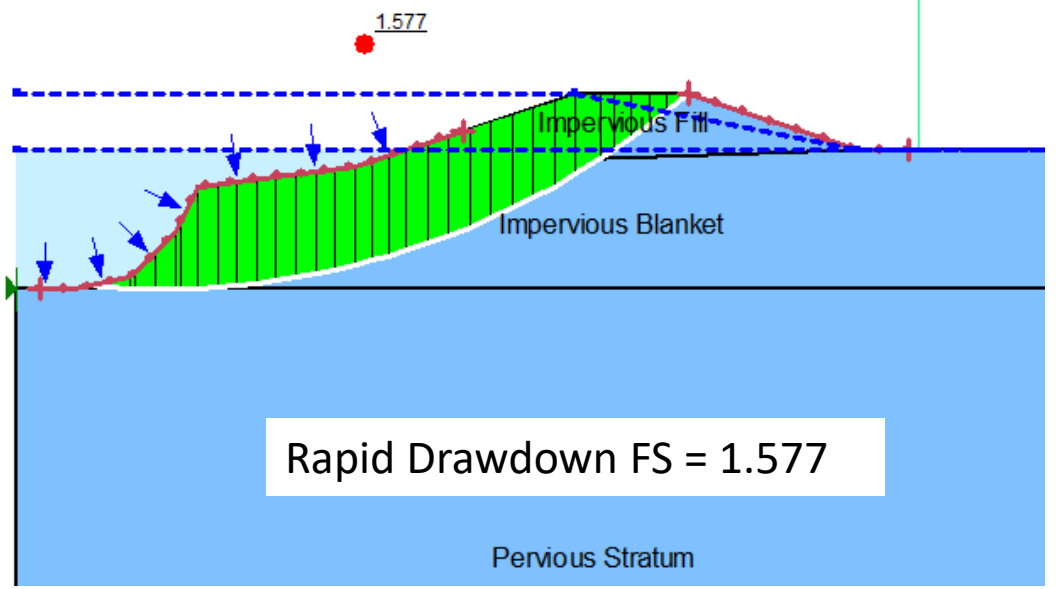
End of Construction FS = 7.819



End of Construction w Tension Crack FS = 6.960

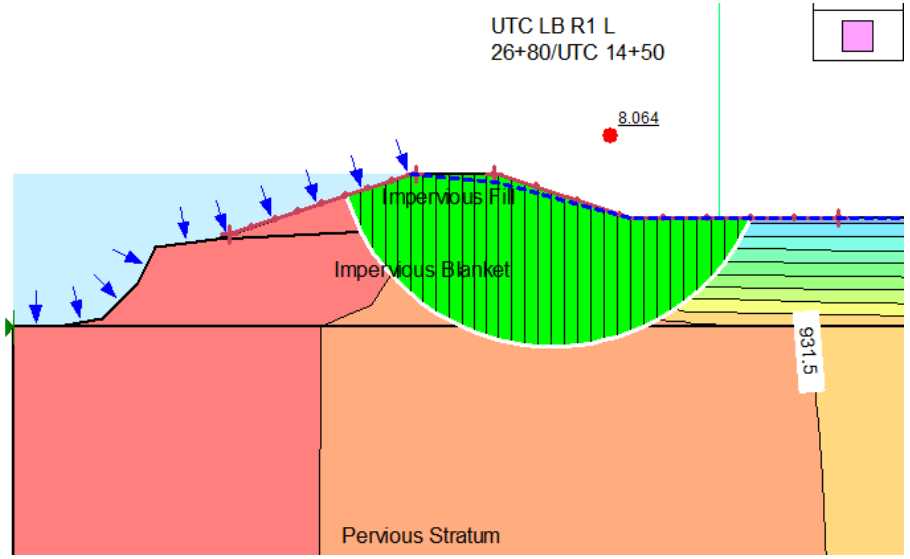




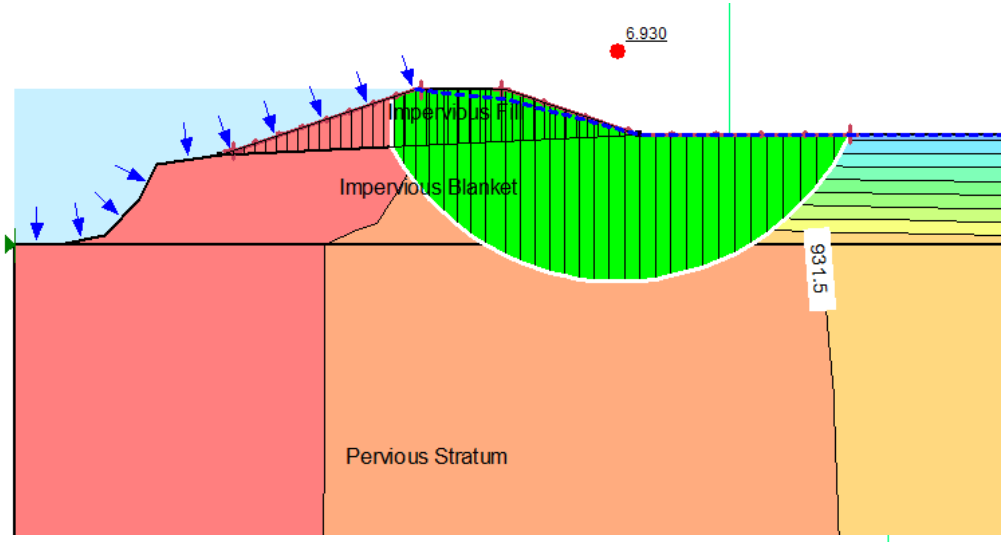


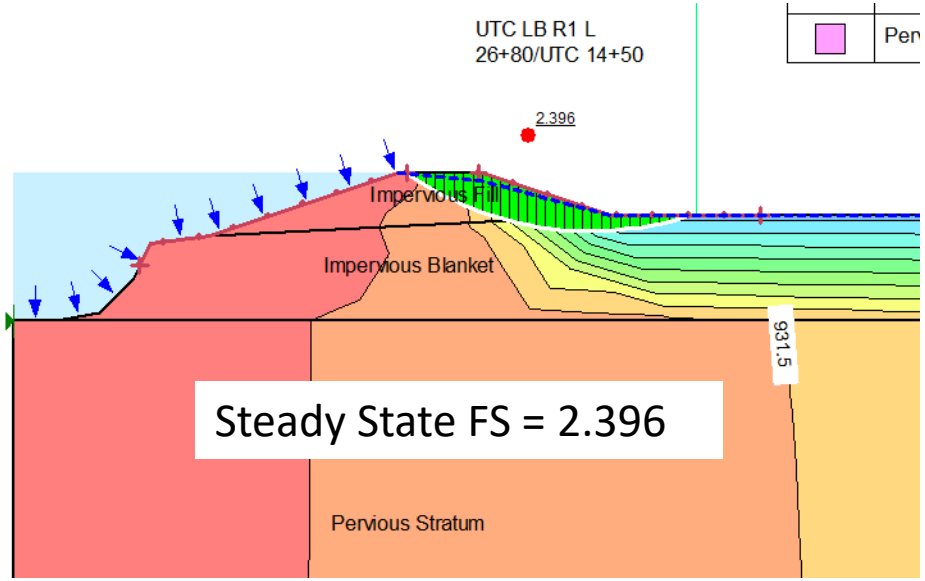
LB R1 26+80/UTC 14+50 ~1V:1.5H Analysis Results

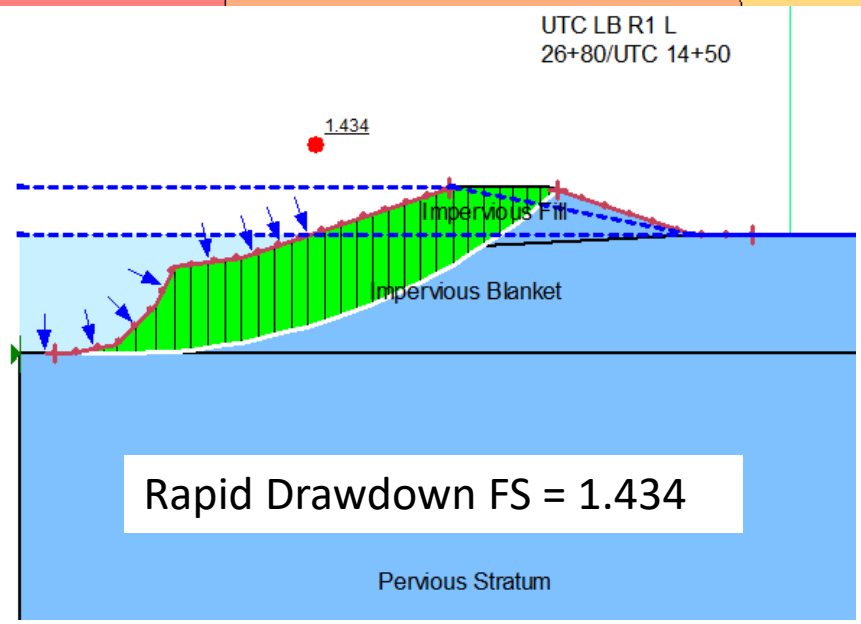
End of Construction FS = 8.064



End of Construction w Tension Crack FS = 6.930







RB R2 18+20/UTC 26+80



RB R2 18+20/UTC 26+80

Levee Unit Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYSIS TAB AND ASSOCIATED PL

Case Analyzed - Water at Levee Top

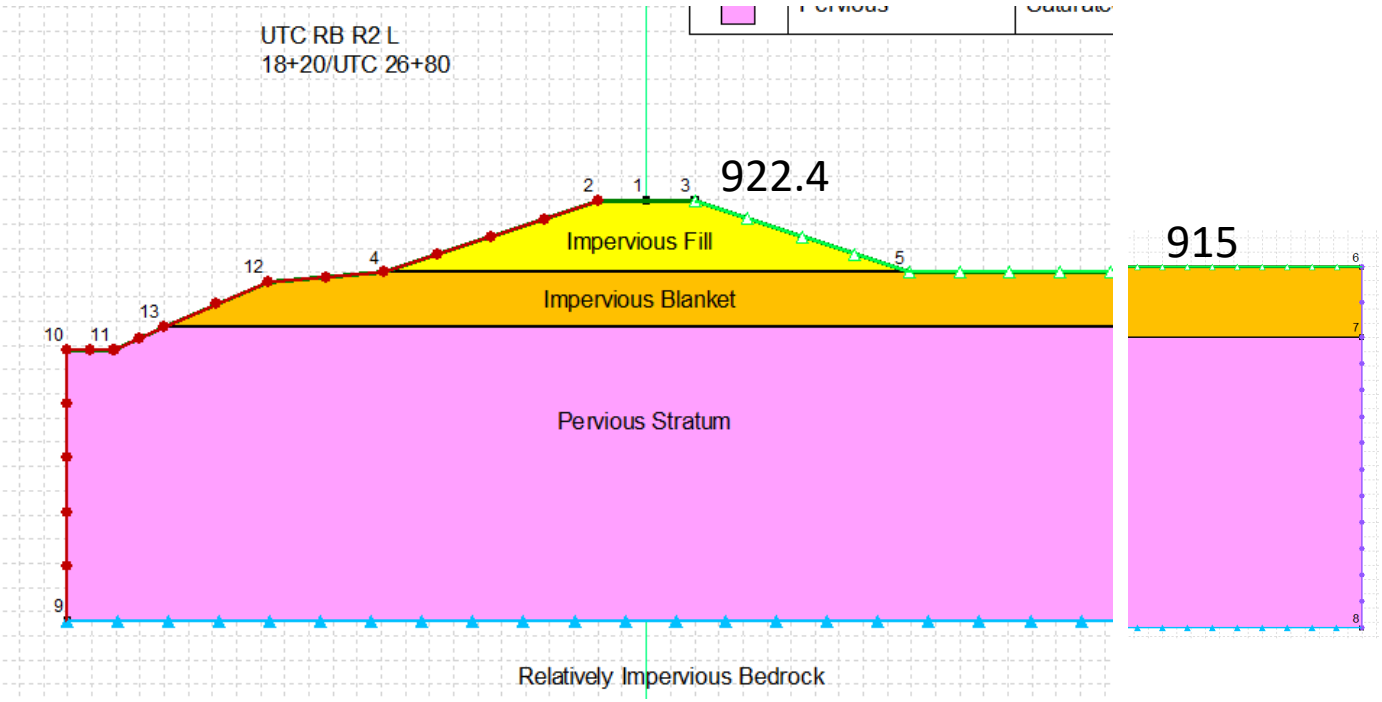
Blanket Unit Weight = 115.0 pcf (saturated)

	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Soil Boring	Land Blanket Bottom	Modified Top of Bedrock*	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Reference	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	Riverside	Levee	Land Side
Station	(msl)	(msl)	(msl)	(msl)	(msl)			(msl)	H	k_r/k_{br}	k_r/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
RB R2 L 18+20/UTC 26+80	922.4	914.5	914.5	914.5	910	AD-203	910	879	7.9	400	400	4.5	4.5	4.5	31	30	58	infinity

calibration

Factor		Effective Seepage Length (ft)		Head at	Head at	Computed Hydraulic	Critical Hydraulic	Factor of Safety for		
		River Side	Land Side	Toe (ft)	Toe (elevation msl)	Gradient	Gradient	Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe	FS horizontal gradient (i_{h-c} = 1/5 = 0.2)
c_r	c_l	x_1	x_3	h_o	h_o	i_o	i_c	(i_c/i_o)	(i_h)	(i_{h-c}/i_h)
0.004233	0.004233	29.84	236.22	5.76	920.26	1.28	0.84	0.66	0.09	2.22

RB R2 18+20/UTC 26+80



All Boundary Conditions

Name	Category	Hydraulic Boundary Type	Color
Bedrock - No Flow	Hydraulic	Water Rate	
Full Head	Hydraulic	Water Total Head	
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate	
Tailwater Head	Hydraulic	Water Total Head	

Name: Bedrock - No Flow

Kind: Water Rate

Boundary Condition Type

Constant: 0 ft³/sec

Function:

Name: Landside Slope - Potential Seepage Face

Kind: Water Rate

Boundary Condition Type

Constant: 0 ft³/sec

Function:

Potential Seepage Face Review

Name: Full Head

Kind: Water Total Head

Boundary Condition Type

Constant: 922.4 ft

Function:

Name: Tailwater Head

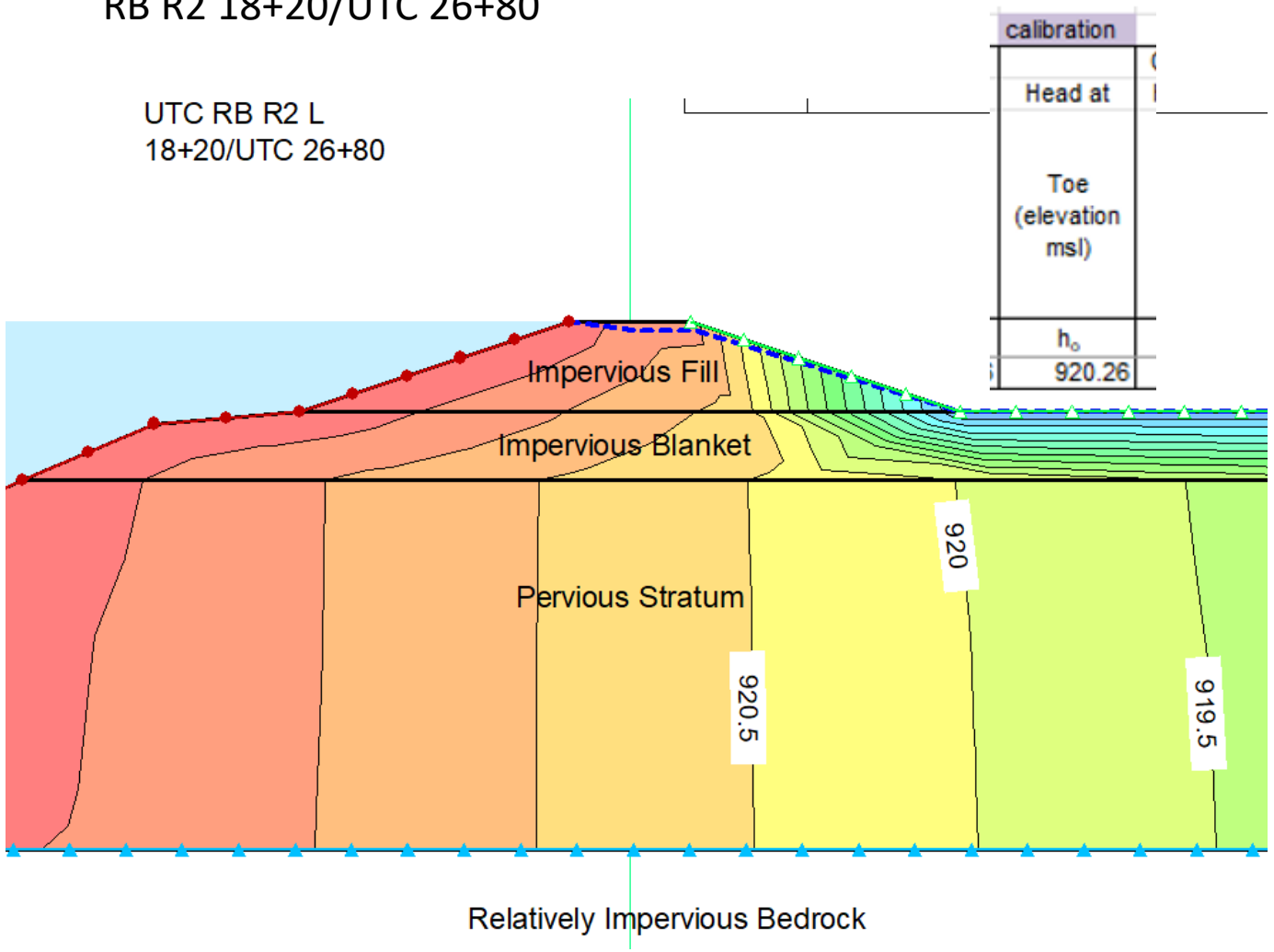
Kind: Water Total Head

Boundary Condition Type

Constant: 915 ft

Function:

RB R2 18+20/UTC 26+80



Name: Impervious Blanket (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	1.5e-05 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

Name: Pervious

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	0.003 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

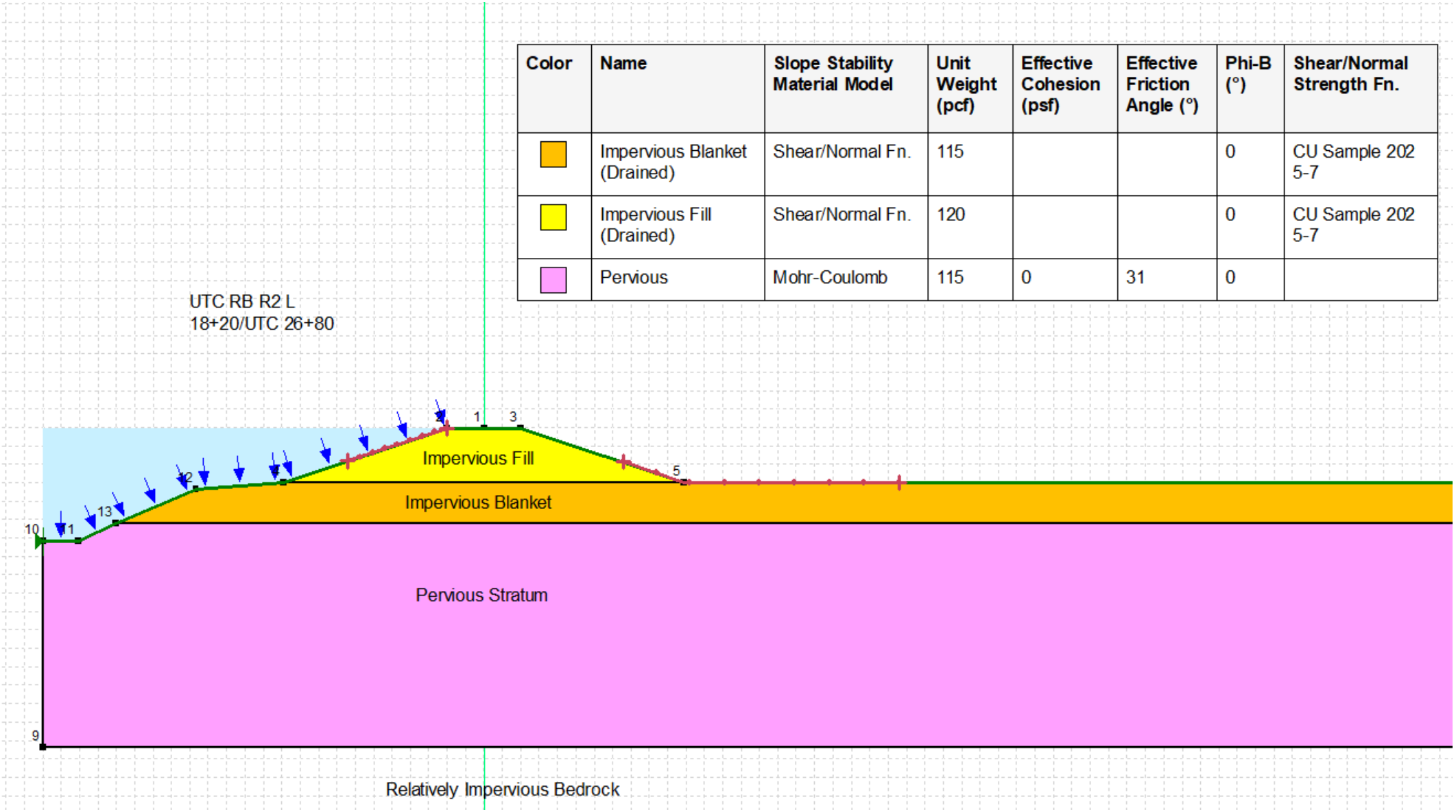
Name: Impervious Fill (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	1.5e-05 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

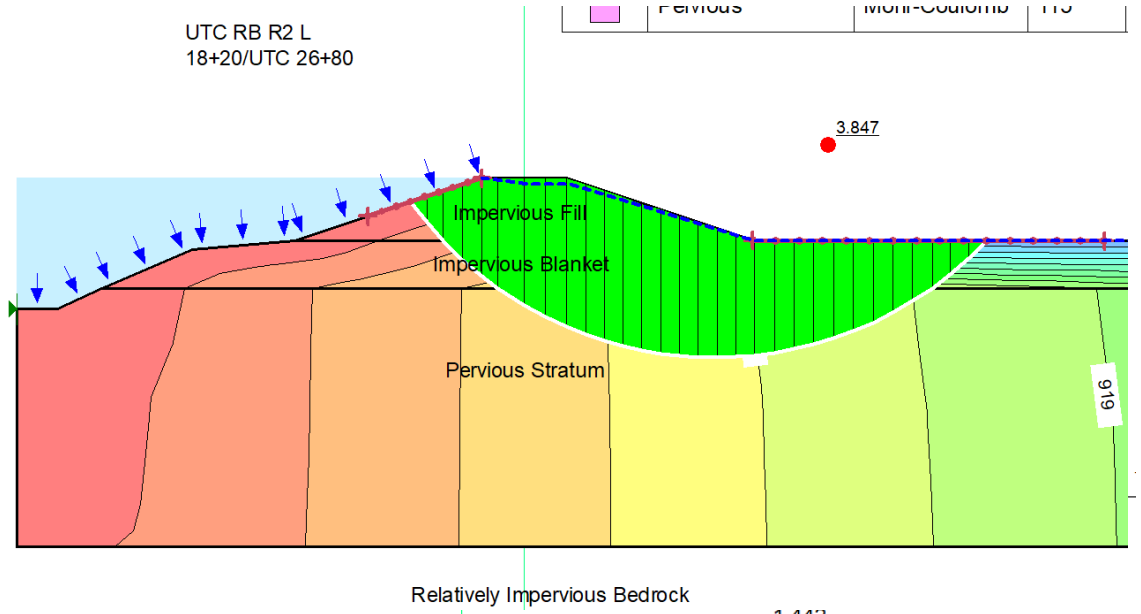
RB R2 18+20/UTC 26+80

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Shear/Normal Strength Fn.
	Impervious Blanket (Drained)	Shear/Normal Fn.	115			0	CU Sample 202 5-7
	Impervious Fill (Drained)	Shear/Normal Fn.	120			0	CU Sample 202 5-7
	Pervious	Mohr-Coulomb	115	0	31	0	

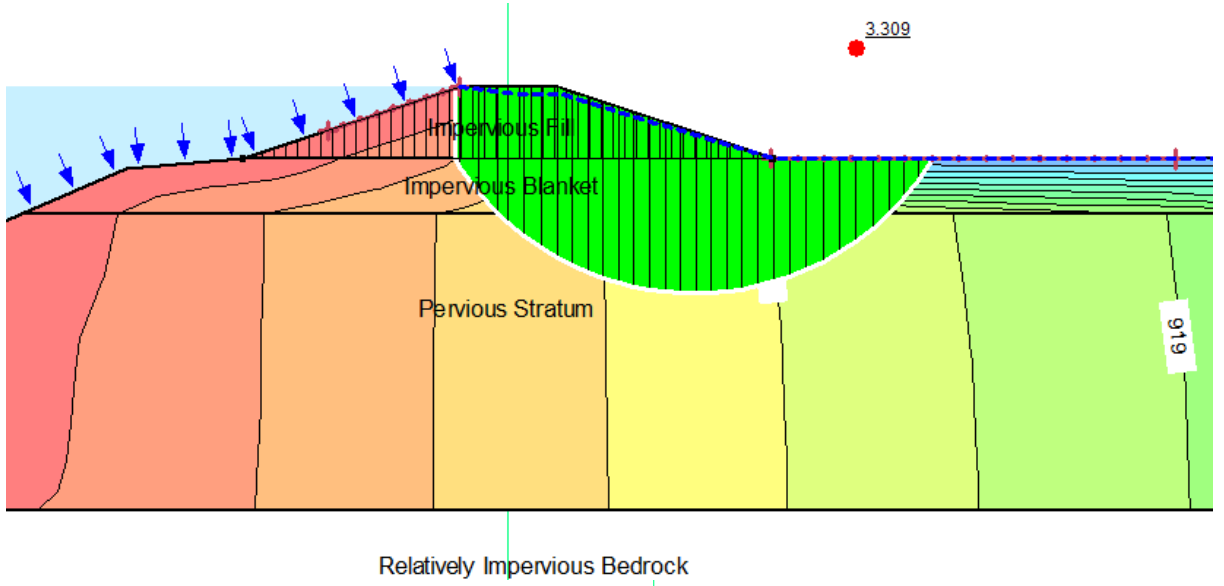


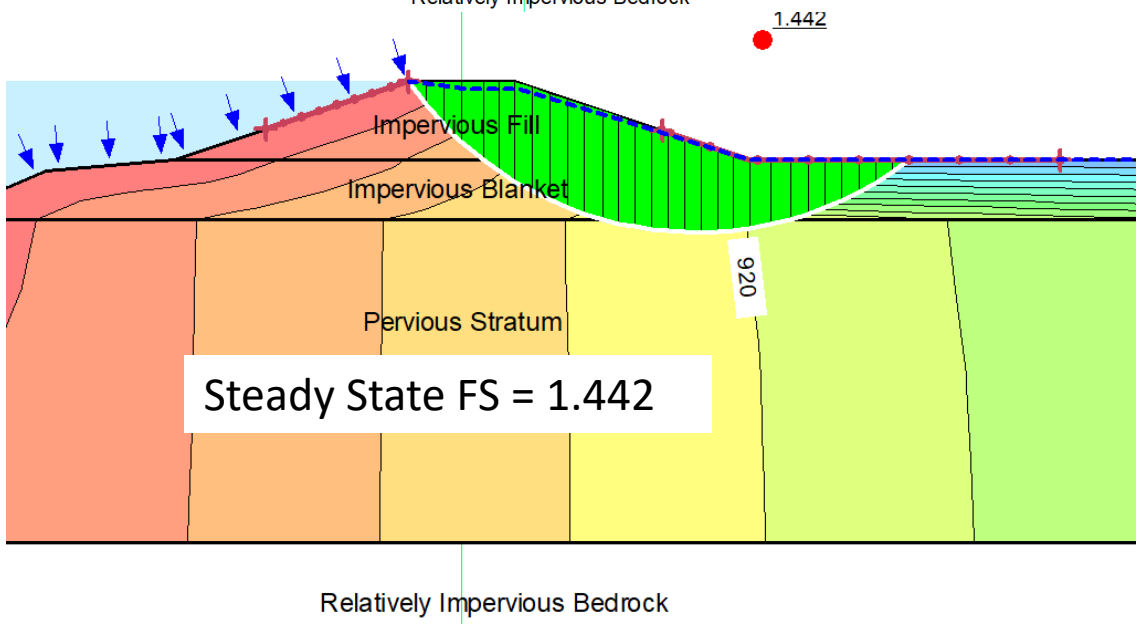
RB R2 18+20/UTC 26+80 ~1V:3H Analysis Results

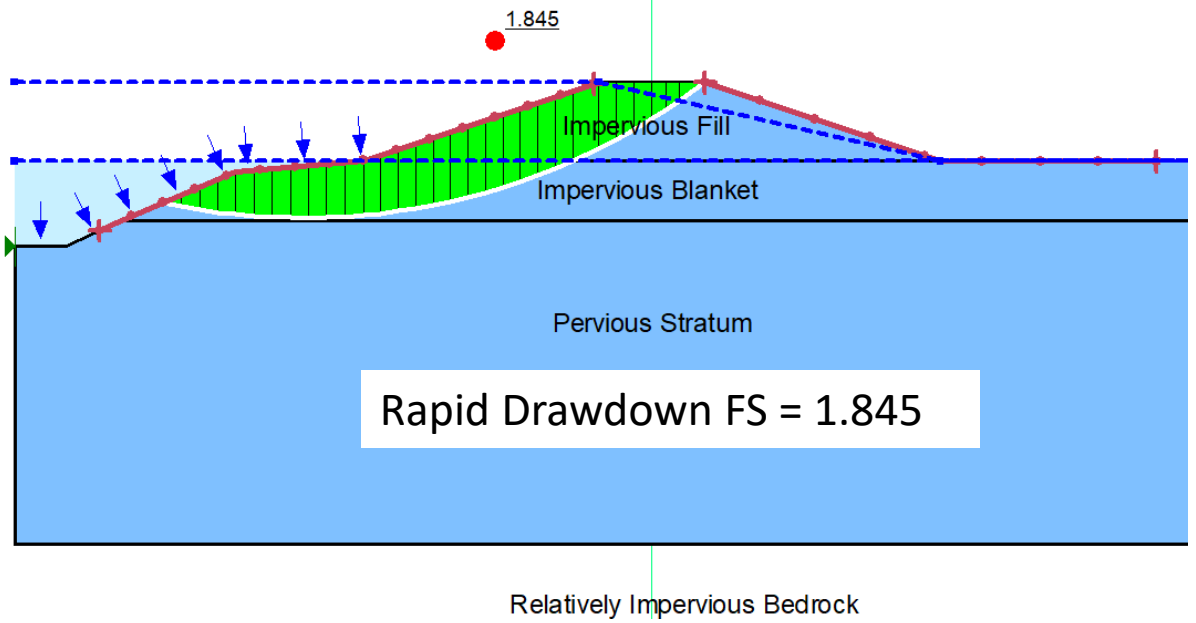
End of Construction FS = 3.847



End of Construction w Tension Crack FS = 3.309

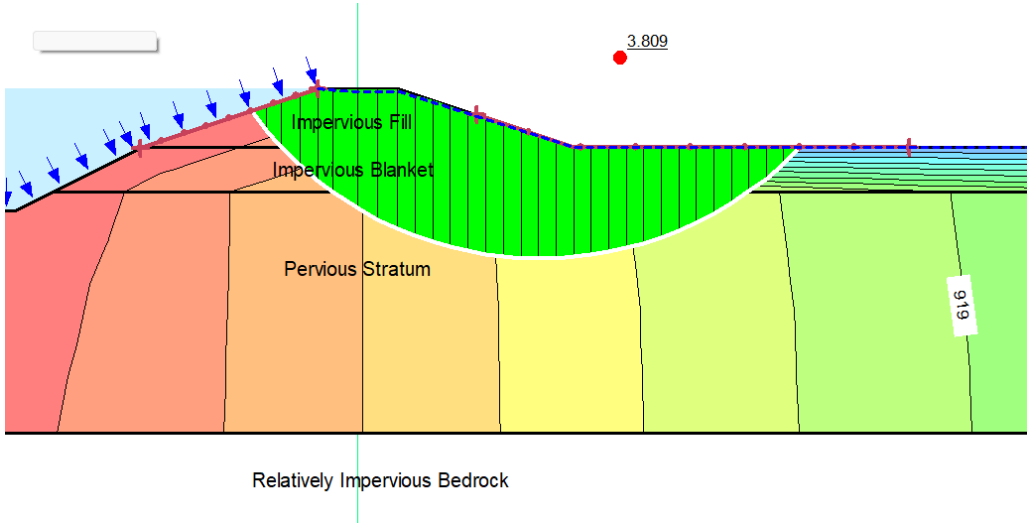




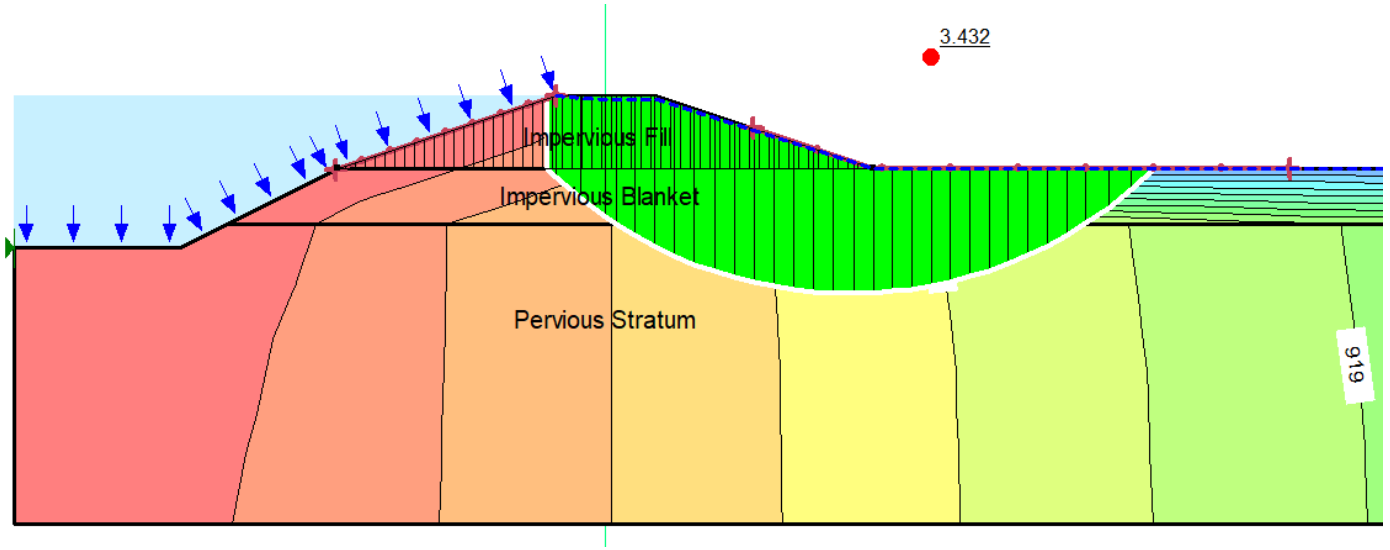


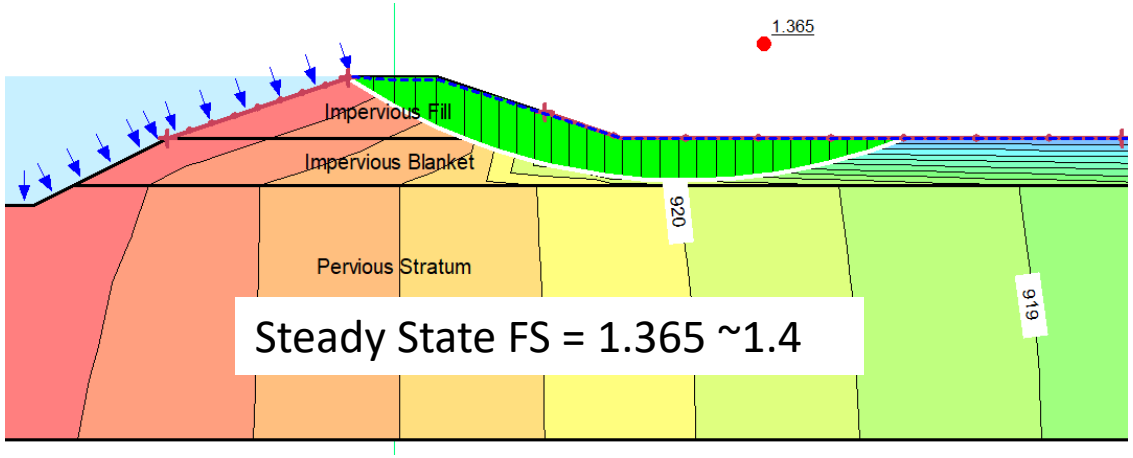
RB R2 18+20/UTC 26+80 ~1V:2H Analysis Results

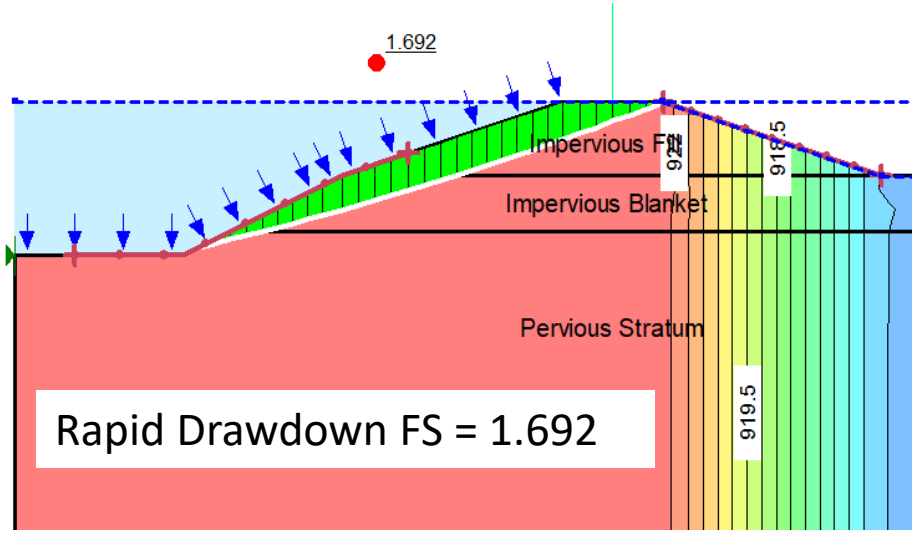
End of Construction FS = 3.809



End of Construction w Tension Crack FS = 3.432

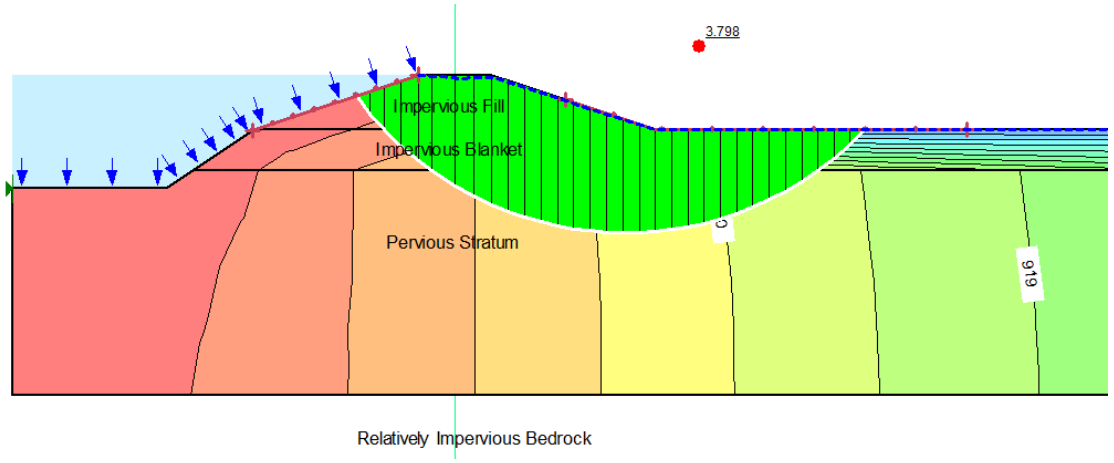




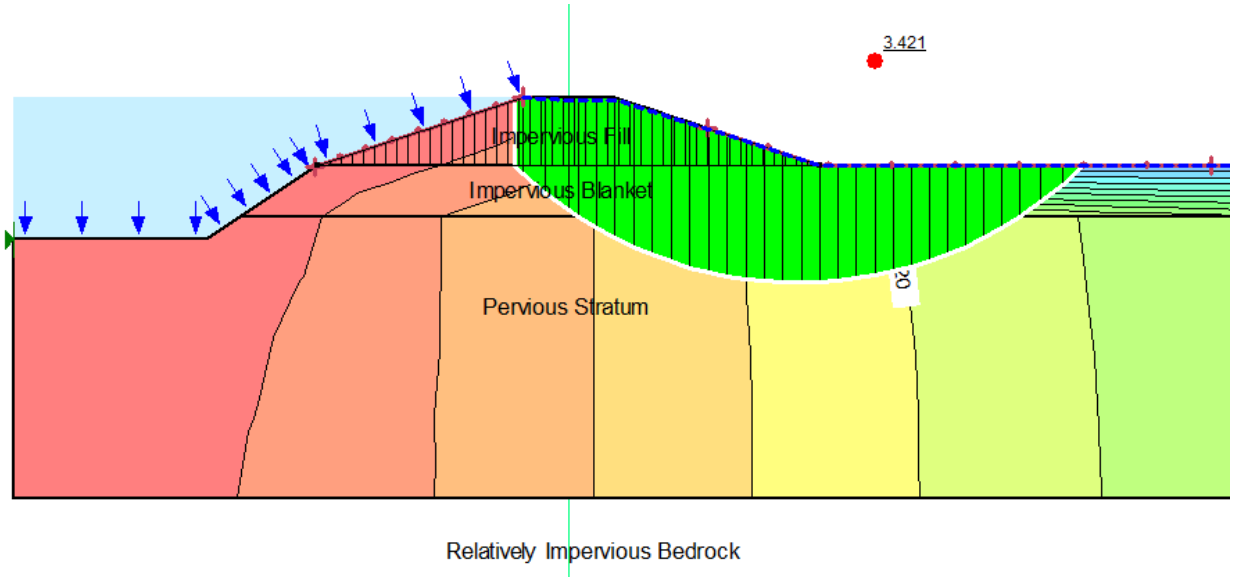


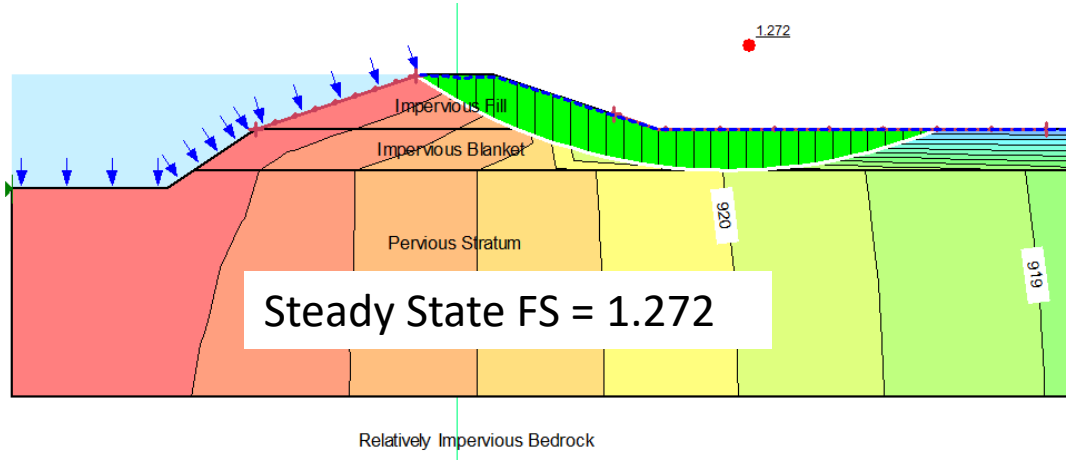
RB R2 18+20/UTC 26+80 ~1V:1.5H Analysis Results

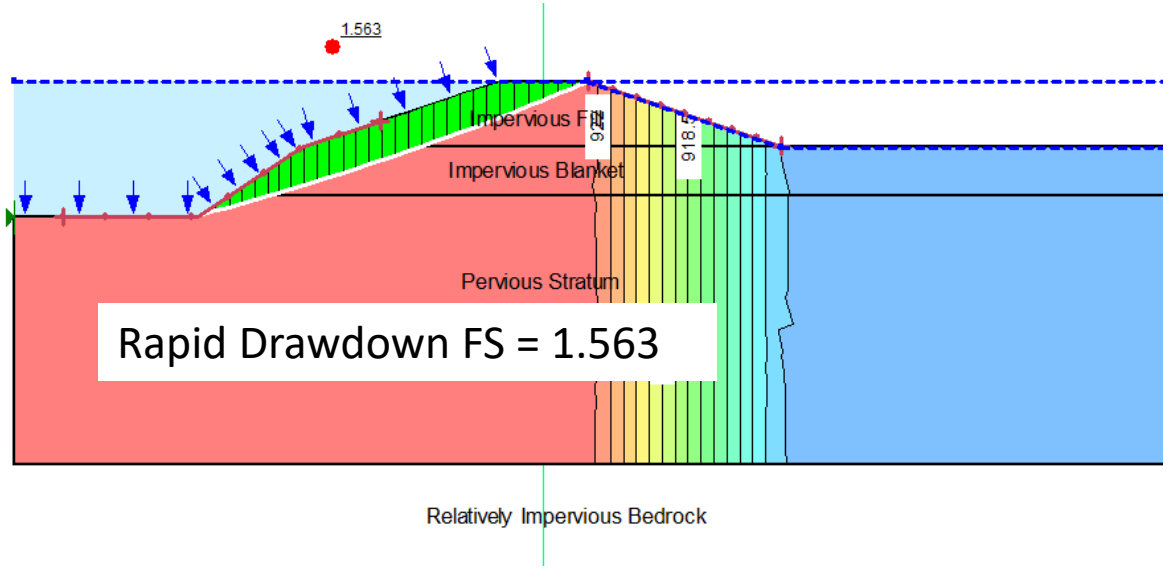
End of Construction FS = 3.798



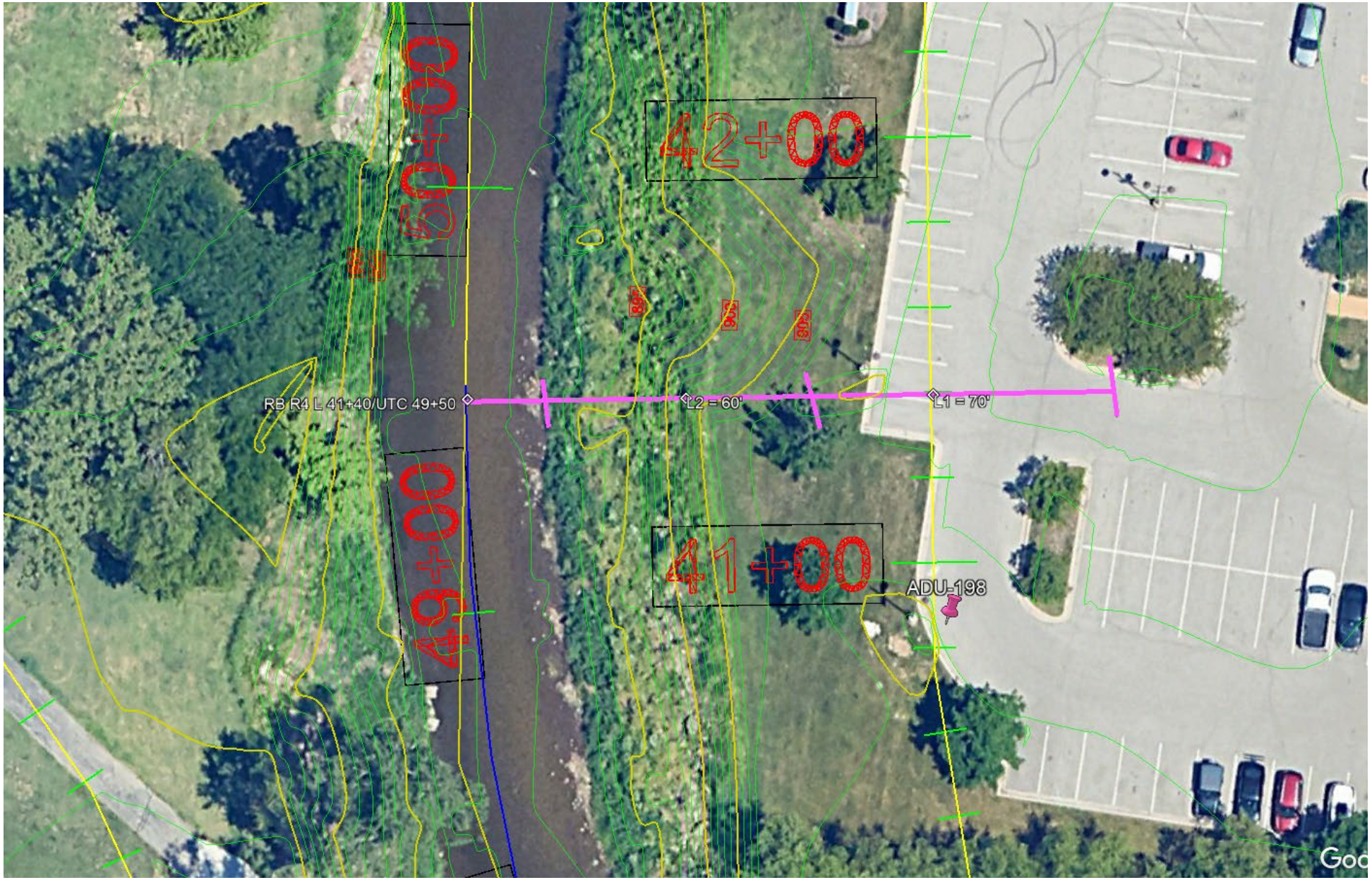
End of Construction w Tension Crack FS = 3.421







RB R4 41+40/UTC 49+50



RB R4 41+40/UTC 49+50

Levee Unit Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYSIS TAB AND ASSOCIATED PLC

Case Analyzed - Water at Levee Top

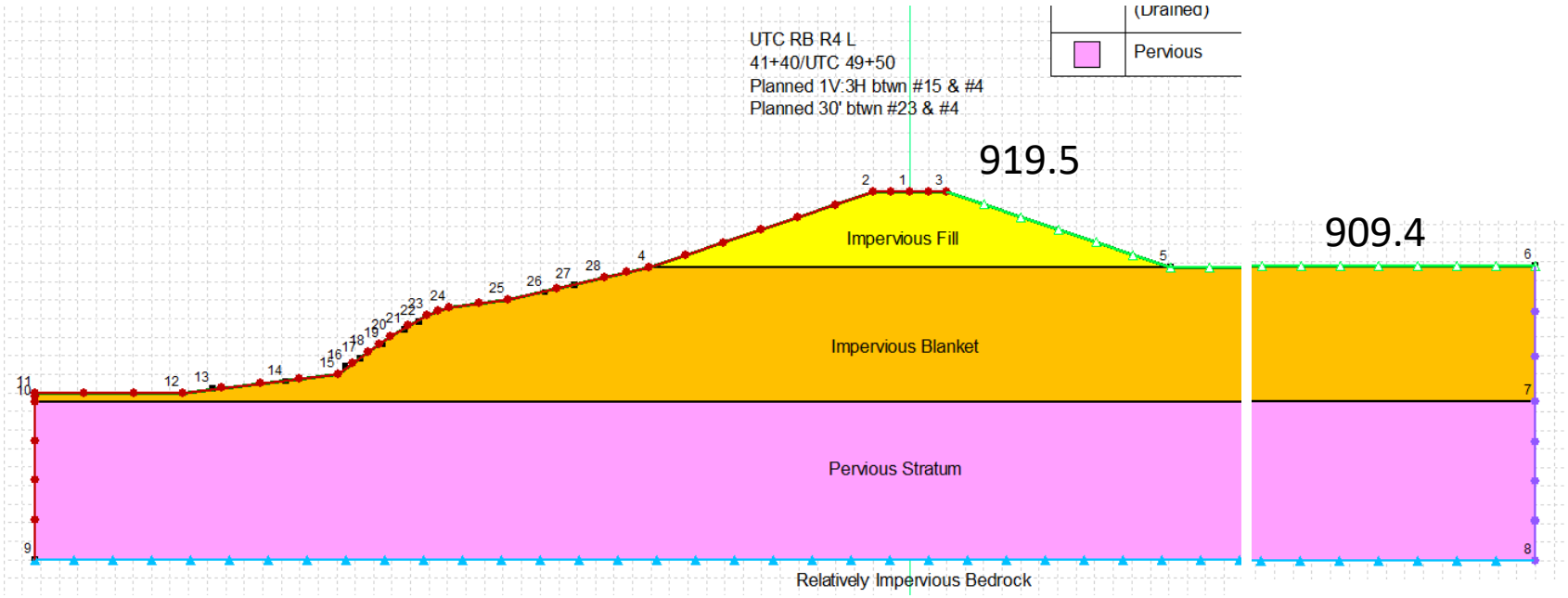
Blanket Unit Weight = 115.0 pcf (saturated)

	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Soil Boring	Land Blanket Bottom	Modified Top of Bedrock*	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Reference	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	Riverside	Levee	Land Side
Station	(msl)	(msl)	(msl)	(msl)	(msl)			(msl)	H	k_r/k_{br}	k_r/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
RB R4 L 41+40/UTC 49+50	917.7	907.6	907.6	907.6	891	avg 198 & 197	891	850	10.1	400	400	16.6	16.6	16.6	41	60	70	infinity

calibration

Factor		Effective Seepage Length (ft)		Head at	Head at	Computed Hydraulic	Critical Hydraulic	Factor of Safety for		
		River Side	Land Side	Toe (ft)	Toe (elevation msl)	Gradient	Gradient	Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe	FS horizontal gradient ($i_{h,c}$ = 1/5 = 0.2)
c_r	c_l	x_1	x_3	h_o	h_o	i_o	i_c	(i_c/i_o)	(i_h)	$(i_{h,c}/i_h)$
0.001917	0.001917	59.74	521.77	8.09	915.69	0.49	0.84	1.73	0.08	2.57

RB R4 41+40/UTC 49+50



Name: Bedrock - No Flow
Kind: Water Rate
Boundary Condition Type
Constant: 0 ft³/sec
Function:

Name: Full Head
Kind: Water Total Head
Boundary Condition Type
Constant: 919.5 ft
Function:

Name: Landside Slope - Potential Seepage Face
Kind: Water Rate
Boundary Condition Type
Constant: 0 ft³/sec
Function:
Potential Seepage Face Review

Name: Tailwater Head
Kind: Water Total Head
Boundary Condition Type
Constant: 909.4 ft
Function:

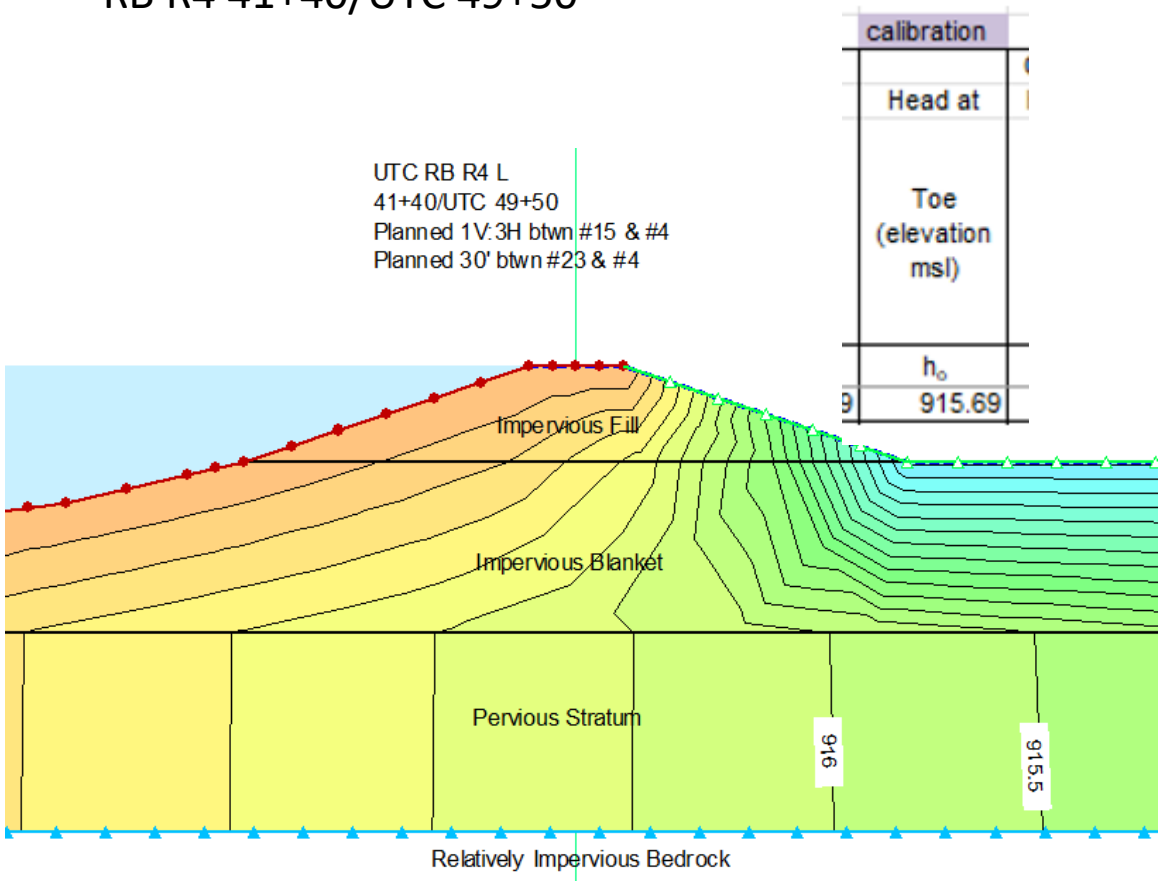
Define Boundary Conditions

BC Category: All

All Boundary Conditions

Name	Category	Hydraulic Boundary Type	Color
Bedrock - No Flow	Hydraulic	Water Rate	
Full Head	Hydraulic	Water Total Head	
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate	
Tailwater Head	Hydraulic	Water Total Head	

RB R4 41+40/UTC 49+50



Name:
Impervious Blanket (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	1.5e-05 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

Name:
Pervious

Hydraulic

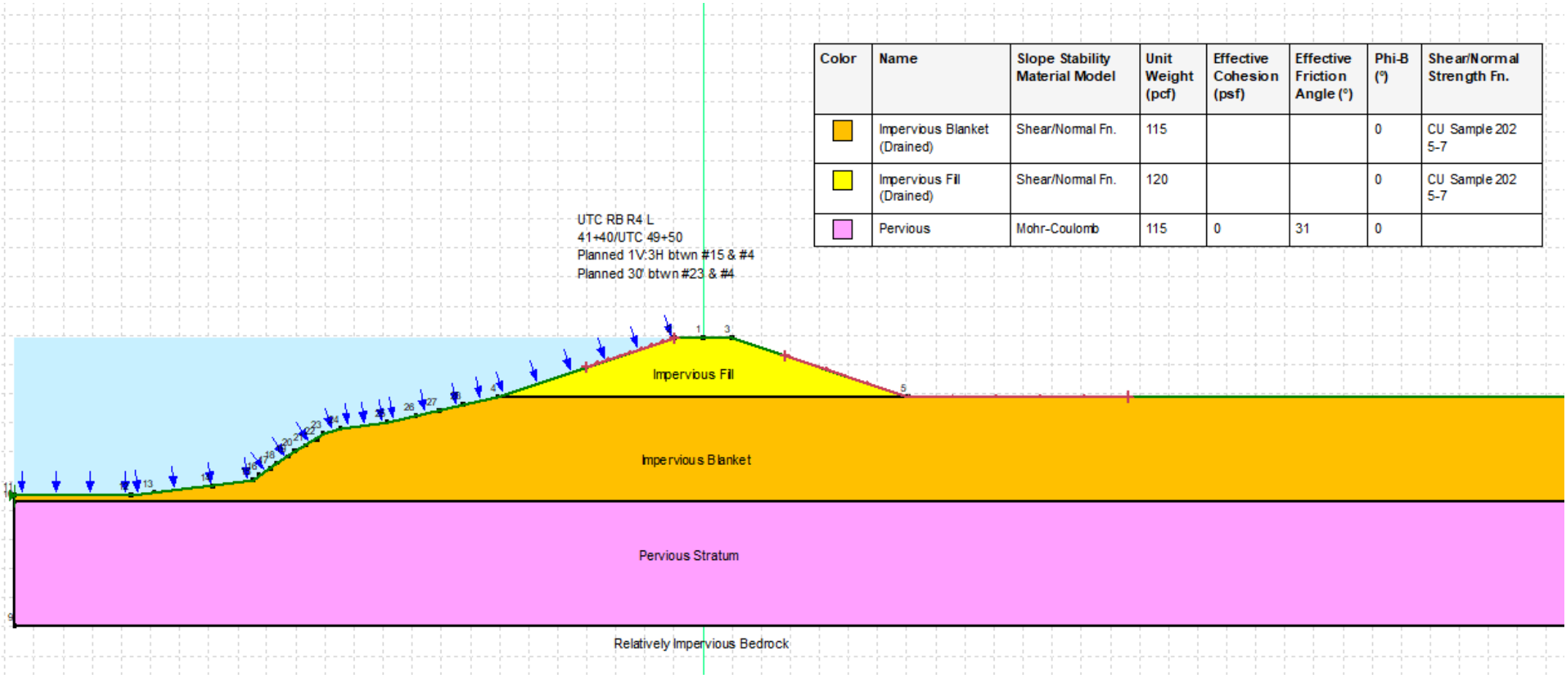
Material Model:	Saturated Only
Saturated X-Conductivity:	0.003 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

Name:
Impervious Fill (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	1.5e-05 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

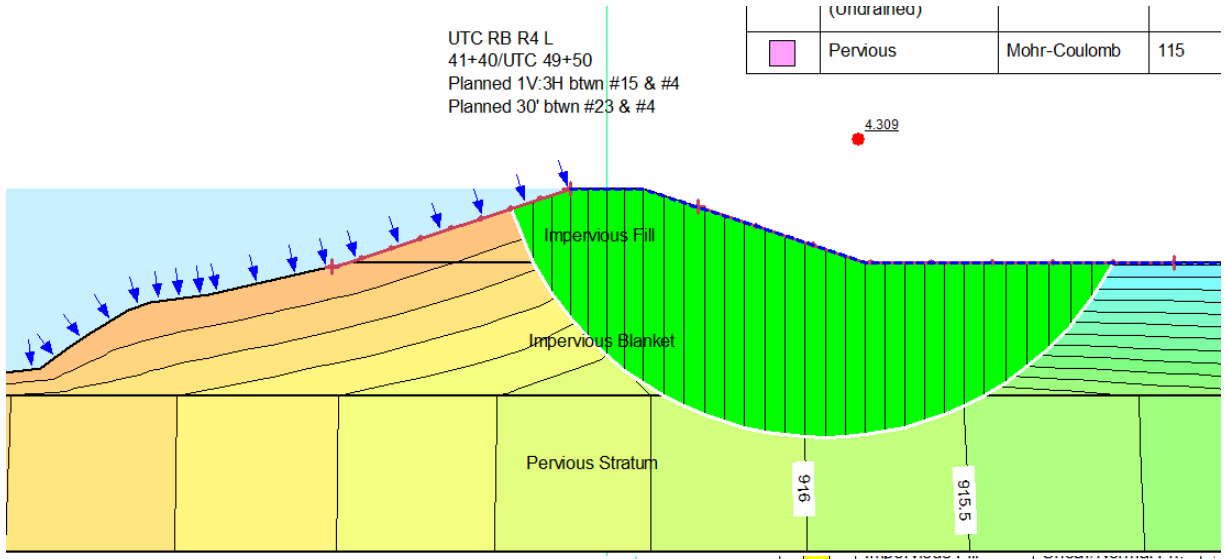
RB R4 41+40/UTC 49+50



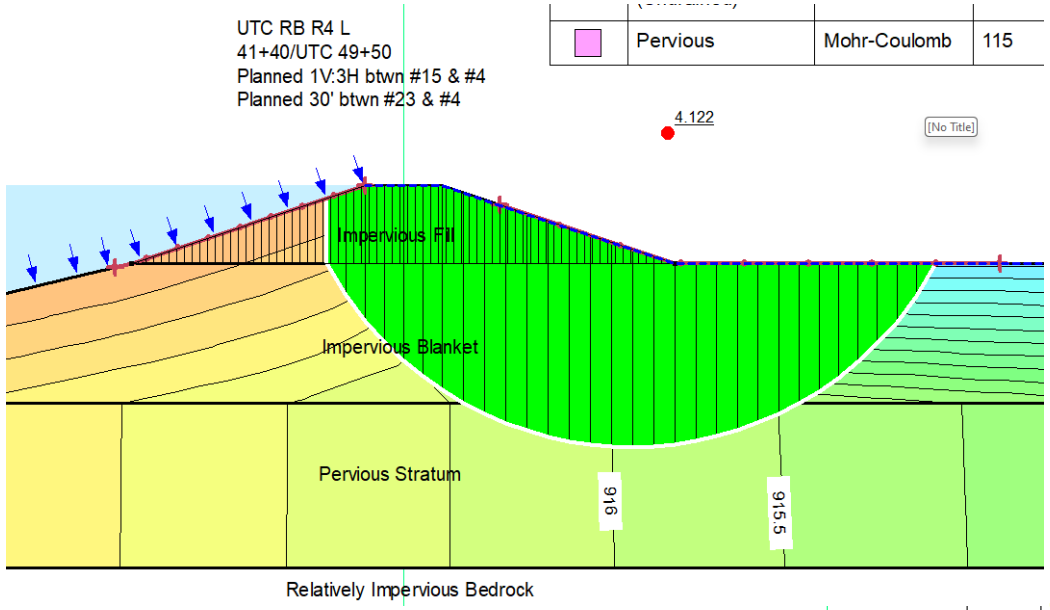
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Shear/Normal Strength Fn.
	Impervious Blanket (Drained)	Shear/Normal Fn.	115			0	CU Sample 202 5-7
	Impervious Fil (Drained)	Shear/Normal Fn.	120			0	CU Sample 202 5-7
	Pervious	Mohr-Coulomb	115	0	31	0	

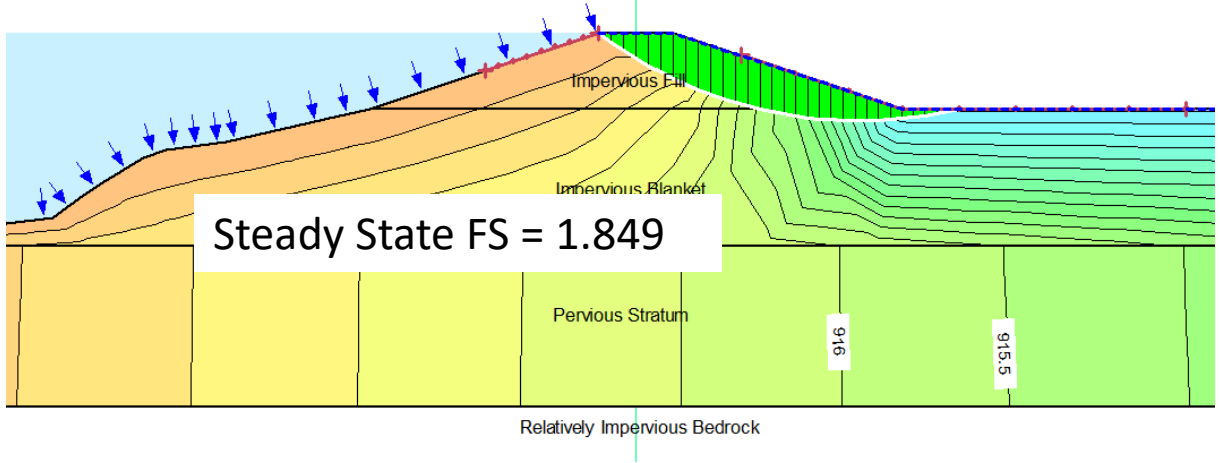
RB R4 41+40/UTC 49+50 ~1V:3H Analysis Results

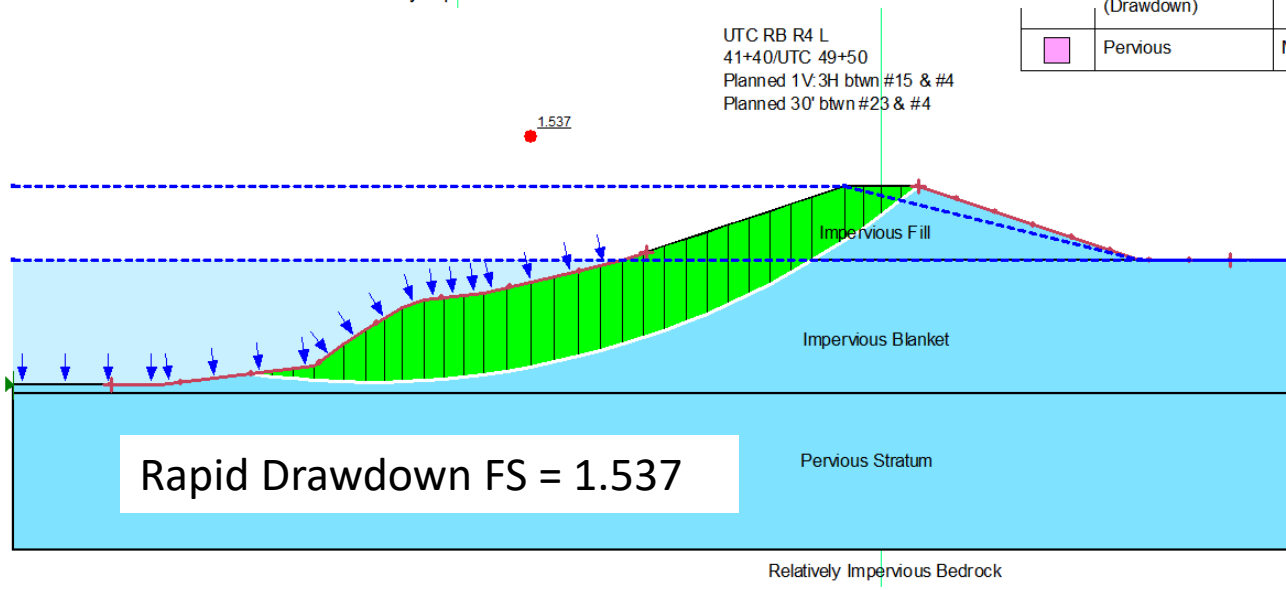
End of Construction FS = 4.309



End of Construction w Tension Crack FS = 4.122

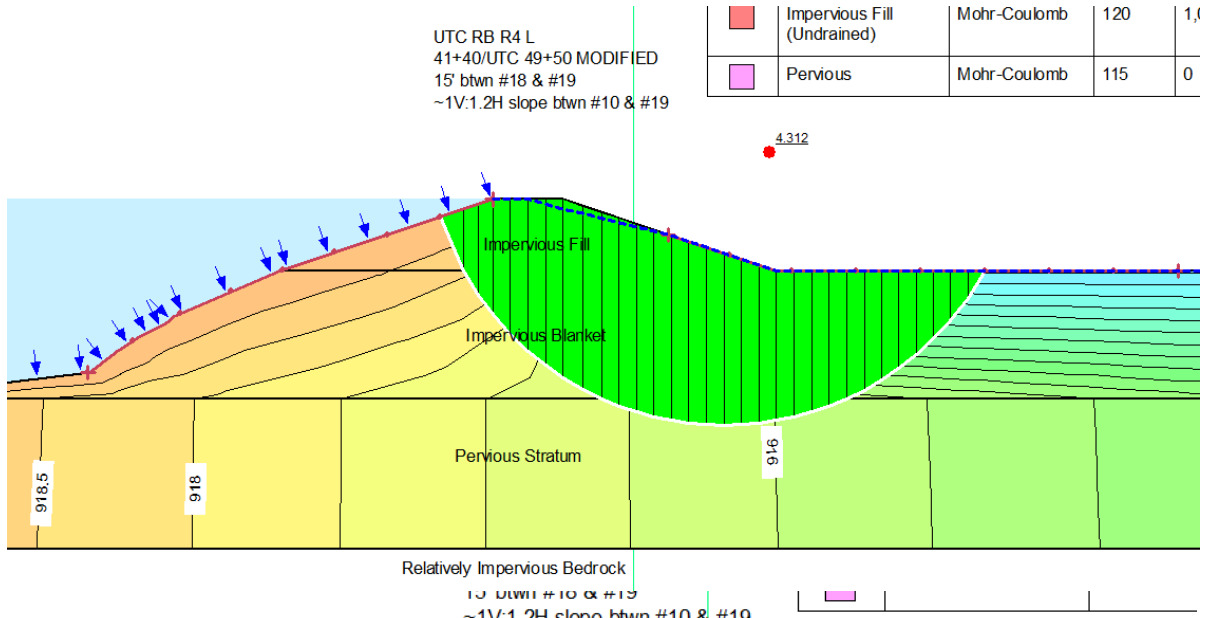




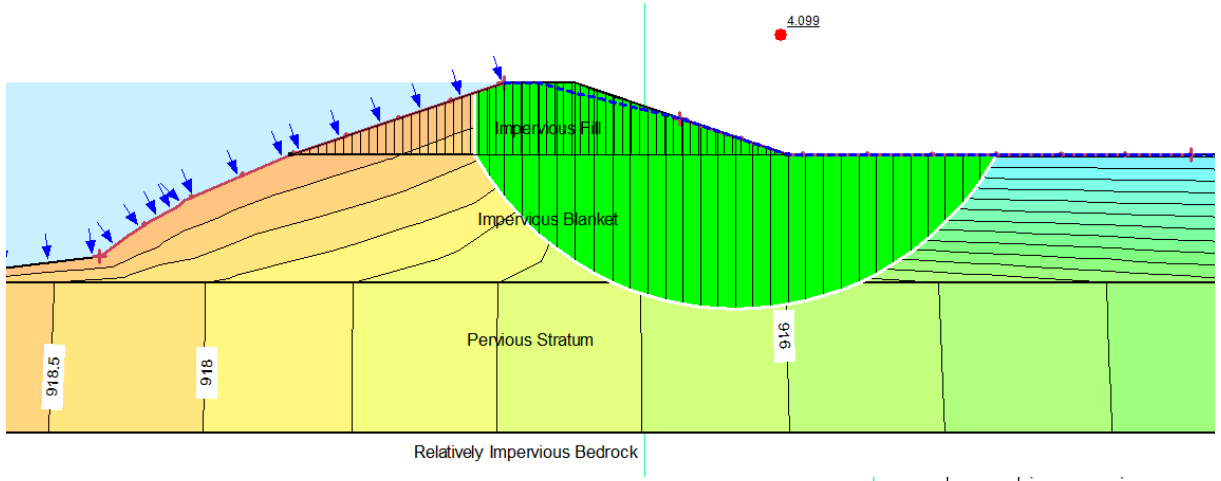


RB R4 41+40/UTC 49+50 ~1V:2H Analysis Results

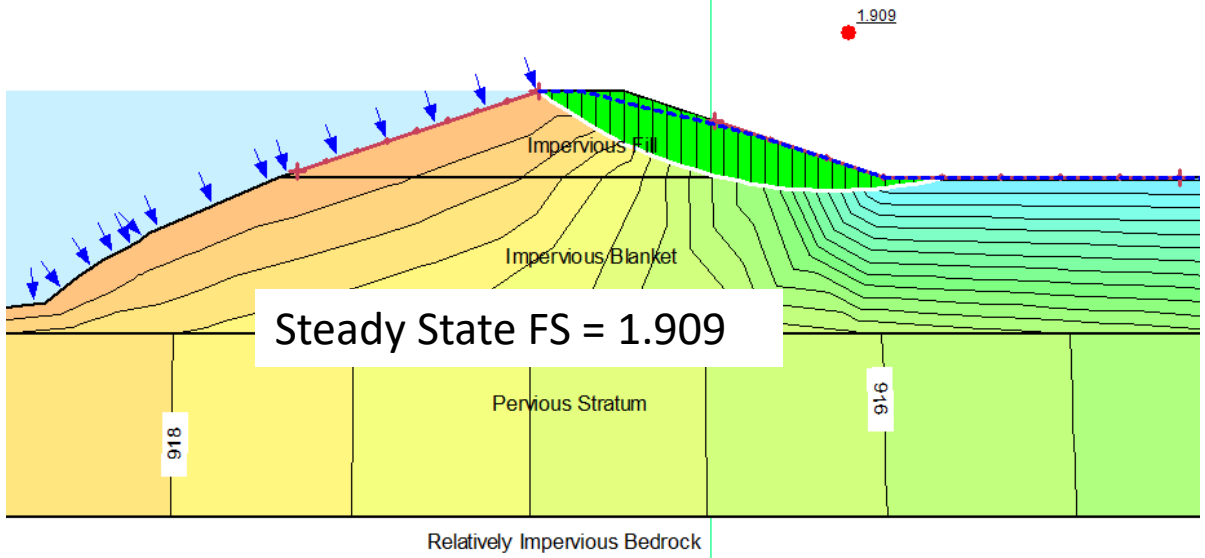
End of Construction FS = 4.312



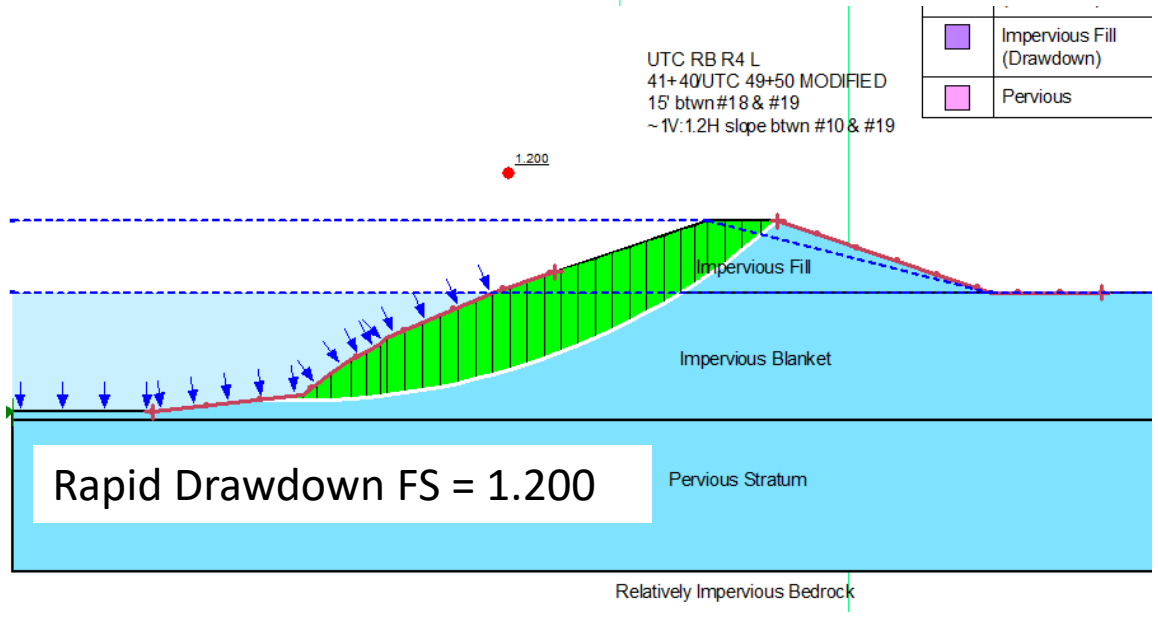
End of Construction w Tension Crack FS = 4.099



Steady State FS = 1.909

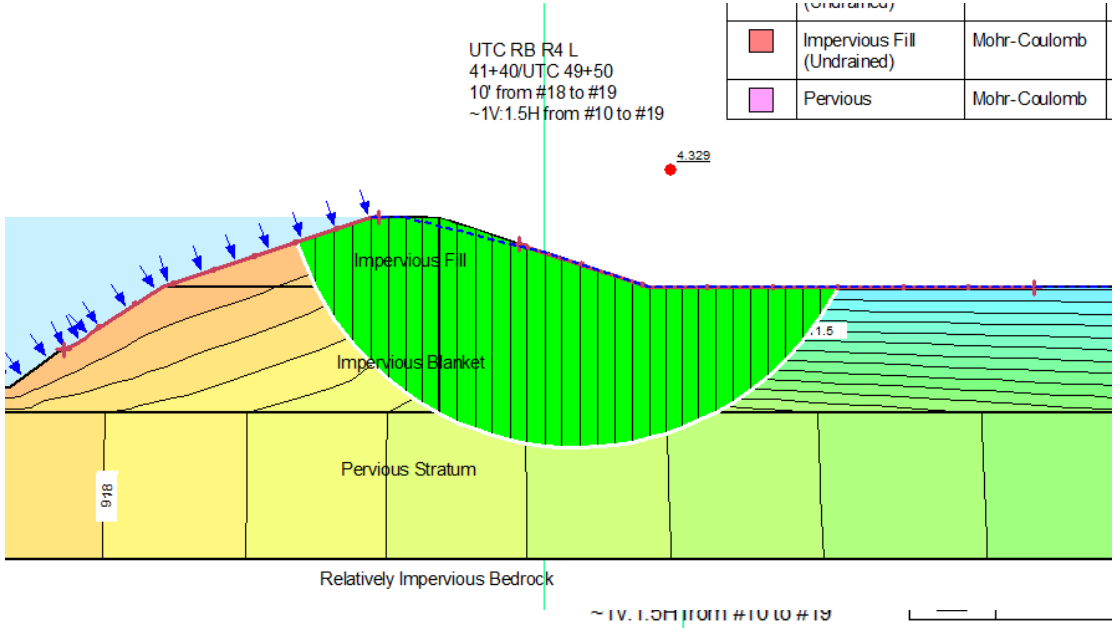


Rapid Drawdown FS = 1.200

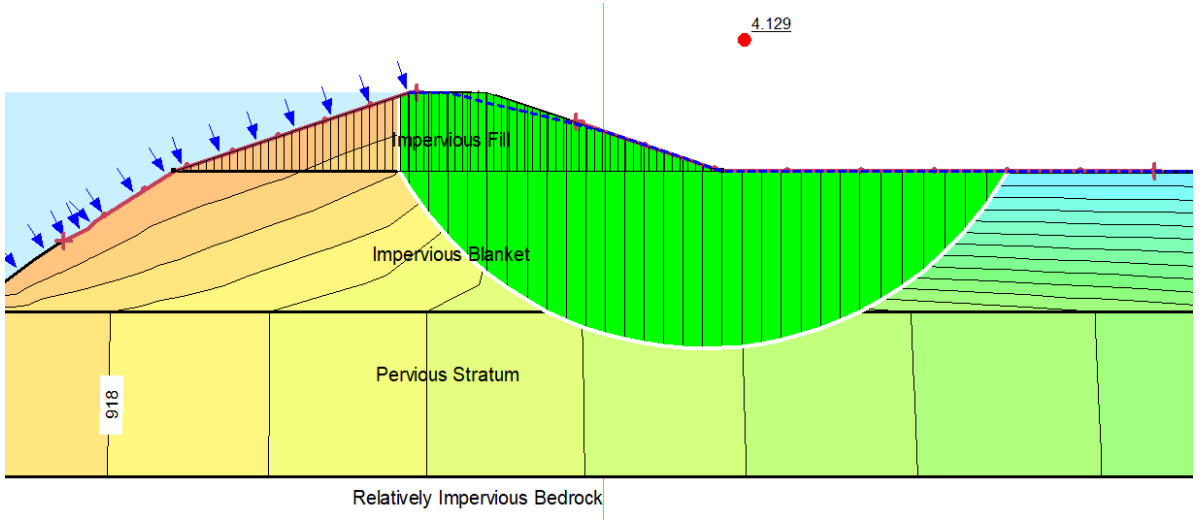


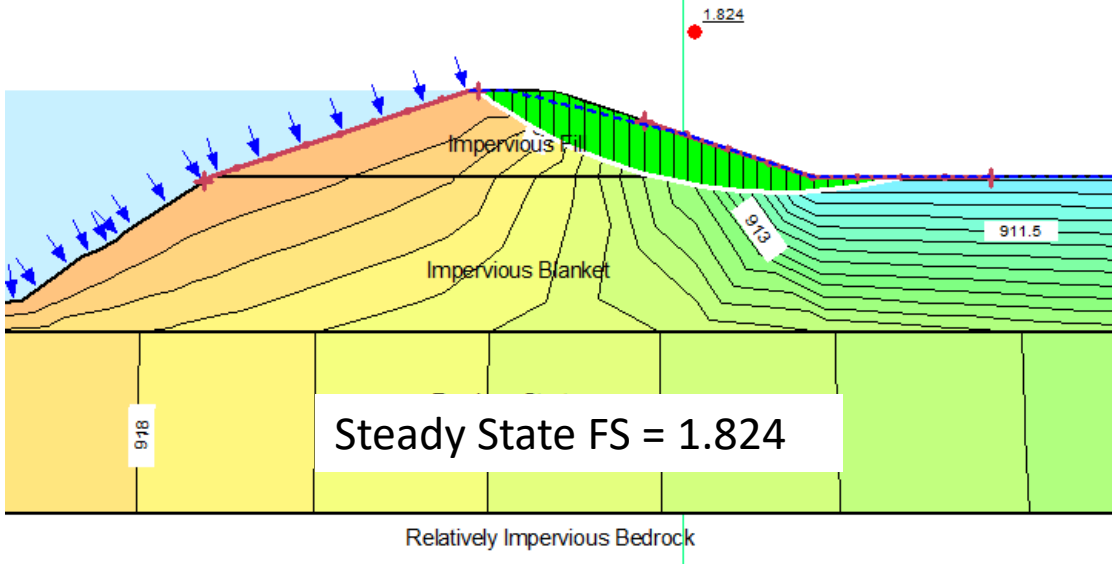
RB R4 41+40/UTC 49+50 ~1V:1.5H Analysis Results

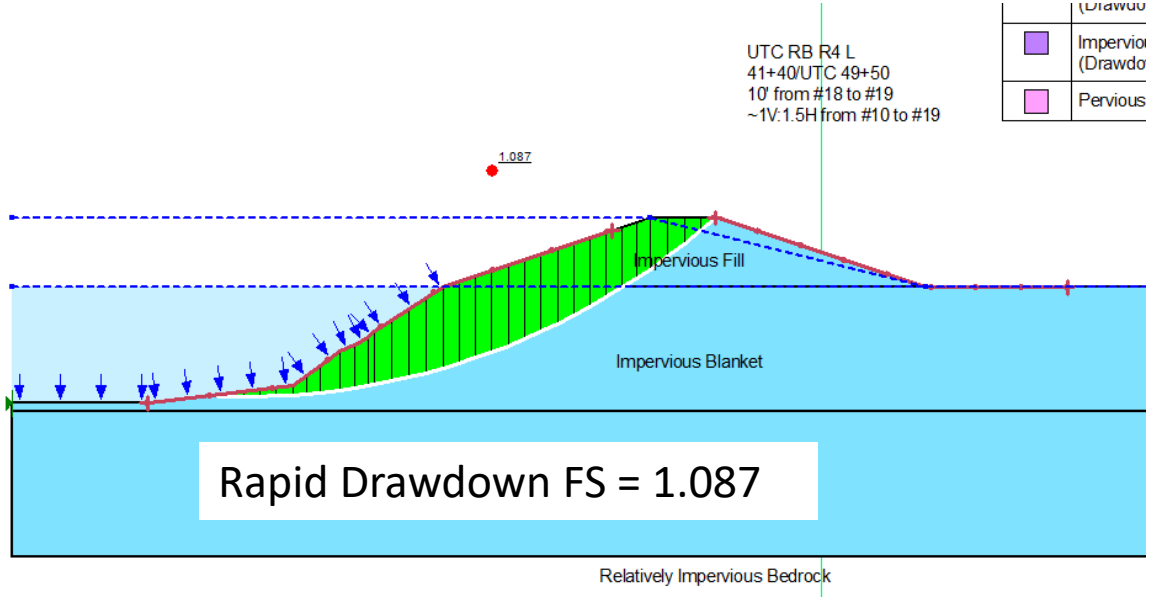
End of Construction FS = 4.329



End of Construction w Tension Crack FS = 4.129





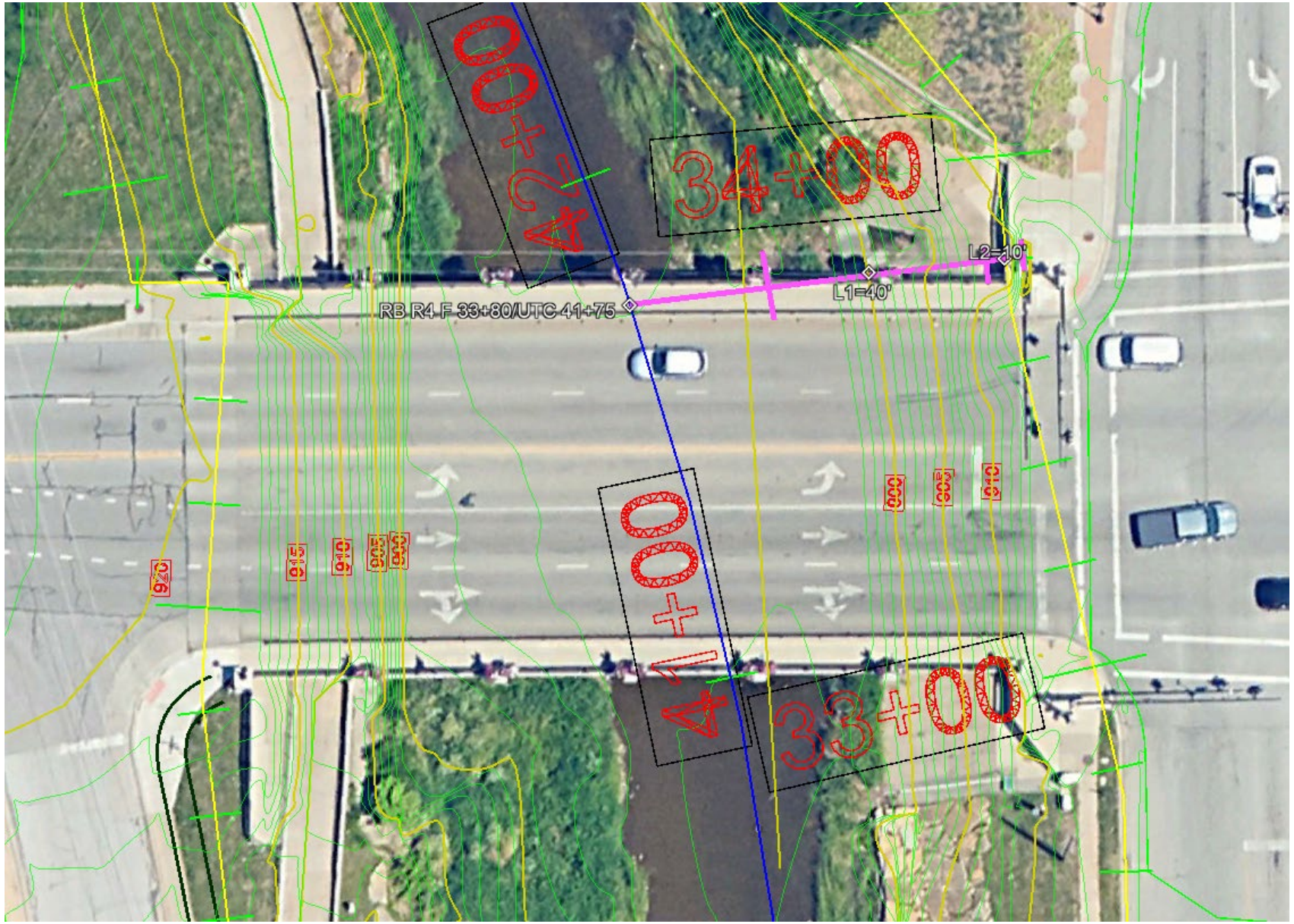


Floodwalls

Performed calcs for two representative levee sections:
LB R1 F 13+20/UTC 0+50
RB R4 F 33+80UTC 41+75

Detailed example provided for RB R4 F 33+80UTC 41+75
Major inputs and all results provided for other sections
(See Google Earth file, underseepage spreadsheet, and GeoStudio files in project file for additional information)

RB F 33+80/UTC 41+75



L2 = 10’ footing width, centered on levee centerline

L1 = distance from riverside edge of footing to Turkey Creek

Slope cross section based on survey topo and distances on Google Earth

RB F 33+80/UTC 41+75

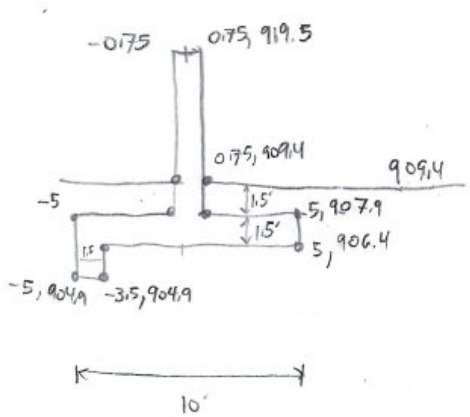
Underseepage Calcs

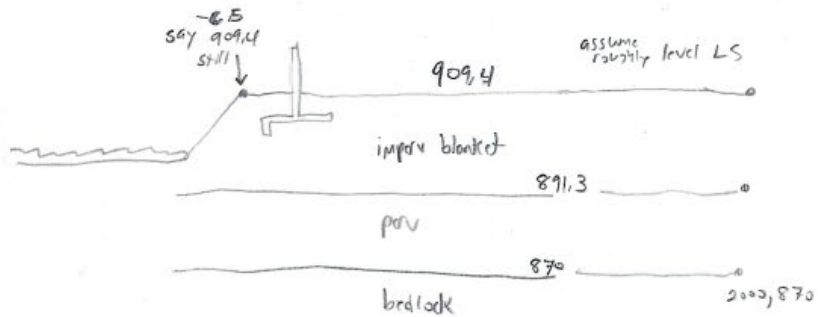
	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Land Blanket Bottom	Top of Bedrock	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	River Side	Levee	Land Side
Station	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	(NAVD88)	H	k_f/k_{br}	k_f/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
RB R4 F 33+80/UTC 41+75	919.5	909.4	909.4	909.4	891.3	891.3	870	10.1	400	400	18.1	18.1	18.1	21.3	40	10	infinity

	Head at Toe (ft)	Head at Toe (elevation msl)	Computed Hydraulic Gradient	Critical Hydraulic Gradient	Factor of Safety for Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe
Station	h_o	h_o	i_o	i_c	(i_c/i_o)	(i_h)
RB R4 F 33+80/UTC 41+75	8.96	918.36	0.50	0.84	1.70	0.20

RB F 33+80/UTC 41+75 Developing Cross Section

UTC RB R4 33+80 / UTC 41+75



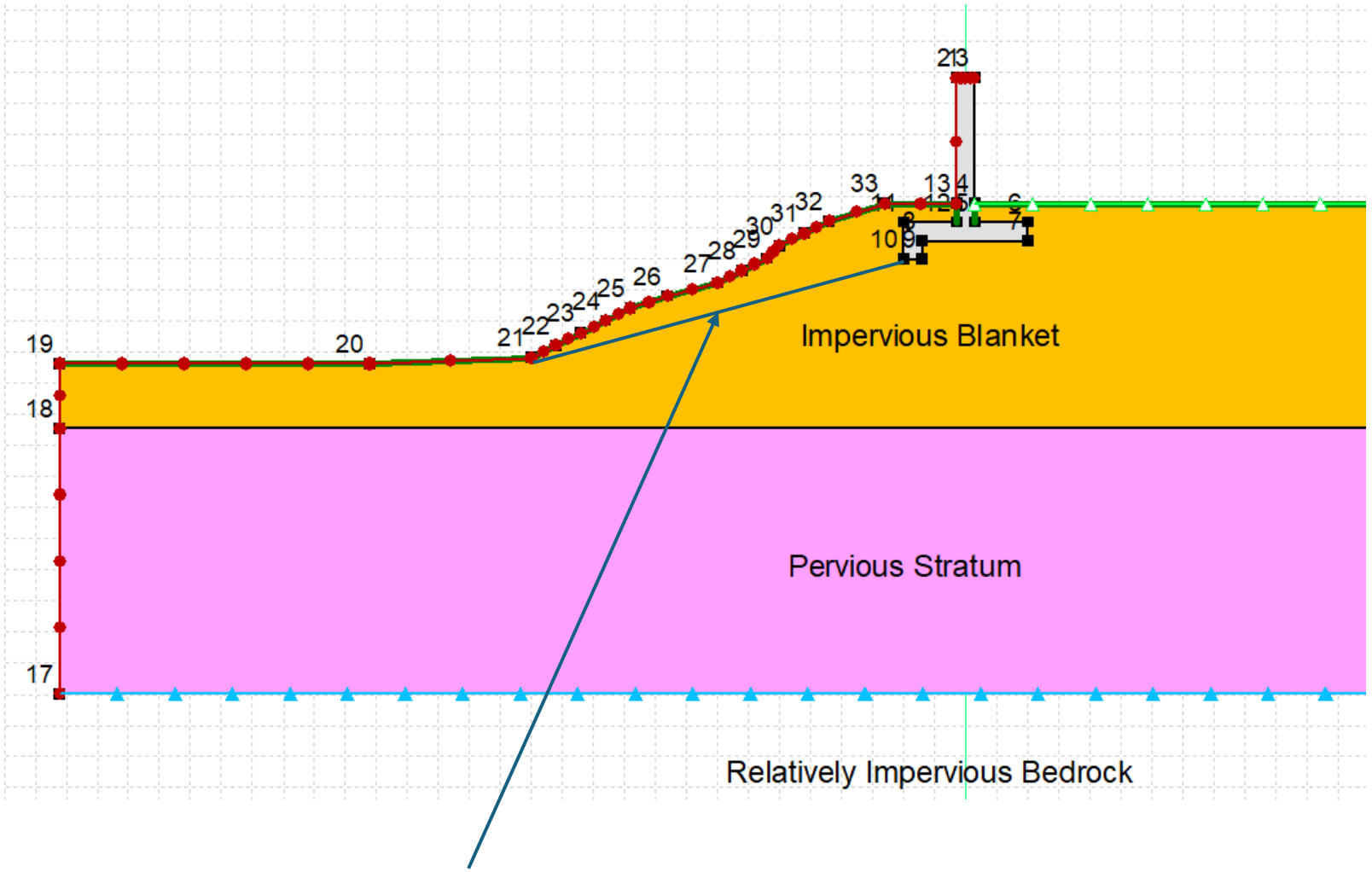


RS from toe	5+ coord	EL
6'	-11	908
8'	-13	907
10'	-15	906
11'	-16	905
13'	-18	904
15'	-20	903
19'	-24	902
22'	-27	901
24'	-29	900
26'	-31	899
28'	-33	898
30'	-35	897
43'	-48	896.5

channel N50'
go 25' to -73 896.5

RB F 33+80/UTC 41+75

Developing Cross Section

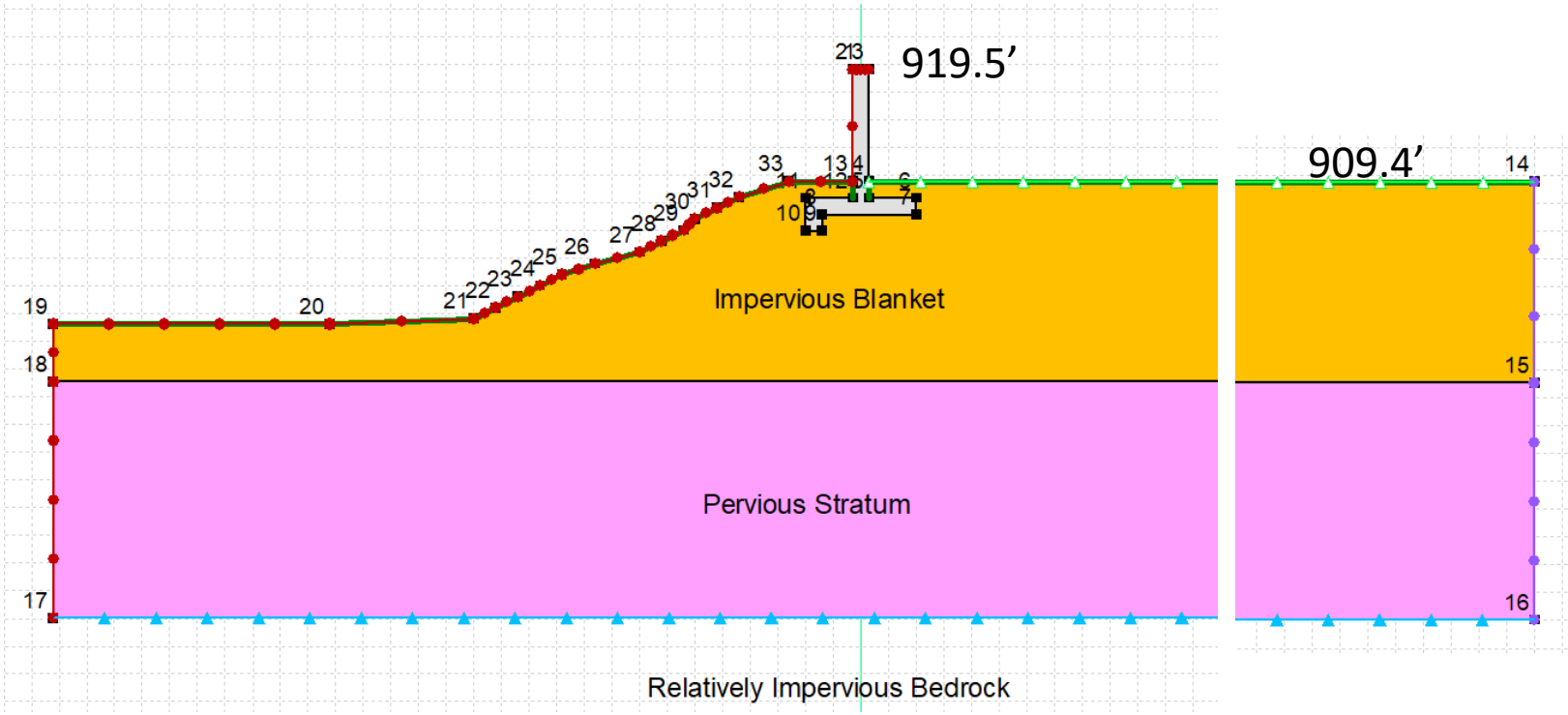


(904.9-897)/(35-5) = 7.9/30 = 0.263 = 1V/3.8H
as shown on Aug 2025 alignment

Define Points

ID	X (ft)	Y (ft)
1	0	919.5
2	-0.75	919.5
3	0.75	919.5
4	0.75	909.4
5	0.75	907.9
6	5	907.9
7	5	906.4
8	-3.5	906.4
9	-3.5	904.9
10	-5	904.9
11	-5	907.9
12	-0.75	907.9
13	-0.75	909.4
14	2,000	909.4
15	2,000	891.3
16	2,000	870
17	-73	870
18	-73	891.3
19	-73	896.5
20	-48	896.5
21	-35	897
22	-33	898
23	-31	899
24	-29	900
25	-27	901
26	-24	902
27	-20	903
28	-18	904
29	-16	905
30	-15	906
31	-13	907
32	-11	908
33	-6.5	909.4
34	---	---

RB F 33+80/UTC 41+75 SEEP/W



Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)
	Concrete	(none)	
	Impervious Blanket (Drained)	Saturated Only	3e-06
	Pervious	Saturated Only	0.003

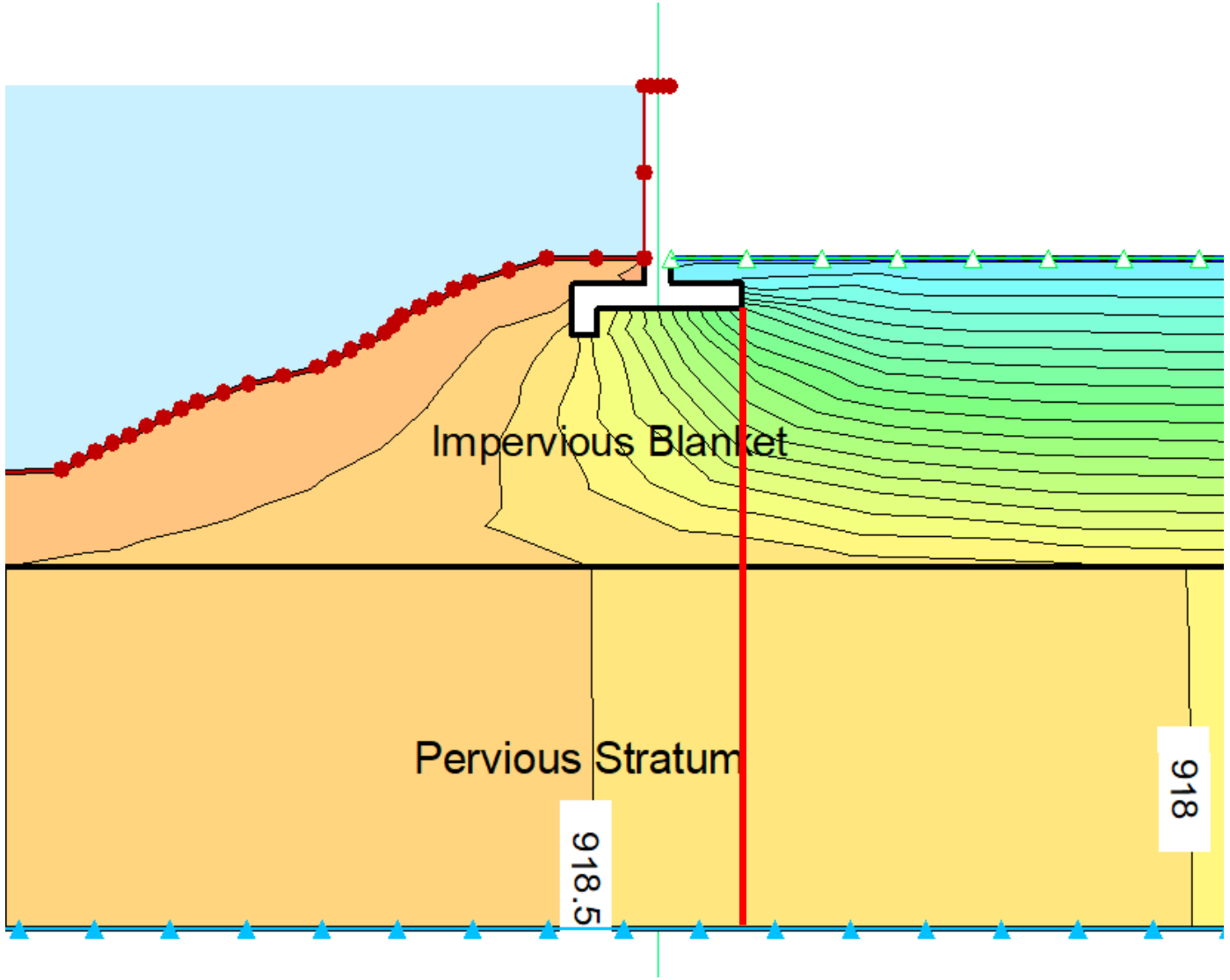
All Boundary Conditions

Name	Category	Hydraulic Boundary Type	Color	
Bedrock - No Flow	Hydraulic	Water Rate		Water Rate = 0 ft³/sec
Full Head	Hydraulic	Water Total Head		Water Total Head = 919.5 ft
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate		Water Rate = 0 ft³/sec
Tailwater Head	Hydraulic	Water Total Head		Water Total Head = 909.4 ft

Potential Seepage Face Review

RB F 33+80/UTC 41+75

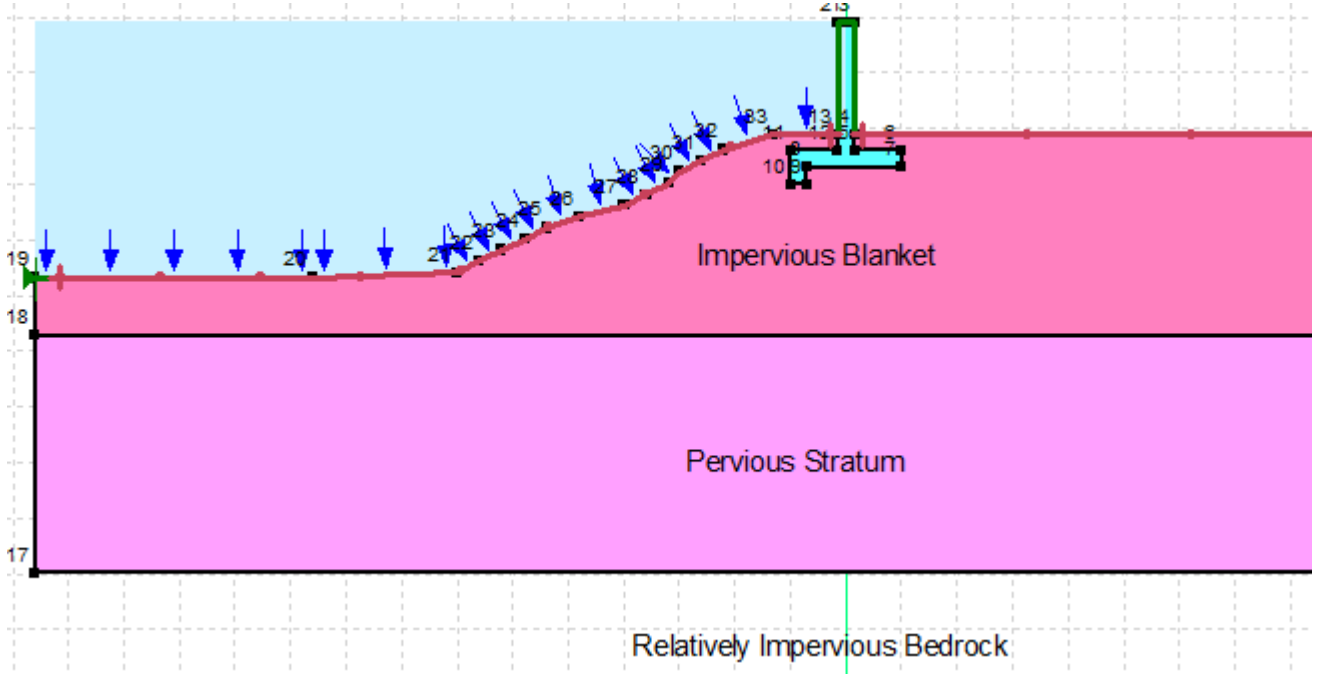
SEEP/W calibration to blanket theory spreadsheet



Station	Head at Toe (elevation msl) h_o
RB R4 F 33+80/UTC 41+75	918.36

RB F 33+80/UTC 41+75 SLOPE/W End of Construction Input

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)
	Concrete	High Strength	145			
	Impervious Blanket (Undrained)	Mohr-Coulomb	115	1,000	0	0
	Pervious	Mohr-Coulomb	115	0	31	0

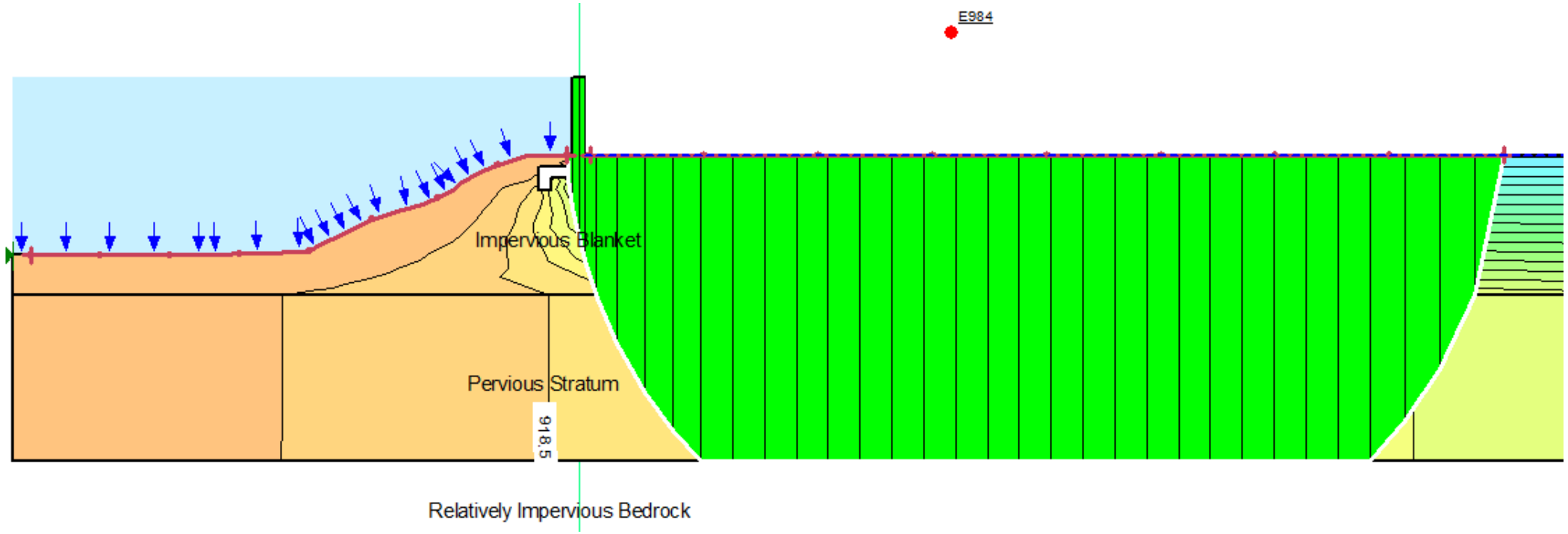


RB F 33+80/UTC 41+75 SLOPE/W End of Construction Results

No feasible failure surface could be determined based on the soil conditions, shallow bedrock, and lack of downward slope on the landside
Steeper slopes or cracks resulted in the same error message

Error message for auto-selected critical failure surface (shown)

Slip #	F of S	X Center (ft)	Y Center (ft)	Radius (ft)	Details
405	E984	59.0000	912.50461	60.58	Error 984: Slip surface intersects vertical/overhanging bedrock or weak surface, contains air gap, bedrock, or weak surface



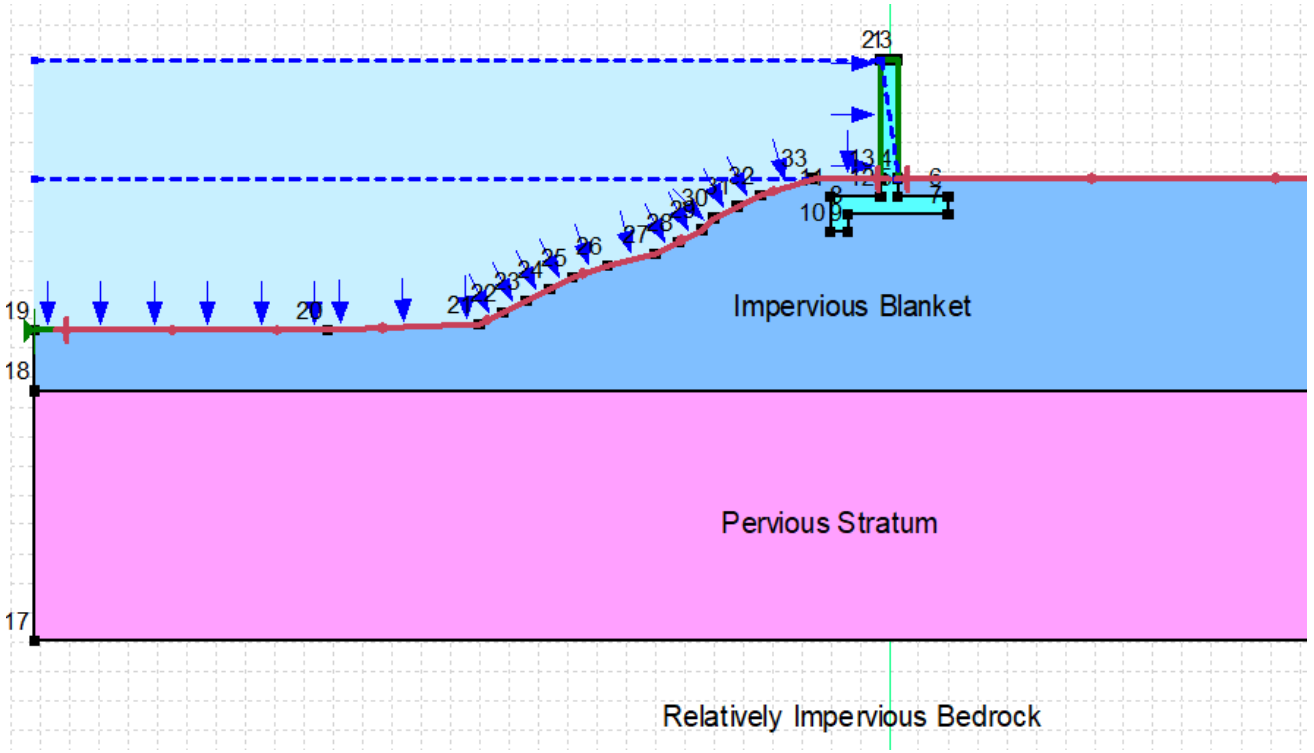
Error messages for other slip surfaces

Error 998: Slip surface enters or exits beyond specified slip surface limits

Error 996: Slip surface direction of movement is opposite to specified direction

RB F 33+80/UTC 41+75 SLOPE/W Rapid Drawdown Input

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Piezometric Surface	Piezometric Surface After Drawdown	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Cohesion R (psf)	Phi R (°)
	Concrete	High Strength	145	1	2					
	Impervious Blanket (Drawdown)	Mohr-Coulomb	115	1	2	0	28	0	200	18
	Pervious	Mohr-Coulomb	115	1	2	0	31	0	0	0

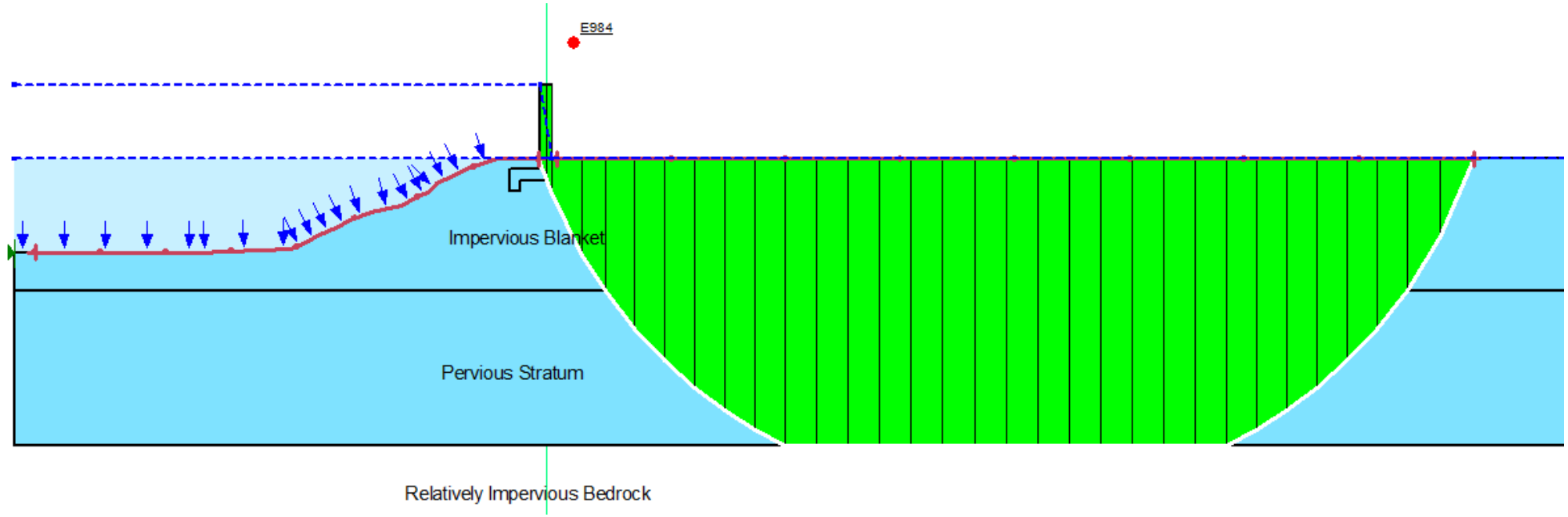


RB F 33+80/UTC 41+75 SLOPE/W Rapid Drawdown Results

No feasible failure surface could be determined based on the soil conditions and shallow bedrock
Steeper slopes or cracks resulted in same error message

Error message for auto-selected critical failure surface (shown)

Error 984: Slip surface intersects vertical/overhanging bedrock or weak surface, contains air gap, bedrock, or weak surface



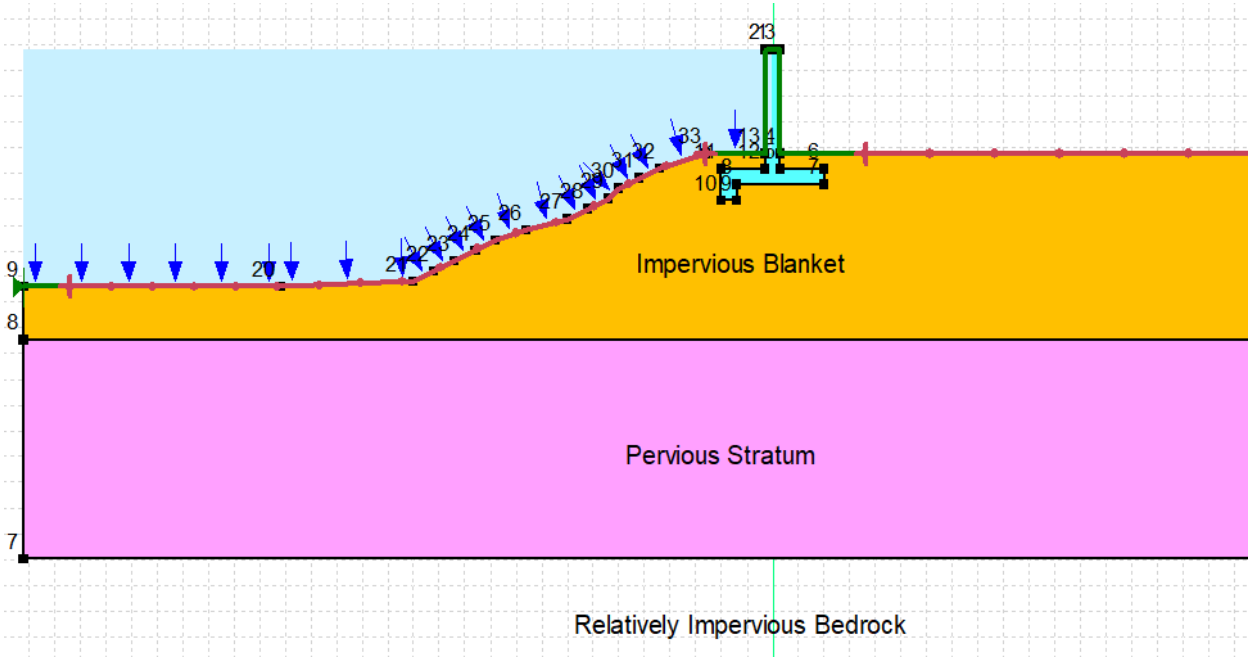
Error messages for other slip surfaces

Error 998: Slip surface enters or exits beyond specified slip surface limits

Error 994: Cannot find intersection of factor of safety and Lambda

RB F 33+80/UTC 41+75 SLOPE/W Steady State Input

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Shear/Normal Strength Fn.
	Concrete	High Strength	145				
	Impervious Blanket (Drained)	Shear/Normal Fn.	115			0	CU Sample 202 5-7
	Pervious	Mohr-Coulomb	115	0	31	0	

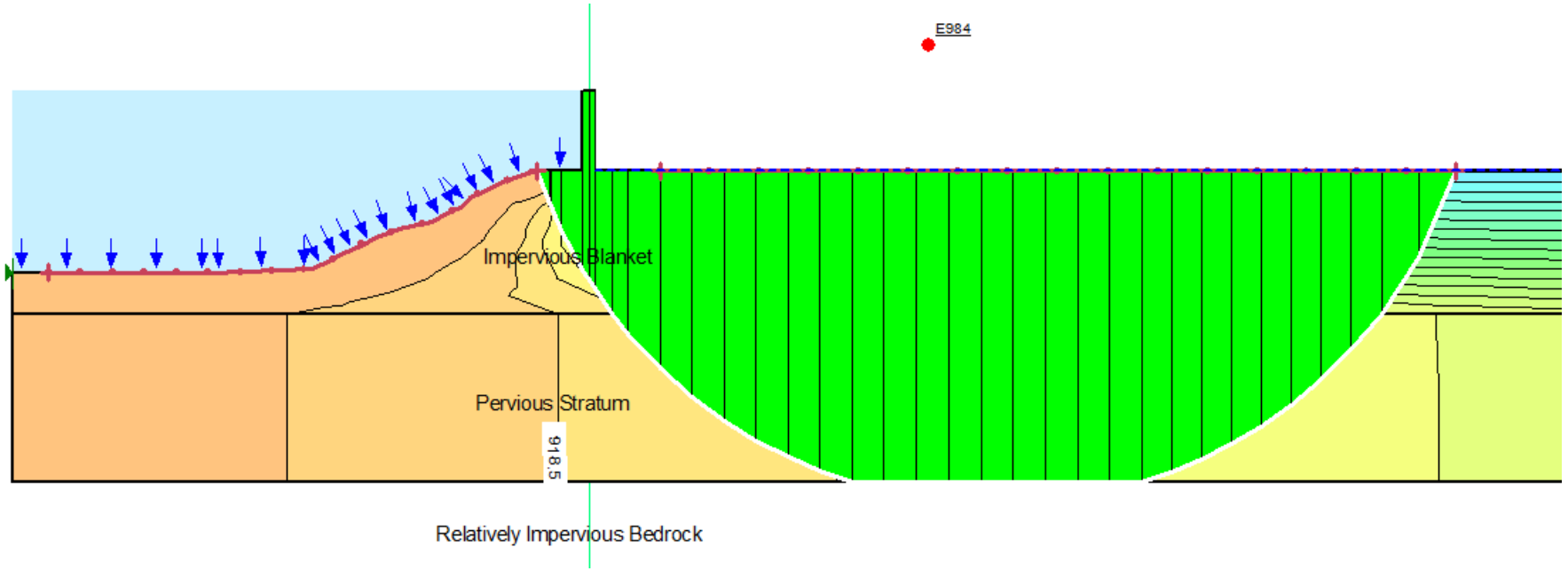


RB F 33+80/UTC 41+75 SLOPE/W Steady State Results

No feasible failure surface could be determined based on the soil conditions, shallow bedrock, and lack of downward slope on the landside
Steeper slopes or cracks resulted in same error message.

Error message for auto-selected critical failure surface (shown)

Error 984: Slip surface intersects vertical/overhanging bedrock or weak surface, contains air gap, bedrock, or weak surface



Error messages for other slip surfaces

Error 998: Slip surface enters or exits beyond specified slip surface limits

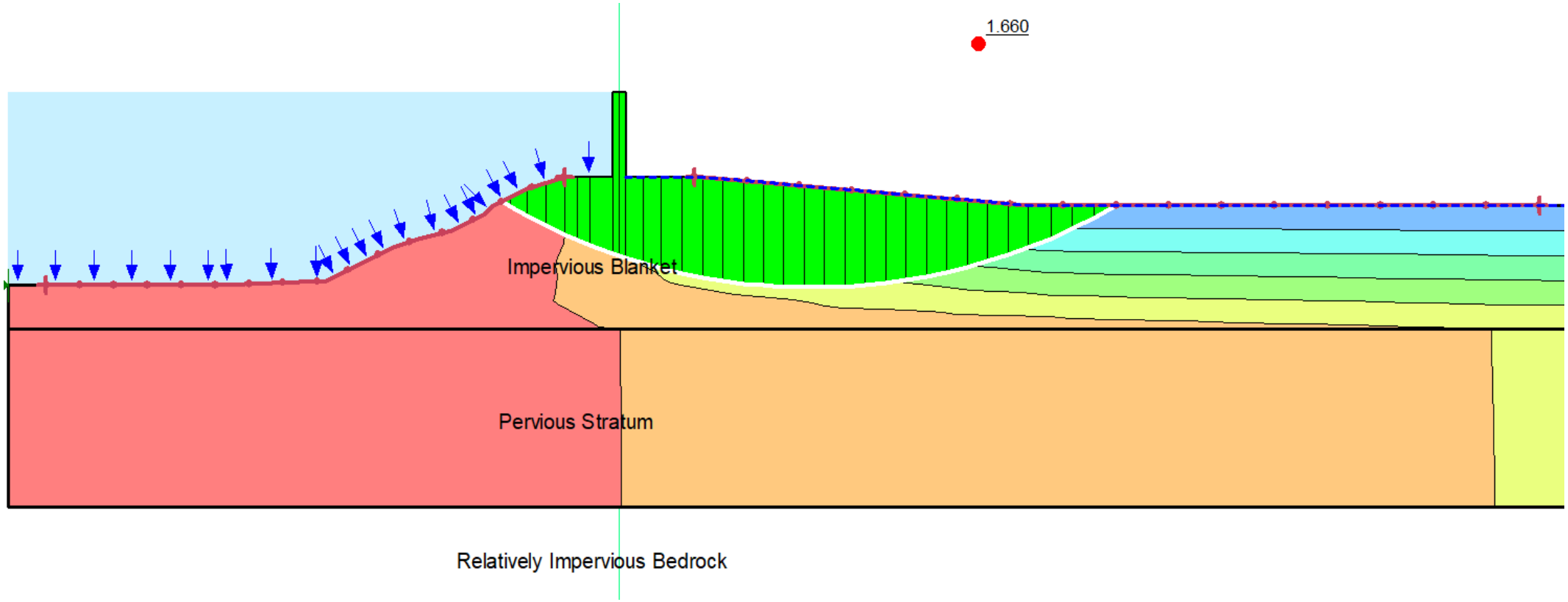
Error 996: Slip surface direction of movement is opposite to specified direction

Error 993: Slip surface is shallower than defined minimum depth

RB F 33+80/UTC 41+75 SLOPE/W Steady State Results

Analyzed with a landside slope rather than the existing flat landward surface in order to find a feasible slope failure surface

With this more conservative slope, FS is still greater than 1.4 required

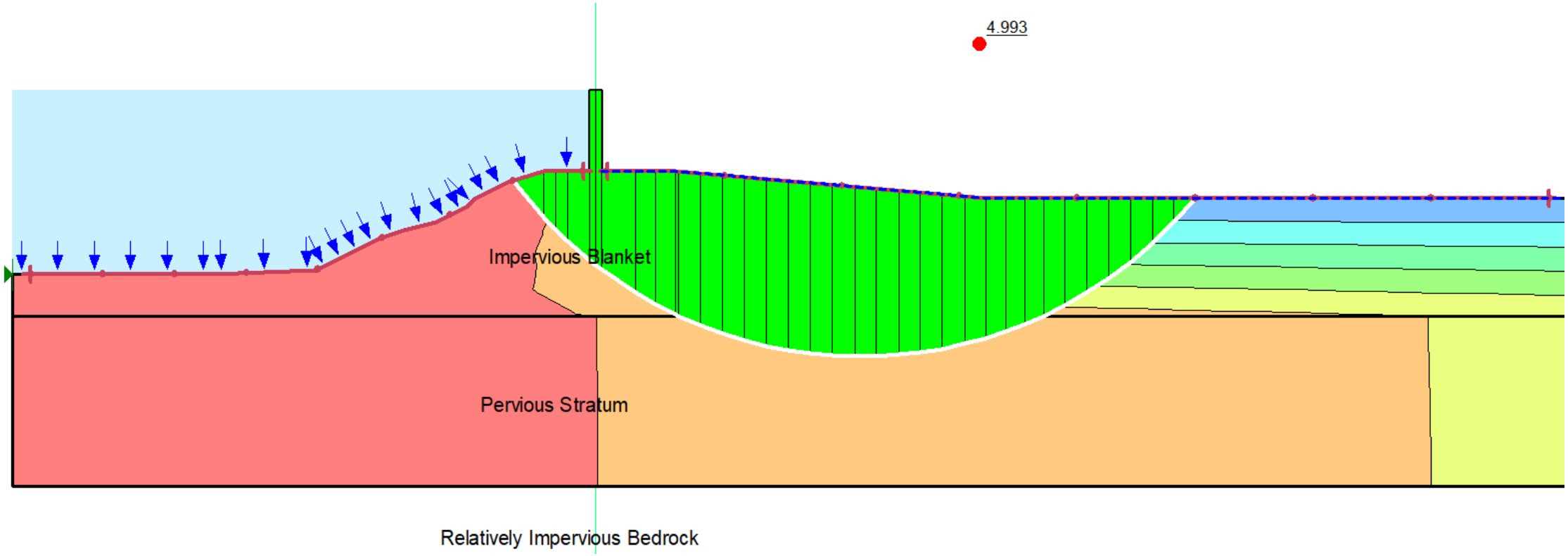


RB F 33+80/UTC 41+75

SLOPE/W End of Construction Results

Analyzed with a landside slope rather than the existing flat landward surface in order to find a feasible slope failure surface

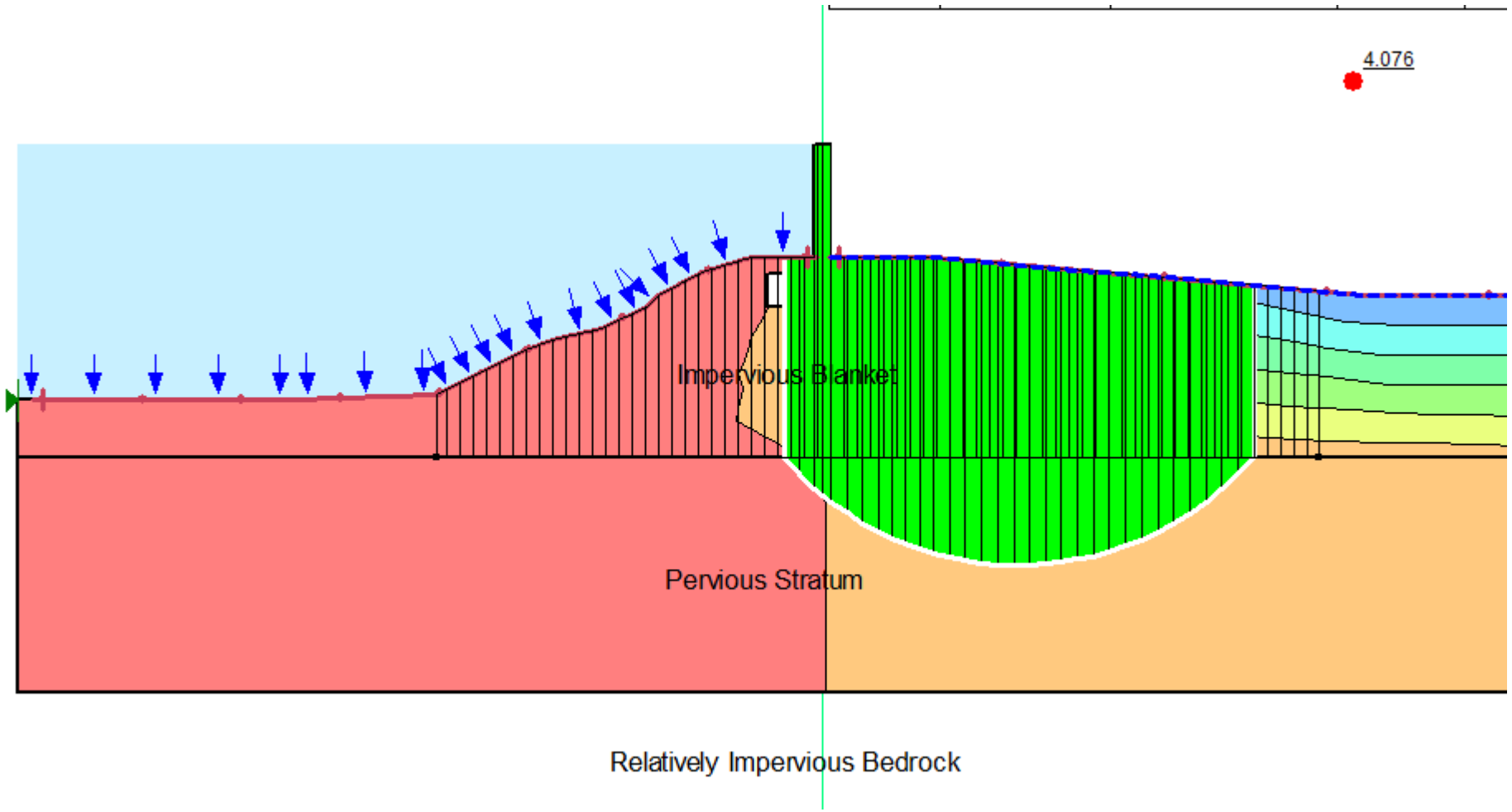
With this more conservative slope, FS is still greater than 1.3 required



RB F 33+80/UTC 41+75 SLOPE/W End of Construction w Tension Crack Results

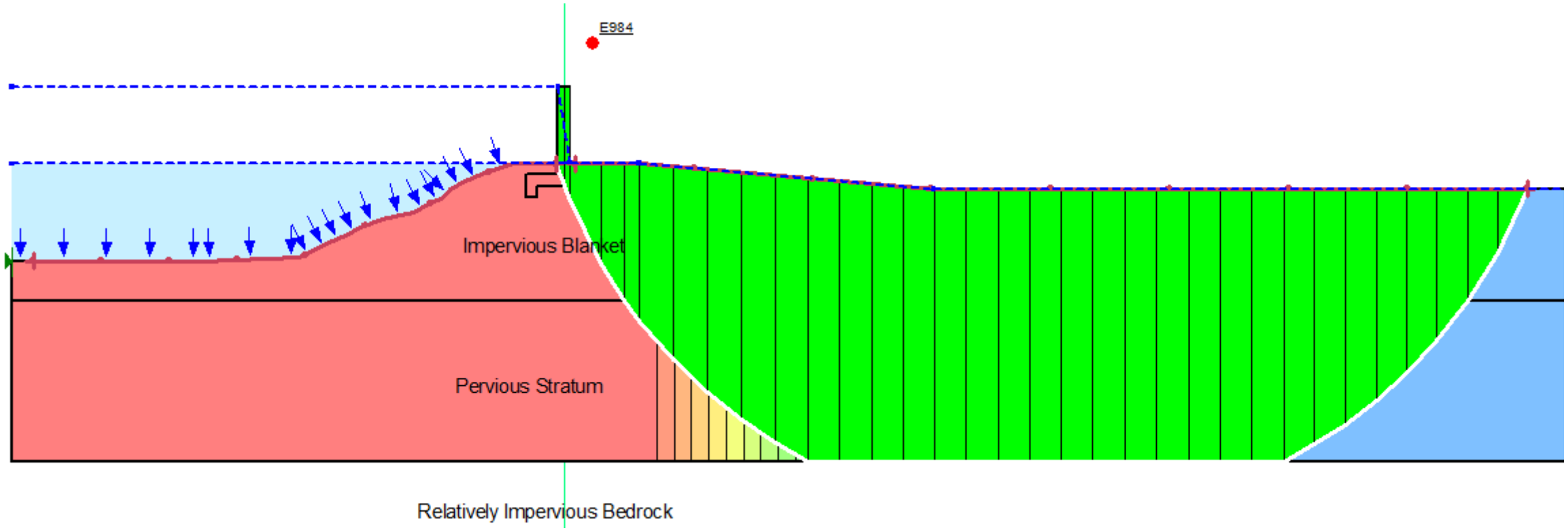
Analyzed with a landside slope rather than the existing flat landward surface in order to find a feasible slope failure surface

With this more conservative slope, FS is still greater than 1.3 required



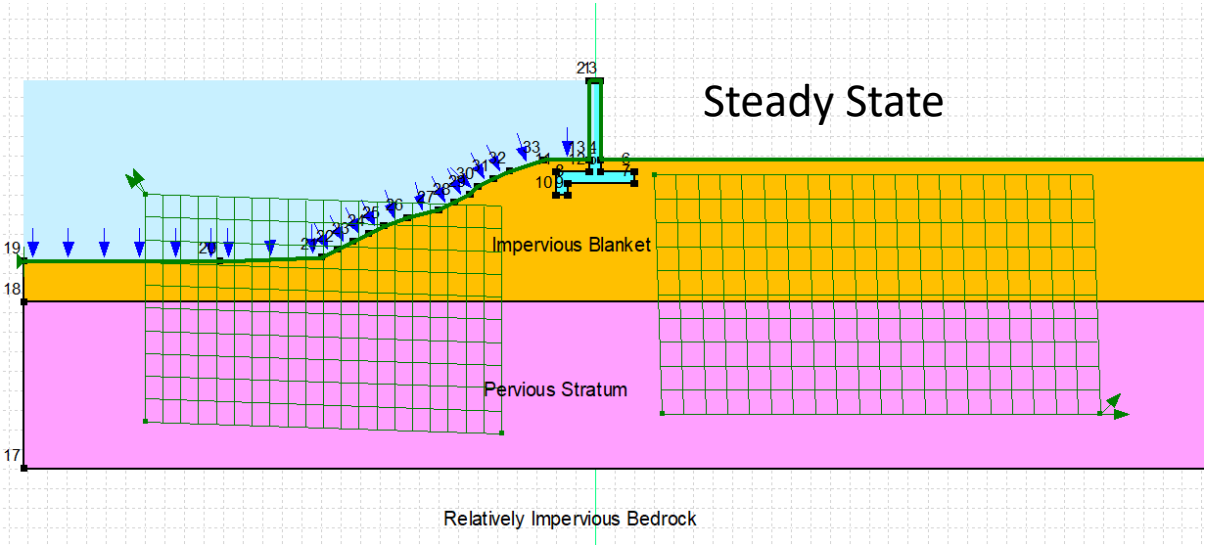
RB F 33+80/UTC 41+75 SLOPE/W Rapid Drawdown Results

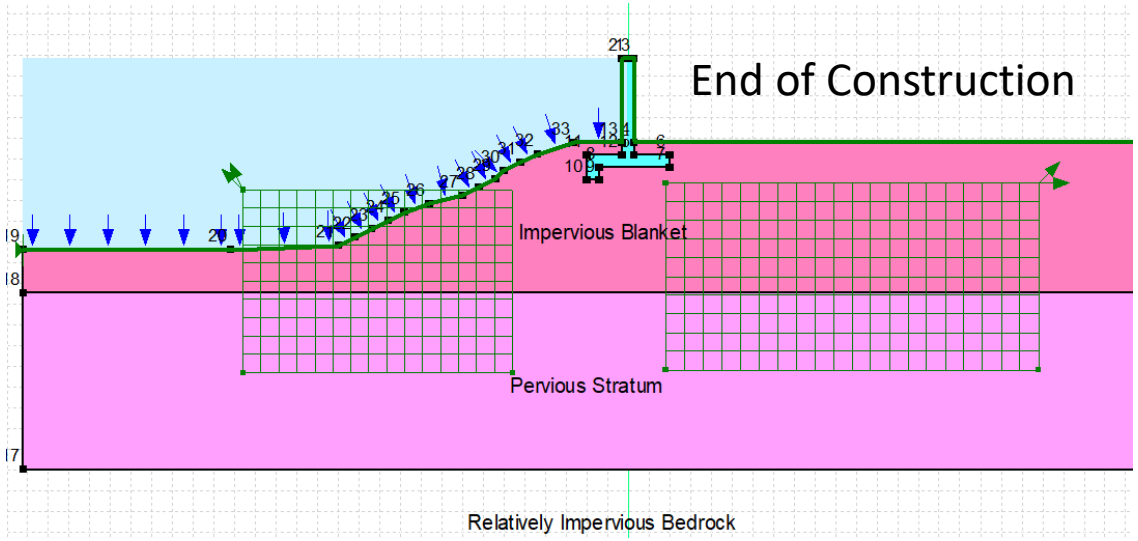
The landside slope added to evaluate steady state and end of construction did not affect rapid drawdown with its failure on the riverside slope

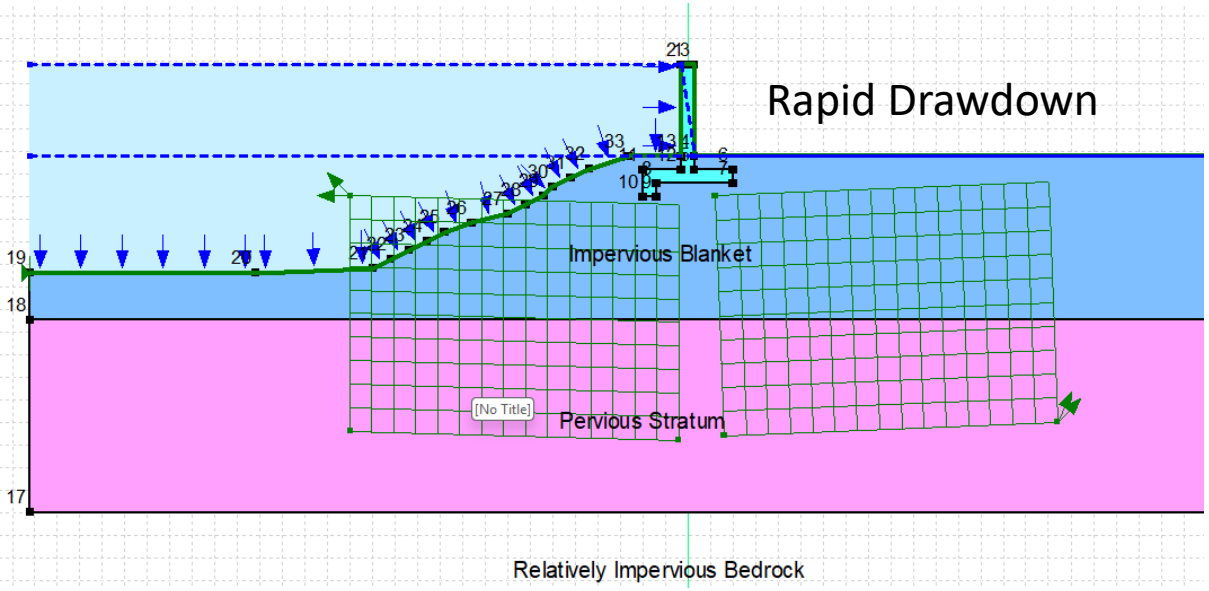


RB F 33+80/UTC 41+75 SLOPE/W Block Failure Inputs

Also tried stability runs with block-specified failure surface. Only rapid drawdown analyses ran without an error message.





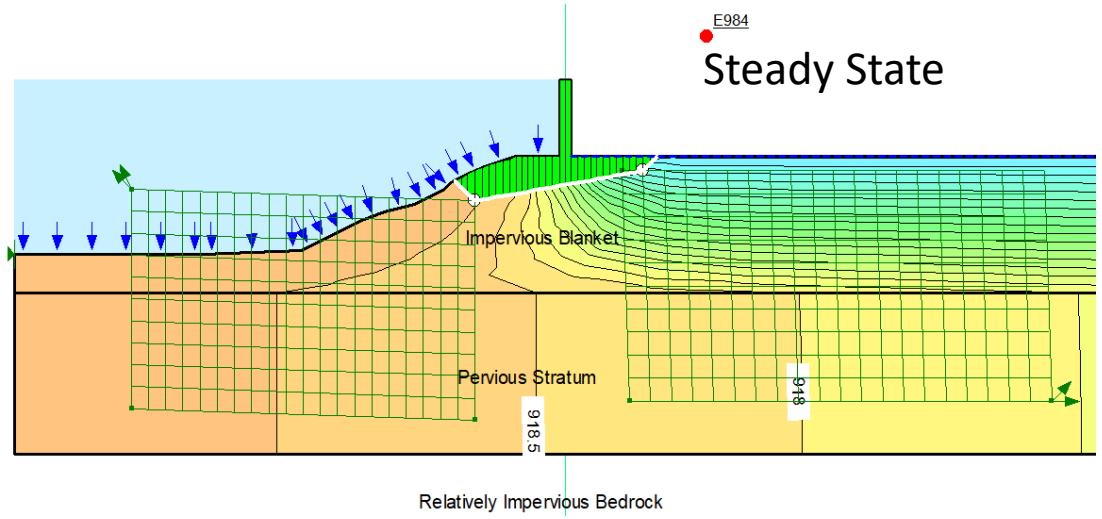


RB F 33+80/UTC 41+75

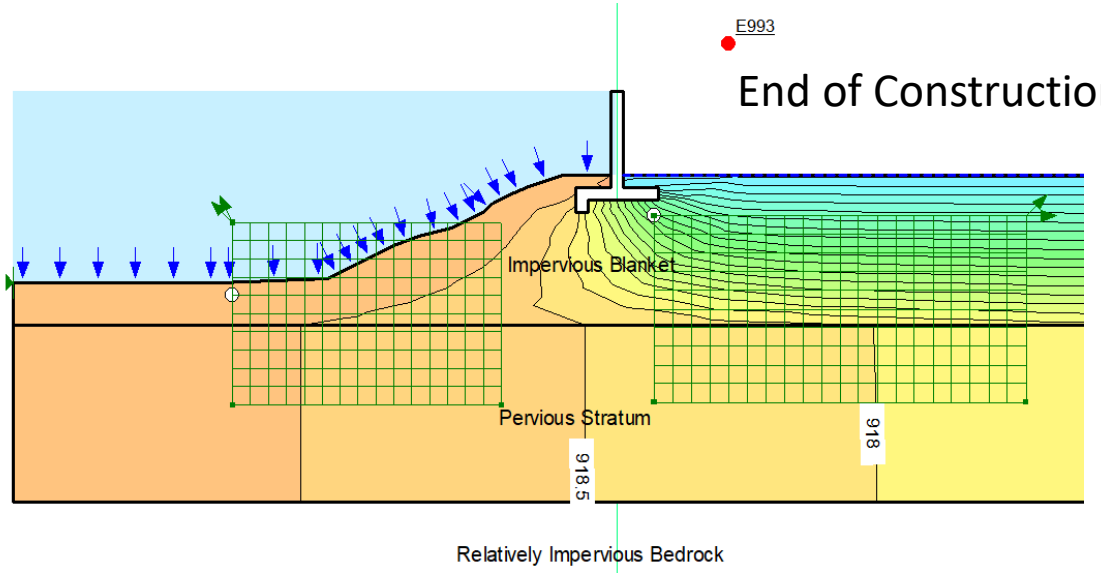
SLOPE/W Block Failure Output

Also tried stability runs with block-specified failure surface. Tried several different variations with similar results. Only rapid drawdown analyses ran without an error message.

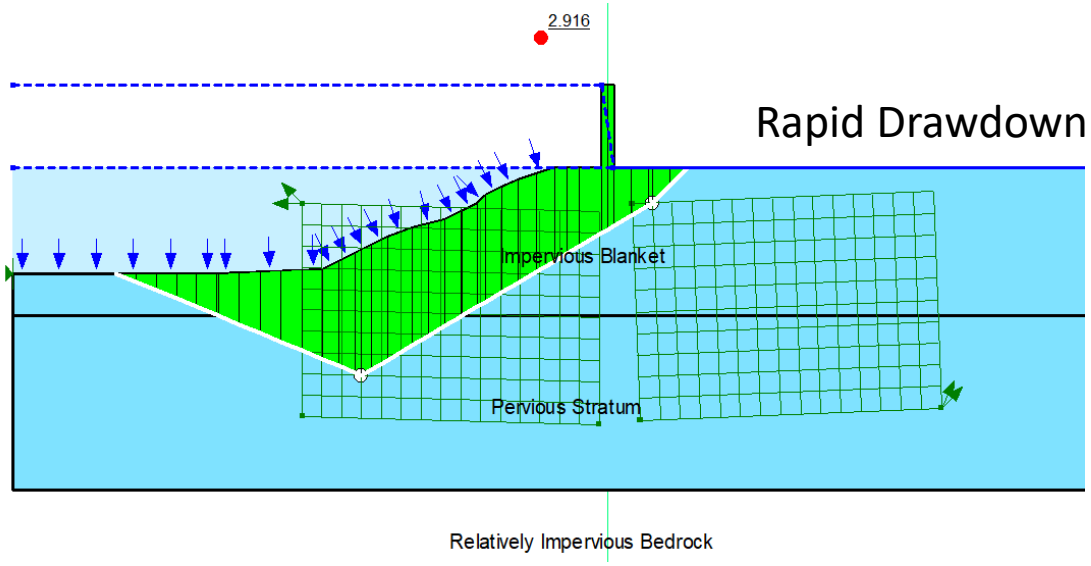
Steady State



End of Construction



Rapid Drawdown



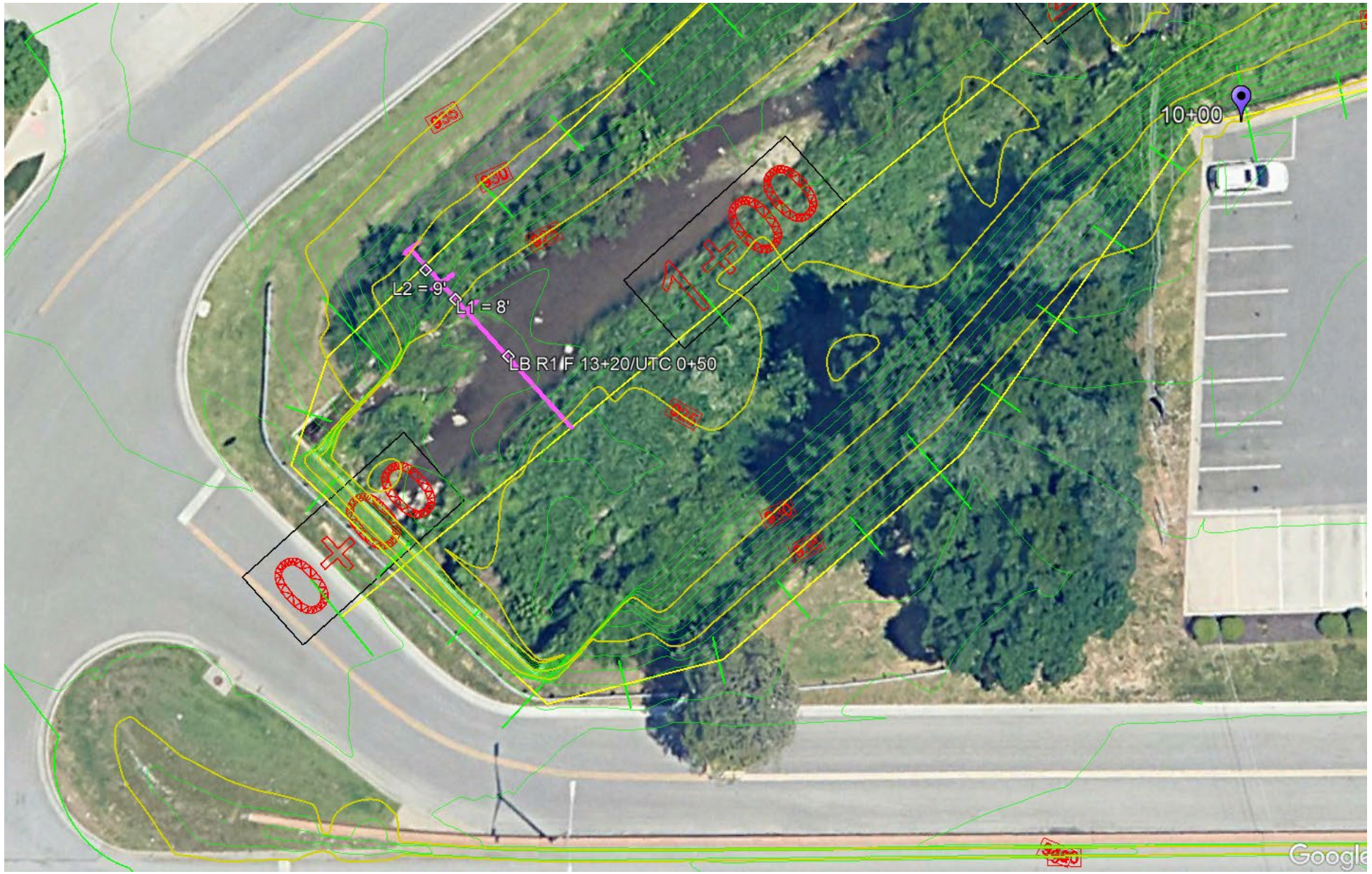
Error 993: Slip surface is shallower than defined minimum depth

Error 984: Slip surface intersects vertical/overhanging bedrock or weak surface, contains air gap, bedrock, or weak surface

For steady state, the same error messages were obtained for runs at reduce slopes. Based on high FS or error messages and lack of sensitivity to the slope on other runs, additional runs at reduced slopes were not attempted at this time for end of construction or drawdown

Other Floodwall Analyses

LB R1 F 13+20/UTC 0+50



LB R1 F 13+20/UTC 0+50

Levee Unit Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYSIS TAB AND ASSOCIATED PL

Case Analyzed - Water at Levee Top

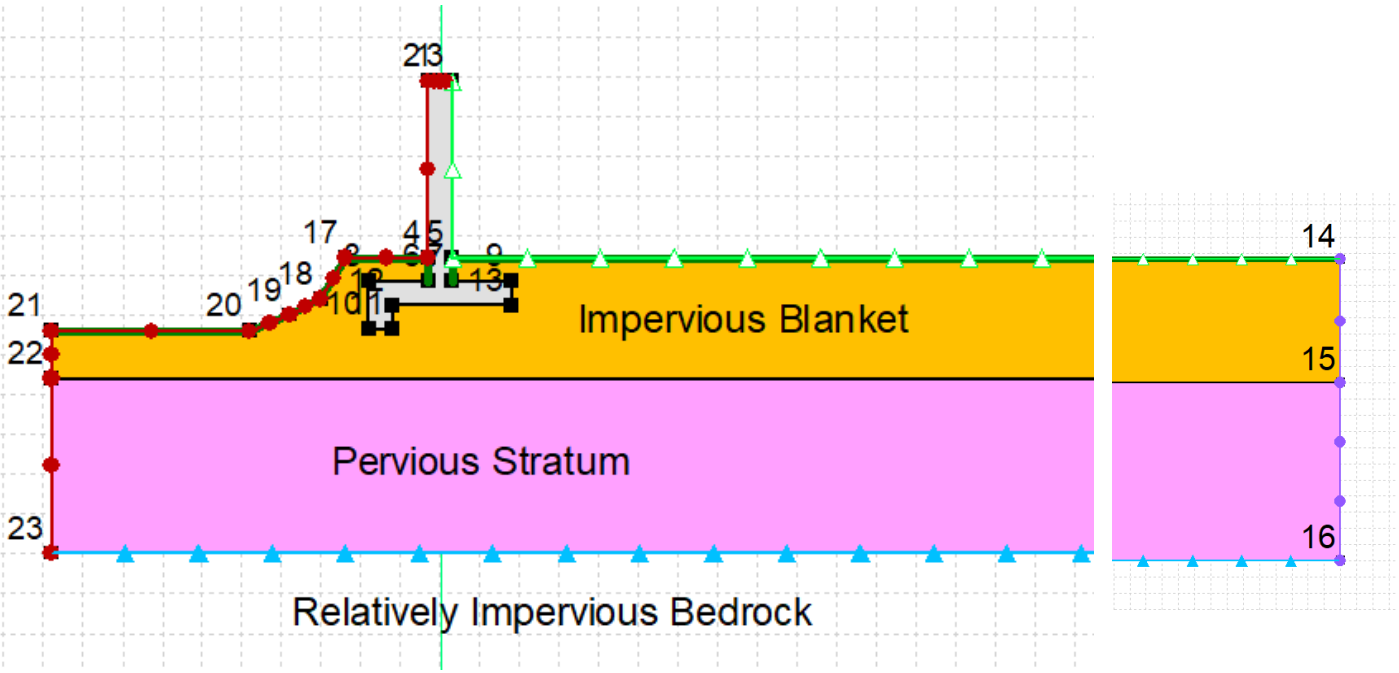
Blanket Unit Weight = 115.0 pcf (saturated)

	Top of Levee	River Side Ground	Land Side Ground	Land Side Tailwater	River Blanket Bottom	Soil Boring	Land Blanket Bottom	Modified Top of Bedrock*	Driving	Permeability Ratio		Impervious Blanket Thickness (ft)			Pervious Blanket	Seepage Length (ft)		
	Elevation	Elevation	Elevation	Elevation	Elevation	Reference	Elevation	Elevation	Head (ft)	River Side	Land Side	River Side	Levee	Land Side	Thickness (ft)	Riverside	Levee	Land Side
Station	(msl)	(msl)	(msl)	(msl)	(msl)			(msl)	H	k_r/k_{br}	k_r/k_{bl}	z_{br}	z_{bo}	z_{bl}	d	L_1	L_2	L_3
LB R1 F 13+20/UTC 0+50	939.7	928.6	928.6	928.6	921	ADU-207	921	910	11.1	400	400	7.6	7.6	7.6	11	8	9	infinity

calibration

Factor		Effective Seepage Length (ft)		Head at	Head at	Computed Hydraulic	Critical Hydraulic	Factor of Safety for		
		River Side	Land Side	Toe (ft)	Toe (elevation msl)	Gradient	Gradient	Effective Stress Uplift / Blanket Rupture	average horizontal gradient between entrance and levee toe	FS horizontal gradient (i_{h-c} = 1/5 = 0.2)
c_r	c_l	x_1	x_3	h_o	h_o	i_o	i_c	(i_c/i_o)	(i_h)	(i_{h-c}/i_h)
0.005468	0.005468	7.99	182.87	10.16	938.76	1.34	0.84	0.63	0.65	0.31

LB R1 F 13+20/UTC 0+50



Define Boundary Conditions

BC Category: All

All Boundary Conditions

Name	Category	Hydraulic Boundary Type	Color
Bedrock - No Flow	Hydraulic	Water Rate	
Full Head	Hydraulic	Water Total Head	
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate	
Tailwater Head	Hydraulic	Water Total Head	

Name: Bedrock - No Flow

Kind: Water Rate

Boundary Condition Type

Constant: 0 ft³/sec

Function:

Name: Tailwater Head

Kind: Water Total Head

Boundary Condition Type

Constant: 928.6 ft

Function:

Name: Full Head

Kind: Water Total Head

Boundary Condition Type

Constant: 939.7 ft

Function:

Name: Landside Slope - Potential Seepage Face

Kind: Water Rate

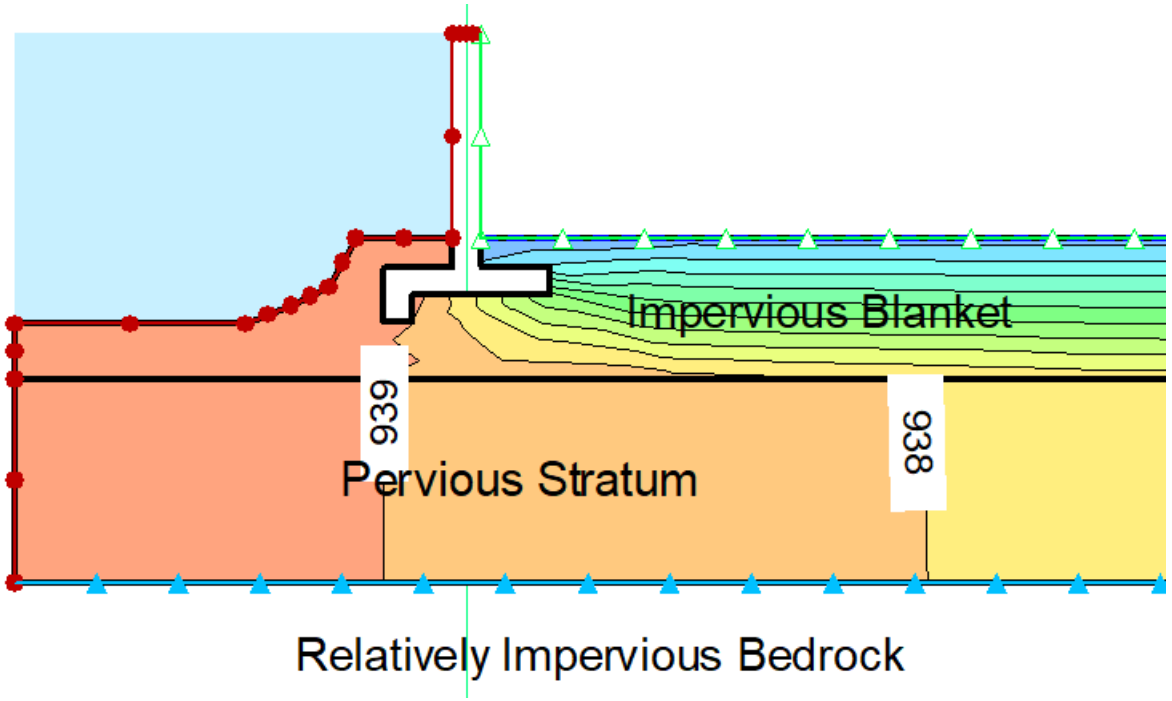
Boundary Condition Type

Constant: 0 ft³/sec

Function:

Potential Seepage Face Review

LB R1 F 13+20/UTC 0+50



calibration	
Head at	
Toe (elevation msl)	
h_o	
938.76	

Name: Impervious Blanket (Drained)

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	3e-06 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

Name: Pervious

Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	0.003 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

Name: Impervious Fill (Drained)

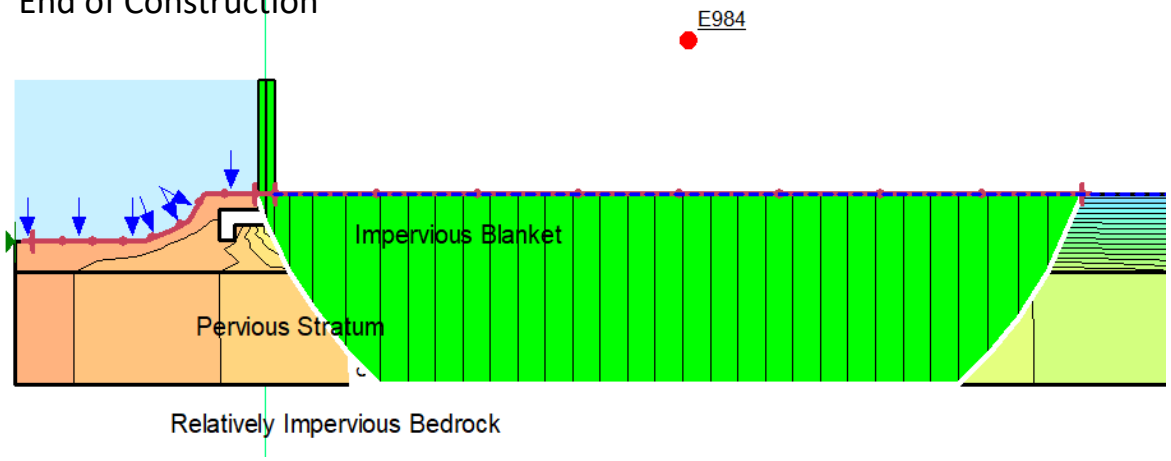
Hydraulic

Material Model:	Saturated Only
Saturated X-Conductivity:	1.5e-05 ft/sec
Sat. Vol. Water Content:	0
Compressibility:	0 /psf
Anisotropy	
Ky'/Kx' Ratio:	1
Kz'/Kx' Ratio:	1
Dip Direction:	90 °
Dip:	0 °

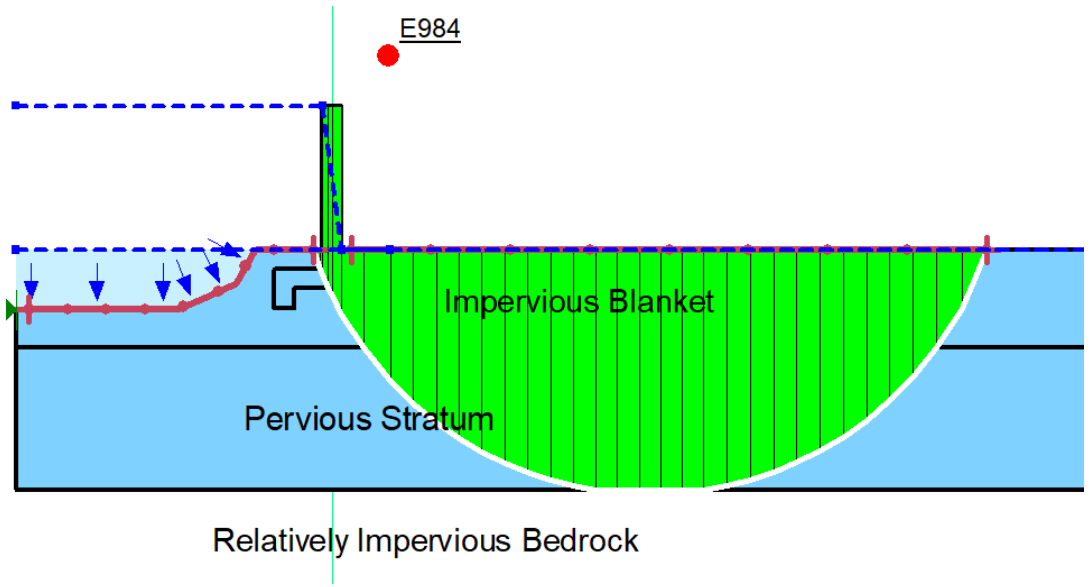
LB R1 F 13+20/UTC 0+50 SLOPE/W Results

Like with the other floodwall, the analyses resulted in errors due to the failure surface intersecting shallow bedrock

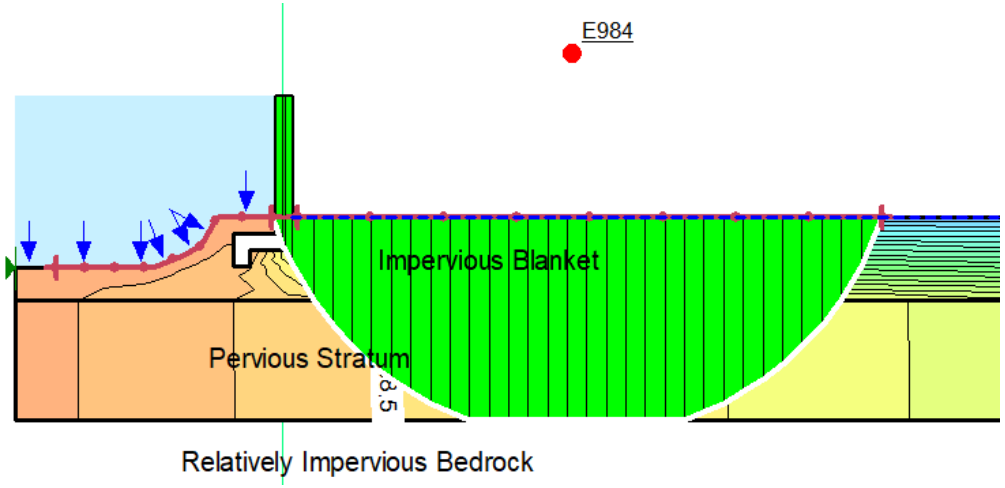
End of Construction



Rapid Drawdown



Steady State



For this cross section with very shallow bedrock, adding a landside slope did not help find a feasible failure surface

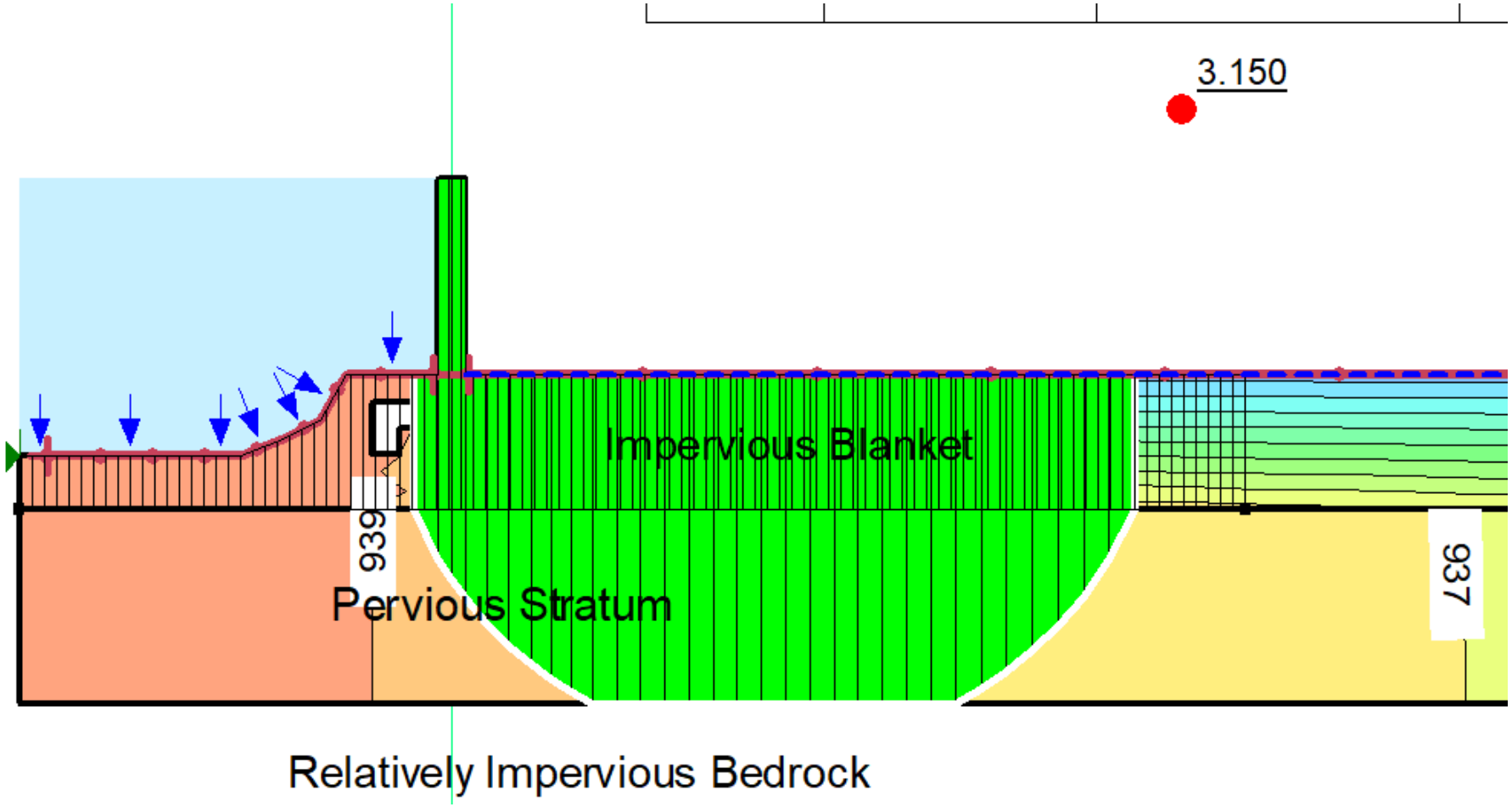
The end of construction with tension crack case did obtain a factor of safety (see next page)

For steady state, the same error messages were obtained for runs at reduced slopes. Based on high FS or error messages and lack of sensitivity to the slope on other runs, additional runs at reduced slopes were not attempted at this time for end of construction or drawdown

LB R1 F 13+20/UTC 0+50 SLOPE/W Results

Factor of Safety was obtained from end of construction with tension crack case

End of Construction w Tension Crack



APPENDIX B.3 – I-WALL DESIGN

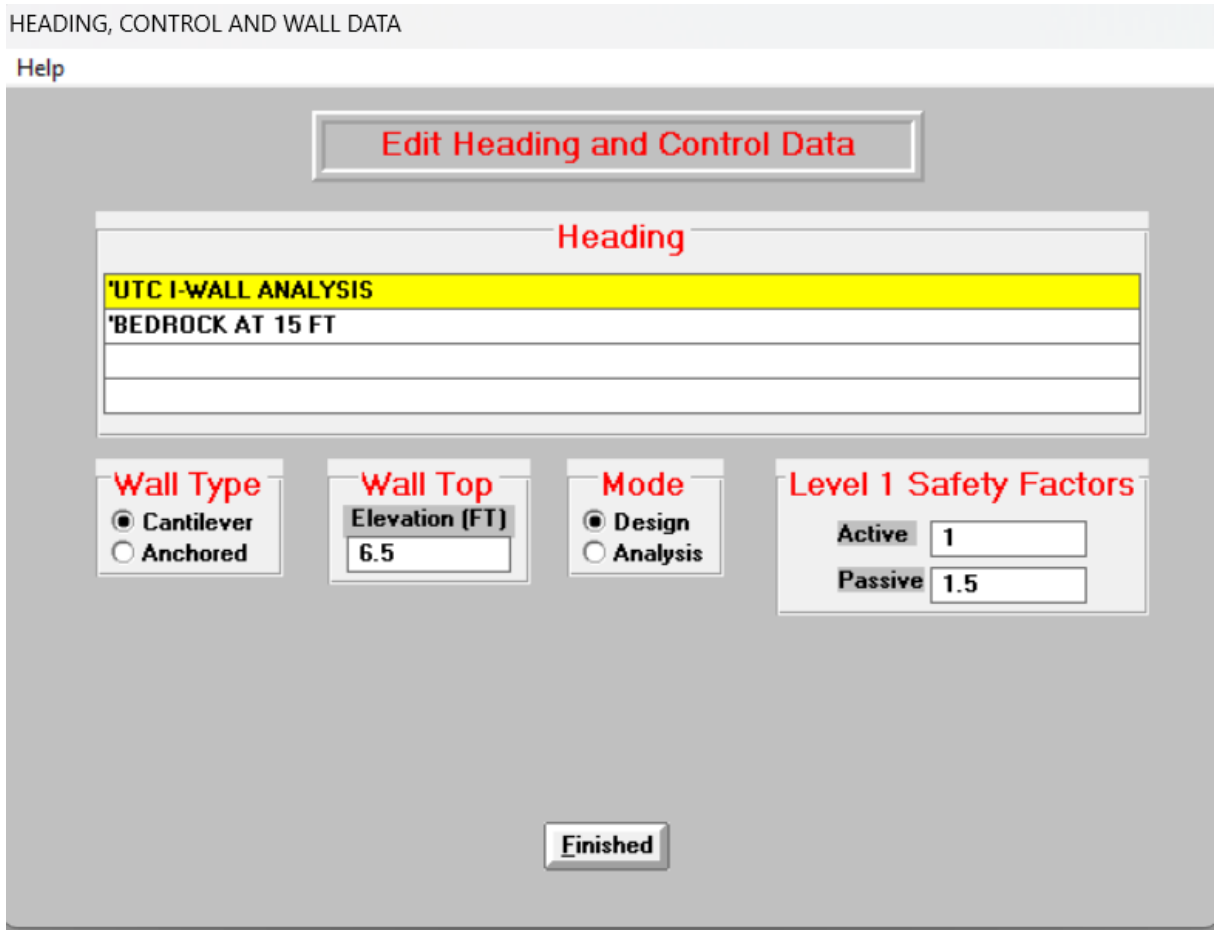
I-wall Rotational Stability

CWALSHT Analysis

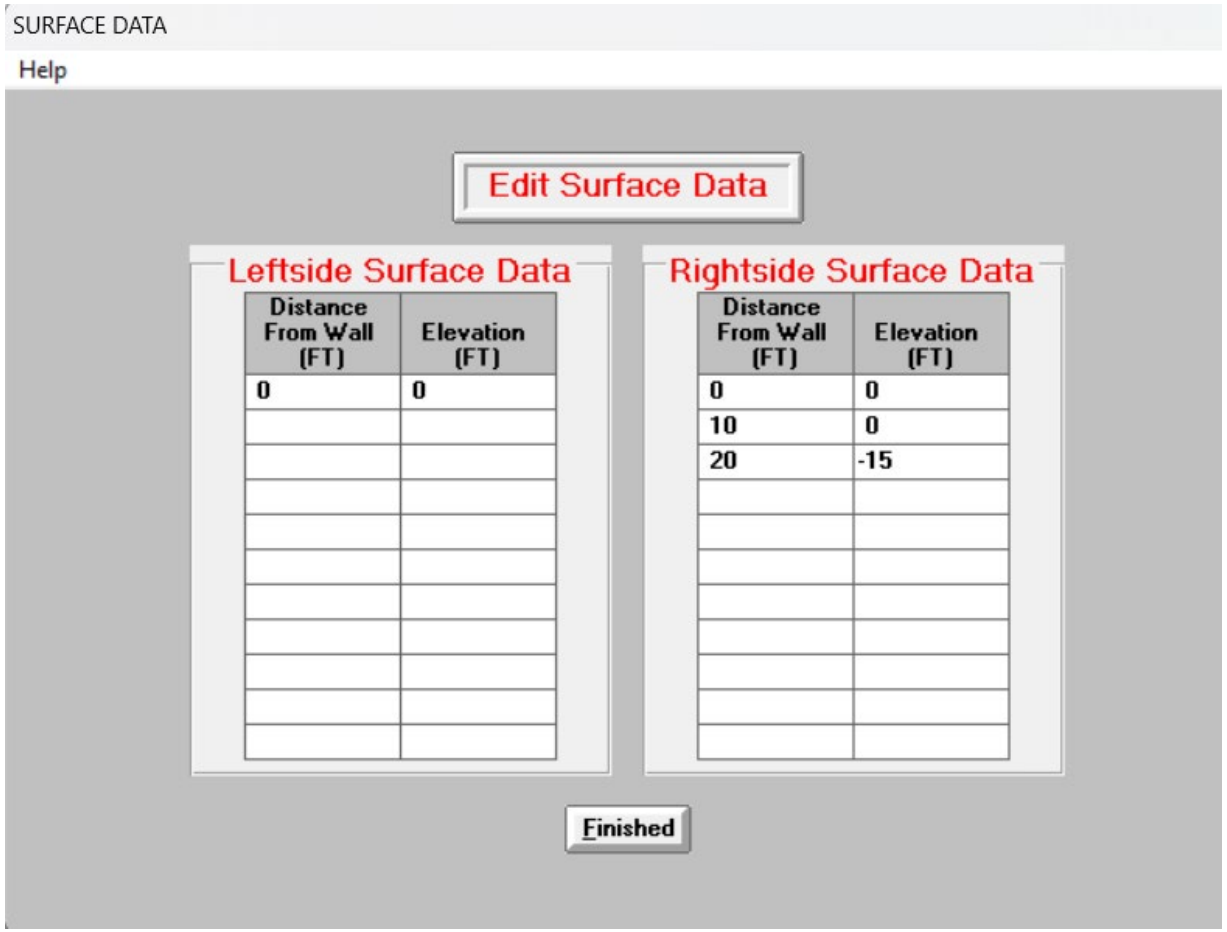
CWALSHT Inputs

Guidance Followed from
EM 1110-2-2502 Appendix F

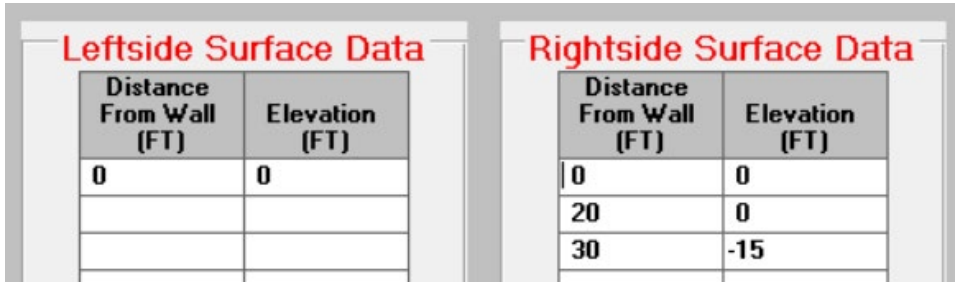
Note that water side is the right side in CWALSHT



Wall 10 ft from Stream Bank



Wall 20 ft from Stream Bank





Default Level 2 and Level 3 safety factors were used by leaving the sections blank, this follows the example given in EM 1110-2-2502 Appendix F.

Without Pervious Layer

SOIL LAYER DATA

Help

Drained

Edit RIGHTSIDE Soil Data

Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
115	115	28	0	14	0	-15	0		
140	140	35	500	17.5	250				

Undrained

Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
115	115	0	1000	0	500	-15	0		
140	140	35	500	17.5	250				

With Pervious Layer

SOIL LAYER DATA

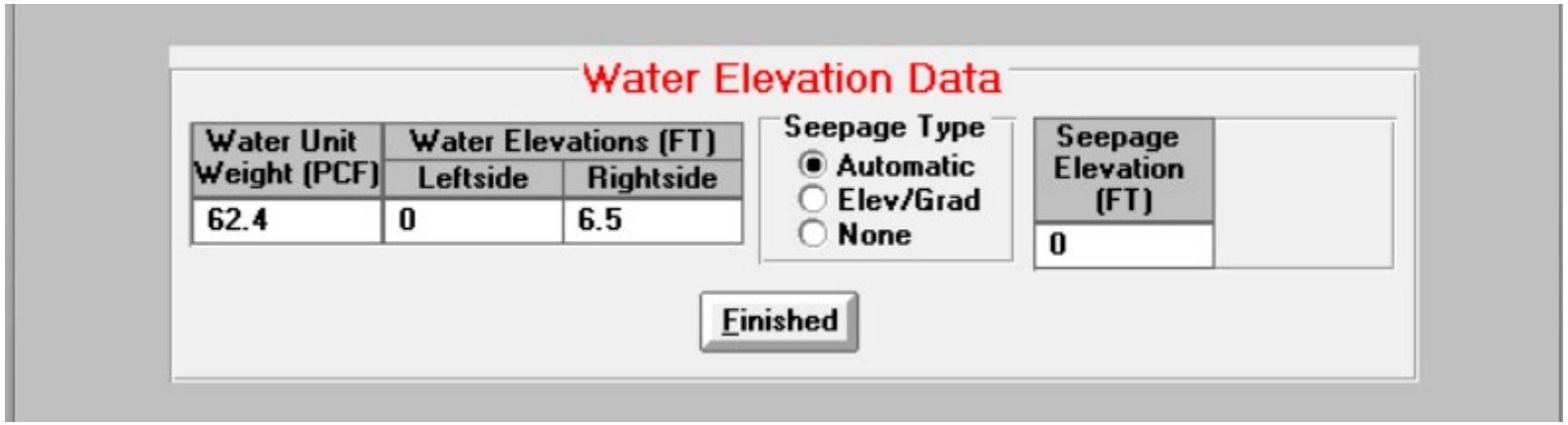
Help

Edit RIGHTSIDE Soil Data

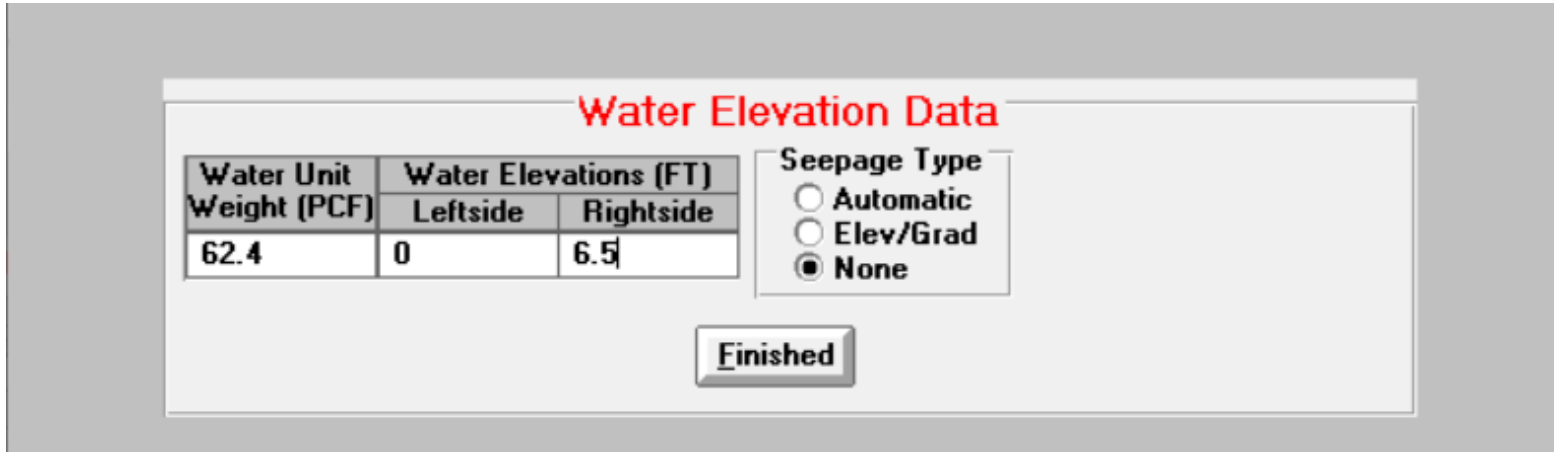
Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
115	115	28	0	14	0	-15	0		
115	115	31	0	15.5	0	-20	0		
140	140	35	500	17.5	250				

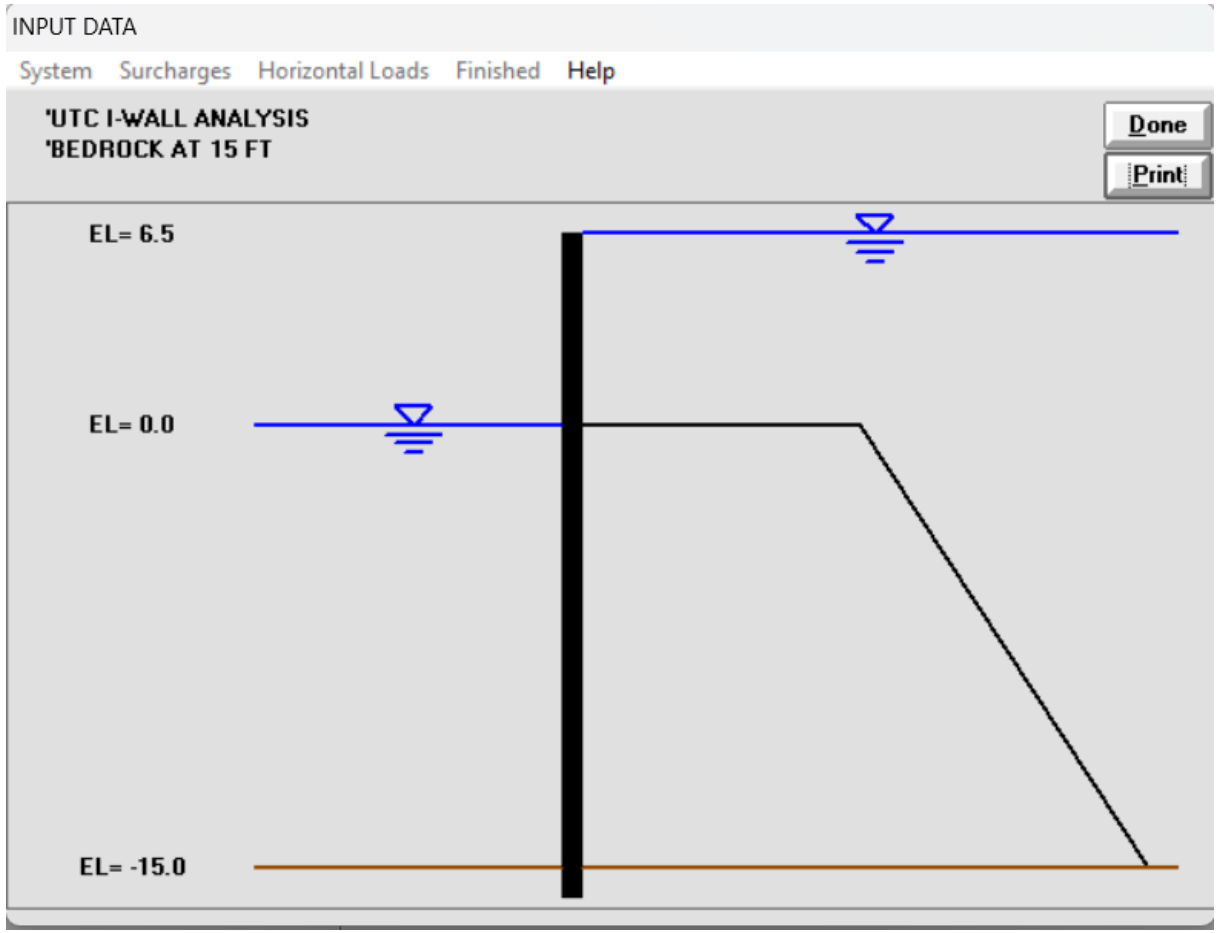
Finished With Rightside Soil

Drained

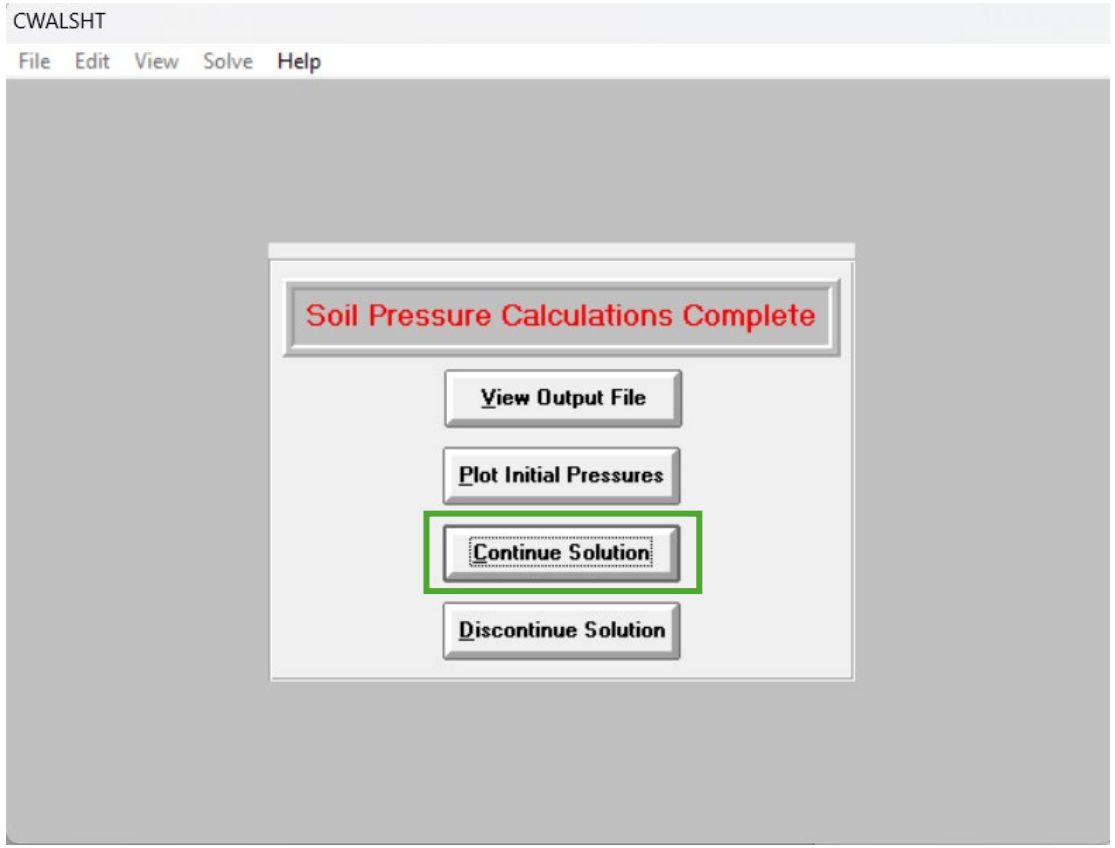


Undrained



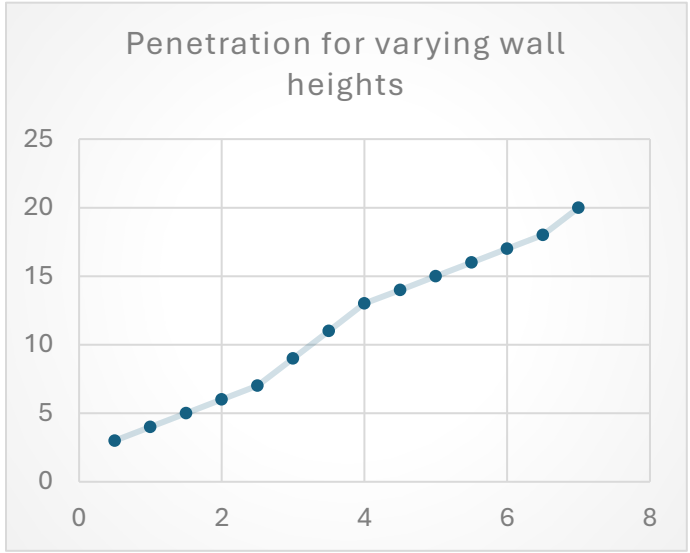

INPUT DATA
System
Surcharges
Horizontal Loads
Finished
Help
'UTC I-WALL ANALYSIS
'BEDROCK AT 15 FT
Done
Print
EL= 6.5
EL= 0.0
EL= -15.0


WEDGE METHOD?
Help
WEDGE METHOD REQUIRED FOR SOIL PRESSURES
Select Desired Wedge Method
Sweep Search
Fixed Surface


CWALSHT
File Edit View Solve Help
Soil Pressure Calculations Complete
View Output File
Plot Initial Pressures
Continue Solution
Discontinue Solution

CWALSHT Results

Required I-Wall Penetration Depth for Varying Subsurface Conditions (ft)									
Wall Height Above GS (ft)	Bedrock Depth (ft) No Pervious Layer			Pervious Layer at 15 – 20 ft BGS		Pervious layer 10-15 ft	Continuous Clay Layer, No Bedrock		**Selected Penetration Depth (ft)**
	20	15	10	Wall 10 ft from Bank	Wall 20 ft from Bank	Wall 10 ft from bank	Wall 10 ft from bank	Wall 20 ft from bank	
2	6.4	--	--	6.2	--		6.4	5.6	**6**
2.5	--	--	--	--	7.0		8.0	7.0	**7**
3	--	--	--	--	8.4		9.6	8.4	**9**
3.5	11.2	11.0	9.8	11.0	10.8		11.2	11.1	**11**
4	12.7	--	--	12.7	12.6		12.7	12.6	**13**
4.5	--	14.2	--	--	13.5		Osc -13.9	Osc -14.0	**14**
5	15.9	14.5	14.2	14.8	14.5	--	15.9	16.0	**15**
5.5	17.2	Osc -14.6	14.8	Osc -15.15	Osc -15.1	14.64	17.2	16.8	**16**
6	18.7	15.7	Osc -16.2	18.1	16.1 E*	Osc -15.09	18.7	18.0	**17**
6.5	Osc 19.13	17.1 *4.5	15.8	19.5	17.6	Osc -14.69	20.2	19.6	**18**
7	Osc -19.18 7.42*	18.0 E*		Osc -19.1 E*	18.8 E*	15.13	21.6 7.4*	21.2 5.1*	**20**



*undrained solution

Note – wall heights below 2 feet will not solve, used slope of 2 (1V:0.5H) to project penetrations for 0.5, 1.0, and 1.5 ft wall heights

Selection of I-wall Reaches

Y	penetration available within 1 foot
G	grading or pre-drilling/excavation required
N	wall too tall or grading >3ft required
NA	Not floodwall
	critical section analyzed

Alignment	Existing Elev (ft)	Proposed Elev (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer	Levee, FW or None	Height of protection abv ground	Nearest 0.5	Riprap	Iwall Feasibility	Template Ranges (original)
+	941.897	939	920.2	21.7	0.0	21.7	None	-2.897	0		NA	
+20	941.662	939	920.2	21.5	0.0	21.5	None	-2.662	0		NA	
+40	941.451	939	920.2	21.3	0.0	21.3	None	-2.451	0		NA	
+60	940.556	939	920.2	20.4	0.0	20.4	None	-1.556	0		NA	None
+80	940.031	939	920.2	19.8	0.0	19.8	None	-1.031	0		NA	
1+00	939.571	939	920.2	19.4	0.0	19.4	None	-0.571	0		NA	
1+20	939.322	939	920.2	19.1	0.0	19.1	None	-0.322	0		NA	
1+40	938.775	939	920.2	18.6	0.0	18.6	FW	0.225	0	None	Y	
1+60	938.718	939	920.2	18.5	3.0	15.5	FW	0.282	0.5		Y	
1+80	938.104	939	920.2	17.9	4.0	13.9	FW	0.896	1		Y	
2+00	937.546	939	920.2	17.3	5.0	12.3	FW	1.454	1.5		Y	
2+20	937.774	939	920.2	17.6	4.0	13.6	FW	1.226	1		Y	I wall
2+40	937.509	939	920.2	17.3	5.0	12.3	FW	1.491	1.5		Y	1+40 to 3+20
2+60	936.644	939	920.2	16.4	7.0	9.4	FW	2.356	2.5		Y	
2+80	934.205	939	920.2	14.0	15.0	-1.0	FW	4.795	5	21	Y	
3+00	934.39	939	920.2	14.2	14.0	0.2	FW	4.61	4.5	21	Y	
3+20	934.191	939	920.2	14.0	15.0	-1.0	FW	4.809	5	21	G	
3+40	933.993	939	920.2	13.8	15.0	-1.2	Levee	5.007	5	21	NA	Levee
3+60	933.795	939	920.2	13.6	15.0	-1.4	Levee	5.205	5	21	NA	3+20 to 3+75
3+80	933.593	938.962	920.2	13.4	16.0	-2.6	Levee	5.369	5.5	21	NA	
4+00	933.436	938.809	920.2	13.2	16.0	-2.8	Levee	5.373	5.5	21	NA	
4+20	933.384	938.657	920.2	13.2	16.0	-2.8	Levee	5.273	5.5	21	NA	
4+40	933.355	938.505	920.2	13.2	15.0	-1.8	Levee	5.15	5	21	NA	
4+60	933.31	938.352	920.2	13.1	15.0	-1.9	Levee	5.042	5	21	NA	
4+80	933.1	938.2	920.2	12.9	15.0	-2.1	Levee	5.1	5	21	NA	
5+00	932.907	938.18	920.2	12.7	16.0	-3.3	Levee	5.273	5.5	21	NA	
5+20	932.697	938.16	920.2	12.5	16.0	-3.5	Levee	5.463	5.5	21	NA	Levee w/2.25 RR
5+40	932.564	938.14	920.2	12.4	16.0	-3.6	Levee	5.576	5.5	21	NA	3+75 to 7+00
5+60	932.425	938.12	920.2	12.2	16.0	-3.8	Levee	5.695	5.5	21	NA	
5+80	932.343	938.1	920.2	12.1	17.0	-4.9	Levee	5.757	6	21	NA	
6+00	932.26	938.08	920.2	12.1	17.0	-4.9	Levee	5.82	6	21	NA	
6+20	932.164	938.06	920.2	12.0	17.0	-5.0	Levee	5.896	6	21	NA	
6+40	931.995	938.04	920.2	11.8	17.0	-5.2	Levee	6.045	6	21	NA	
6+60	931.835	938.02	920.2	11.6	17.0	-5.4	Levee	6.185	6	21	NA	
6+80	931.722	938	920.2	11.5	18.0	-6.5	Levee	6.278	6.5	21	NA	
7+00	931.611	938	920.2	11.4	18.0	-6.6	FW	6.389	6.5	21	N	
7+20	931.504	938	920.2	11.3	18.0	-6.7	FW	6.496	6.5	21	N	
7+40	931.398	938	920.2	11.2	18.0	-6.8	FW	6.602	6.5	21	N	
7+60	931.057	938	920.2	10.9	20.0	-9.1	FW	6.943	7	21	N	
7+80	930.812	938	920.2	10.6	20.0	-9.4	FW	7.188	7	21	N	
8+00	930.63	938	920.2	10.4	100.0	-89.6	FW	7.37	7.5	21	N	10 ft Twall w/2.25 RR
8+20	930.535	938	920.2	10.3	100.0	-89.7	FW	7.465	7.5	21	N	7+00 to 9+20
8+40	930.507	938	920.2	10.3	100.0	-89.7	FW	7.493	7.5	21	N	
8+60	930.46	938	919.978	10.5	100.0	-89.5	FW	7.54	7.5	21	N	
8+80	930.716	938	919.757	11.0	100.0	-89.0	FW	7.284	7.5	21	N	
9+00	930.191	938	919.535	10.7	100.0	-89.3	FW	7.809	8	21	N	
9+20	929.781	937.85	919.313	10.5	100.0	-89.5	FW	8.069	8	21	N	
9+40	929.616	937.7	919.091	10.5	100.0	-89.5	FW	8.084	8	21	N	
9+60	929.35	937.55	918.870	10.5	100.0	-89.5	FW	8.2	8	21	N	10 ft Twall
9+80	929.223	937.4	918.648	10.6	100.0	-89.4	FW	8.177	8	21	N	9+20 to 10+00
10+00	928.941	937.192	918.426	10.5	100.0	-89.5	FW	8.251	8.5	21	N	
10+20	928.482	936.985	918.204	10.3	100.0	-89.7	FW	8.503	8.5	33	N	
10+40	928.131	936.777	917.983	10.1	100.0	-89.9	FW	8.646	8.5	33	N	12 ft Twall
10+60	928.053	936.57	917.761	10.3	100.0	-89.7	FW	8.517	8.5	33	N	10+00 to 10+80
10+80	927.787	936.362	917.539	10.2	100.0	-89.8	FW	8.575	8.5	33	N	
11+00	927.719	936.154	917.317	10.4	100.0	-89.6	FW	8.435	8.5	33	N	
11+20	928.007	935.947	917.096	10.9	100.0	-89.1	FW	7.94	8	33	N	10ft Twall
11+40	928.318	935.739	916.874	11.4	100.0	-88.6	FW	7.421	7.5	33	N	10+80 to 11+60
11+60	928.621	935.532	916.652	12.0	20.0	-8.0	FW	6.911	7	33	N	
11+80	928.973	935.324	916.430	12.5	18.0	-5.5	FW	6.351	6.5	33	N	
12+00	929.665	935.116	916.209	13.5	16.0	-2.5	FW	5.451	5.5	33	G	
12+20	929.609	934.909	915.987	13.6	16.0	-2.4	FW	5.3	5.5	33	G	8 ft Twall
12+40	929.752	934.701	915.765	14.0	15.0	-1.0	FW	4.949	5	33	G	11+60 to 12+80
12+60	929.374	934.494	915.543	13.8	15.0	-1.2	FW	5.12	5	33	G	
12+80	929.534	934.286	915.322	14.2	15.0	-0.8	FW	4.752	5	33	Y	
13+00	929.609	934.078	915.1	14.5	14.0	0.5	FW	4.469	4.5	33	Y	
13+20	929.704	933.871	915.030	14.7	13.0	1.7	FW	4.167	4	33	Y	
13+40	929.921	933.663	914.960	15.0	11.0	4.0	FW	3.742	3.5	33	Y	6 ft Twall w/3.25 RR
13+60	930.088	933.456	914.890	15.2	11.0	4.2	FW	3.368	3.5	33	Y	12+80 to 14+20
13+80	930.635	933.248	914.820	15.8	7.0	8.8	FW	2.613	2.5	33	Y	
14+00	930.63	933.021	914.750	15.9	7.0	8.9	FW	2.391	2.5	33	Y	
14+20	929.613	932.789	914.680	14.9	9.0	5.9	FW	3.176	3	33	Y	
14+40	927.886	932.557	914.610	13.3	14.0	-0.7	FW	4.671	4.5	33	Y	
14+60	927.505	932.324	914.540	13.0	15.0	-2.0	FW	4.819	5	None	G	8 ft Twall
14+80	927.515	932.092	914.470	13.0	14.0	-1.0	FW	4.577	4.5	None	Y	14+20 to 14+80
15+00	927.862	931.859	914.400	13.5	13.0	0.5	FW	3.997	4	None	Y	
15+20	928.273	931.627	914.330	13.9	11.0	2.9	FW	3.354	3.5	None	Y	6 ft Twall
15+40	928.493	931.394	914.260	14.2	9.0	5.2	FW	2.901	3	None	Y	14+80 to 15+40
15+60	928.488	931.162	914.190	14.3	7.0	7.3	FW	2.674	2.5	21	Y	
15+80	928.383	930.93	914.120	14.3	7.0	7.3	FW	2.547	2.5	21	Y	
16+00	928.145	930.697	914.050	14.1	7.0	7.1	FW	2.552	2.5	21	Y	
16+20	927.654	930.465	913.980	13.7	9.0	4.7	FW	2.811	3	21	Y	6 ft Twall w/2.25 RR
16+40	927.064	930.232	913.910	13.2	9.0	4.2	FW	3.168	3	21	Y	15+40 to 17+00

Alignment	Existing Elev (ft)	Proposed Elev (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer	Levee, FW or None	Height of protection abv ground	Nearest 0.5	Riprap	Iwall Feasibility	Template Ranges (original)
16+60	926.287	930	913.840	12.4	11.0	1.4	FW	3.713	3.5	21	Y	
16+80	925.78	929.771	913.770	12.0	13.0	-1.0	FW	3.991	4	21	Y	
17+00	925.215	929.545	913.700	11.5	14.0	-2.5	FW	4.33	4.5	21	G	
17+20	924.507	929.319	913.630	10.9	15.0	-4.1	FW	4.812	5	21	N	8 ft Twall w/2.25 RR
17+40	924.516	929.093	913.560	11.0	14.0	-3.0	FW	4.577	4.5	21	N	17+00 to 17+40
17+60	924.625	928.867	913.490	11.1	13.0	-1.9	FW	4.242	4	21	G	
17+80	924.778	928.641	913.420	11.4	13.0	-1.6	FW	3.863	4	21	G	
18+00	924.822	928.415	913.350	11.5	11.0	0.5	FW	3.593	3.5	21	Y	
18+20	924.685	928.189	913.280	11.4	11.0	0.4	FW	3.504	3.5	21	Y	6 ft Twallw/2.25 RR
18+40	924.777	927.963	913.210	11.6	9.0	2.6	FW	3.186	3	21	Y	17+40 to 19+30.37
18+60	924.398	927.738	913.140	11.3	11.0	0.3	FW	3.34	3.5	21	Y	
18+80	923.991	927.512	913.070	10.9	11.0	-0.1	FW	3.521	3.5	21	Y	
19+00	923.677	927.286	913.000	10.7	11.0	-0.3	FW	3.609	3.5	21	Y	
19+20	922.765	927.06	912.930	9.8	14.0	-4.2	FW	4.295	4.5	None	N	6 ft Twall
19+40	922.308	926.834	912.860	9.4	14.0	-4.6	FW	4.526	4.5	None	N	19+30.37 to 19.54.88
19+60	922.624	926.608	912.790	9.8	13.0	-3.2	FW	3.984	4	None	N	
19+80	922.514	926.394	912.720	9.8	13.0	-3.2	FW	3.88	4	None	N	
20+00	922.497	926.315	912.650	9.8	13.0	-3.2	FW	3.818	4	None	N	
20+20	922.019	926.236	912.580	9.4	13.0	-3.6	FW	4.217	4	None	N	4 ft Twall
20+40	923.058	926.157	912.510	10.5	9.0	1.5	FW	3.099	3	21	Y	19+54.88 to 21+00
20+60	923.921	926.078	912.440	11.5	6.0	5.5	FW	2.157	2	21	Y	
20+80	924.457	925.998	912.370	12.1	5.0	7.1	FW	1.541	1.5	21	Y	
21+00	924.985	925.919	912.3	12.7	4.0	8.7	FW	0.934	1	21	Y	
21+20	925.691	925.84	912.126	13.6	0.0	13.6	None	0.149	0	21	NA	
21+40	924.76	925.761	911.952	12.8	4.0	8.8	None	1.001	1	21	NA	
21+60	922.816	925.682	911.778	11.0	9.0	2.0	None	2.866	3	21	NA	
21+80	922.456	925.603	911.603	10.9	9.0	1.9	None	3.147	3	21	NA	
22+00	922.121	925.524	911.429	10.7	11.0	-0.3	None	3.403	3.5	33	NA	
22+20	921.823	925.444	911.255	10.6	11.0	-0.4	None	3.621	3.5	33	NA	
22+40	921.492	925.365	911.081	10.4	13.0	-2.6	None	3.873	4	33	NA	
22+60	920.786	925.286	910.907	9.9	14.0	-4.1	None	4.5	4.5	33	NA	
22+80	920.285	925.207	910.733	9.6	15.0	-5.4	None	4.922	5	33	NA	
23+00	919.576	925.128	910.559	9.0	16.0	-7.0	None	5.552	5.5	33	NA	
23+20	921.014	925.049	910.384	10.6	13.0	-2.4	None	4.035	4	33	NA	
23+40	921.024	924.969	910.210	10.8	13.0	-2.2	None	3.945	4	33	NA	
23+60	920.364	924.89	910.036	10.3	14.0	-3.7	None	4.526	4.5	33	NA	
23+80	919.913	924.811	909.862	10.1	15.0	-4.9	None	4.898	5	33	NA	
24+00	919.385	924.732	909.688	9.7	16.0	-6.3	None	5.347	5.5	33	NA	
24+20	918.858	924.636	909.514	9.3	17.0	-7.7	None	5.778	6	33	NA	
24+40	918.587	924.53	909.340	9.2	17.0	-7.8	None	5.943	6	33	NA	
24+60	918.414	924.423	909.166	9.2	17.0	-7.8	None	6.009	6	33	NA	
24+80	918.321	924.316	908.991	9.3	17.0	-7.7	None	5.995	6	33	NA	
25+00	918.261	924.21	908.817	9.4	17.0	-7.6	None	5.949	6	33	NA	
25+20	918.177	924.103	908.643	9.5	17.0	-7.5	None	5.926	6	33	NA	
25+40	918.007	923.996	908.469	9.5	17.0	-7.5	None	5.989	6	33	NA	
25+60	917.798	923.889	908.295	9.5	17.0	-7.5	None	6.091	6	33	NA	
25+80	917.465	923.783	908.121	9.3	18.0	-8.7	None	6.318	6.5	33	NA	
26+00	917.699	923.676	907.947	9.8	17.0	-7.2	None	5.977	6	33	NA	
26+20	917.664	923.569	907.772	9.9	17.0	-7.1	None	5.905	6	33	NA	
26+40	917.523	923.462	907.598	9.9	17.0	-7.1	None	5.939	6	33	NA	
26+60	917.294	923.356	907.424	9.9	17.0	-7.1	None	6.062	6	No Riprap	NA	
26+80	917.285	923.249	907.250	10.0	17.0	-7.0	None	5.964	6	No Riprap	NA	
27+00	917.415	923.142	907.076	10.3	16.0	-5.7	None	5.727	5.5	No Riprap	NA	
27+20	917.193	923.035	906.902	10.3	17.0	-6.7	None	5.842	6	No Riprap	NA	
27+40	916.709	922.929	906.728	10.0	17.0	-7.0	None	6.22	6	No Riprap	NA	
27+60	917.511	922.822	906.553	11.0	16.0	-5.0	None	5.311	5.5	No Riprap	NA	
27+80	917.71	922.715	906.379	11.3	15.0	-3.7	None	5.005	5	21	NA	
28+00	917.4	922.67	906.205	11.2	16.0	-4.8	None	5.27	5.5	21	NA	
28+20	916.775	922.634	906.031	10.7	17.0	-6.3	None	5.859	6	21	NA	
28+40	916.342	922.599	905.857	10.5	18.0	-7.5	None	6.257	6.5	21	NA	
28+60	915.965	922.563	905.683	10.3	18.0	-7.7	None	6.598	6.5	21	NA	
28+80	916.211	922.527	905.509	10.7	18.0	-7.3	None	6.316	6.5	21	NA	
29+00	916.424	922.492	905.334	11.1	17.0	-5.9	None	6.068	6	21	NA	
29+20	916.384	922.456	905.160	11.2	17.0	-5.8	None	6.072	6	21	NA	
29+40	916.416	922.421	904.986	11.4	17.0	-5.6	None	6.005	6	21	NA	
29+60	916.323	922.385	904.812	11.5	17.0	-5.5	None	6.062	6	21	NA	
29+80	915.935	922.35	904.638	11.3	18.0	-6.7	None	6.415	6.5	21	NA	
30+00	915.677	922.314	904.464	11.2	18.0	-6.8	None	6.637	6.5	21	NA	
30+20	915.781	922.279	904.290	11.5	18.0	-6.5	None	6.498	6.5	21	NA	
30+40	915.848	922.243	904.116	11.7	18.0	-6.3	None	6.395	6.5	21	NA	
30+60	915.812	922.208	903.941	11.9	18.0	-6.1	None	6.396	6.5	21	NA	None
30+80	916.214	922.137	903.767	12.4	17.0	-4.6	None	5.923	6	21	NA	
31+00	917.115	922.057	903.593	13.5	15.0	-1.5	None	4.942	5	21	NA	
31+20	917.952	921.977	903.419	14.5	13.0	1.5	None	4.025	4	21	NA	
31+40	918.156	921.897	903.245	14.9	11.0	3.9	None	3.741	3.5	No Riprap	NA	
31+60	918.447	921.816	903.071	15.4	11.0	4.4	None	3.369	3.5	No Riprap	NA	
31+80	919.113	921.736	902.897	16.2	7.0	9.2	None	2.623	2.5	No Riprap	NA	
32+00	919.668	921.656	902.722	16.9	6.0	10.9	None	1.988	2	No Riprap	NA	
32+20	919.619	921.576	902.548	17.1	6.0	11.1	None	1.957	2	No Riprap	NA	
32+40	919.364	921.495	902.374	17.0	6.0	11.0	None	2.131	2	No Riprap	NA	
32+60	918.036	921.415	902.2	15.8	11.0	4.8	None	3.379	3.5	No Riprap	NA	
32+80	916.945	921.335	902.252	14.7	14.0	0.7	None	4.39	4.5	No Riprap	NA	
33+00	915.38	921.259	902.304	13.1	17.0	-3.9	None	5.879	6	No Riprap	NA	
33+20	914.17	921.187	902.356	11.8	20.0	-8.2	None	7.017	7	No Riprap	NA	
33+40	913.867	921.115	902.407	11.5	20.0	-8.5	None	7.248	7	No Riprap	NA	
33+60	913.826	921.042	902.459	11.4	20.0	-8.6	None	7.216	7	21	NA	
33+80	913.782	920.97	902.511	11.3	20.0	-8.7	None	7.188	7	21	NA	
34+00	913.766	920.898	902.563	11.2	20.0	-8.8	None	7.132	7	21	NA	
34+20	913.803	920.826	902.615	11.2	20.0	-8.8	None	7.023	7	21	NA	
34+40	913.853	920.753	902.667	11.2	20.0	-8.8	None	6.9	7	21	NA	

Alignment	Existing Elev (ft)	Proposed Elev (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer	Levee, FW or None	Height of protection abv ground	Nearest 0.5	Riprap	Iwall Feasibility	Template Ranges (original)
34+60	914.155	920.681	902.719	11.4	18.0	-6.6	None	6.526	6.5	21	NA	
34+80	914.306	920.609	902.770	11.5	18.0	-6.5	None	6.303	6.5	21	NA	
35+00	914.382	920.536	902.822	11.6	17.0	-5.4	None	6.154	6	21	NA	
35+20	914.505	920.464	902.874	11.6	17.0	-5.4	None	5.959	6	21	NA	
35+40	914.191	920.392	902.926	11.3	17.0	-5.7	None	6.201	6	21	NA	
35+60	914.145	920.32	902.978	11.2	17.0	-5.8	None	6.175	6	21	NA	
35+80	913.849	920.247	903.030	10.8	18.0	-7.2	None	6.398	6.5	21	NA	
36+00	913.791	920.198	903.081	10.7	18.0	-7.3	None	6.407	6.5	21	NA	
36+20	913.652	920.19	903.133	10.5	18.0	-7.5	None	6.538	6.5	21	NA	
36+40	913.582	920.183	903.185	10.4	18.0	-7.6	None	6.601	6.5	21	NA	
36+60	913.752	920.176	903.237	10.5	18.0	-7.5	None	6.424	6.5	21	NA	
36+80	914.059	920.169	903.289	10.8	17.0	-6.2	None	6.11	6	21	NA	
37+00	914.227	920.162	903.341	10.9	17.0	-6.1	None	5.935	6	21	NA	
37+20	914.244	920.155	903.393	10.9	17.0	-6.1	None	5.911	6	21	NA	
37+40	914.259	920.148	903.444	10.8	17.0	-6.2	None	5.889	6	No Riprap	NA	
37+60	914.52	920.141	903.496	11.0	16.0	-5.0	None	5.621	5.5	No Riprap	NA	
37+80	914.924	920.134	903.548	11.4	15.0	-3.6	None	5.21	5	No Riprap	NA	
38+00	915.045	920.127	903.6	11.4	15.0	-3.6	None	5.082	5	No Riprap	NA	
38+20	915.287	920.12	903.296	12.0	15.0	-3.0	None	4.833	5	No Riprap	NA	
38+40	915.44	920.113	902.992	12.4	14.0	-1.6	None	4.673	4.5	No Riprap	NA	
38+60	915.69	920.105	902.688	13.0	14.0	-1.0	None	4.415	4.5	No Riprap	NA	
38+80	915.863	920.098	902.384	13.5	13.0	0.5	None	4.235	4	No Riprap	NA	
39+00	915.792	920.091	902.080	13.7	14.0	-0.3	None	4.299	4.5	No Riprap	NA	
39+20	915.759	920.084	901.776	14.0	14.0	0.0	None	4.325	4.5	No Riprap	NA	
39+40	915.644	920.077	901.472	14.2	14.0	0.2	None	4.433	4.5	No Riprap	NA	
39+60	915.543	920.07	901.168	14.4	14.0	0.4	None	4.527	4.5	No Riprap	NA	
39+80	915.27	920.063	900.864	14.4	15.0	-0.6	None	4.793	5	No Riprap	NA	
40+00	915.006	920.056	900.560	14.4	15.0	-0.6	FW	5.05	5	No Riprap	Y	Iwall
40+20	914.814	920.049	900.256	14.6	15.0	-0.4	FW	5.235	5	No Riprap	Y	40+10.91 to 40+30.91
40+40	914.632	920.042	899.952	14.7	16.0	-1.3	FW	5.41	5.5	No Riprap	G	
40+60	914.123	920.035	899.648	14.5	17.0	-2.5	FW	5.912	6	No Riprap	G	8 ft Twall
40+80	914.171	920.028	899.344	14.8	17.0	-2.2	FW	5.857	6	No Riprap	G	40+30.91 to 41+10.91
41+00	914.631	920.02	899.040	15.6	16.0	-0.4	FW	5.389	5.5	No Riprap	Y	
41+20	915.369	920.013	898.736	16.6	14.0	2.6	FW	4.644	4.5	No Riprap	Y	Iwall
41+40	916.684	920.006	898.432	18.3	11.0	7.3	FW	3.322	3.5	No Riprap	Y	41+10.91 to 41+79.08
41+60	919.056	919.995	898.128	20.9	4.0	16.9	FW	0.939	1	No Riprap	Y	
41+80	919.312	919.944	897.824	21.5	3.0	18.5	None	0.632	0.5	No Riprap	NA	
42+00	919.74	919.893	897.520	22.2	0.0	22.2	None	0.153	0	No Riprap	NA	
42+20	920.023	919.843	897.216	22.8	0.0	22.8	None	-0.18	0	No Riprap	NA	
42+40	919.69	919.792	896.912	22.8	0.0	22.8	None	0.102	0	No Riprap	NA	
42+60	919.616	919.742	896.608	23.0	0.0	23.0	None	0.126	0	No Riprap	NA	
42+80	919.97	919.691	896.304	23.7	0.0	23.7	None	-0.279	0	No Riprap	NA	
43+00	918.817	919.641	896	22.8	4.0	18.8	None	0.824	1	No Riprap	NA	
43+20	919.416	919.59	895.920	23.5	0.0	23.5	None	0.174	0	No Riprap	NA	
43+40	920.131	919.539	895.840	24.3	0.0	24.3	None	-0.592	0	No Riprap	NA	
43+60	922.227	919.489	895.760	26.5	0.0	26.5	None	-2.738	0	No Riprap	NA	
43+80	923.542	919.438	895.680	27.9	0.0	27.9	None	-4.104	0	No Riprap	NA	
44+00	920.322	919.388	895.600	24.7	0.0	24.7	None	-0.934	0	No Riprap	NA	
44+20	919.931	919.337	895.520	24.4	0.0	24.4	None	-0.594	0	No Riprap	NA	
44+40	918.621	919.287	895.440	23.2	3.0	20.2	None	0.666	0.5	No Riprap	NA	
44+60	916.654	919.236	895.360	21.3	7.0	14.3	None	2.582	2.5	No Riprap	NA	
44+80	912.262	919.185	895.280	17.0	20.0	-3.0	None	6.923	7	No Riprap	NA	
45+00	906.633	919.135	895.200	11.4	100.0	-88.6	None	12.502	12.5	No Riprap	NA	
45+20	905.677	919.084	895.120	10.6	100.0	-89.4	None	13.407	13.5	21	NA	
45+40	906.007	919.034	895.040	11.0	100.0	-89.0	None	13.027	13	21	NA	
45+60	906.349	918.983	894.960	11.4	100.0	-88.6	None	12.634	12.5	21	NA	
45+80	906.75	918.933	894.880	11.9	100.0	-88.1	None	12.183	12	21	NA	
46+00	907.277	918.882	894.800	12.5	100.0	-87.5	None	11.605	11.5	21	NA	
46+20	907.888	918.831	894.720	13.2	100.0	-86.8	None	10.943	11	21	NA	
46+40	908.176	918.781	894.640	13.5	100.0	-86.5	None	10.605	10.5	21	NA	
46+60	908.524	918.73	894.560	14.0	100.0	-86.0	None	10.206	10	21	NA	
46+80	908.658	918.68	894.480	14.2	100.0	-85.8	None	10.022	10	21	NA	
47+00	909.422	918.629	894.400	15.0	100.0	-85.0	None	9.207	9	21	NA	
47+20	910.265	918.579	894.320	15.9	100.0	-84.1	None	8.314	8.5	21	NA	
47+40	911.241	918.528	894.240	17.0	100.0	-83.0	None	7.287	7.5	21	NA	
47+60	912.189	918.477	894.160	18.0	18.0	0.0	None	6.288	6.5	21	NA	
47+80	913.522	918.427	894.080	19.4	15.0	4.4	None	4.905	5	21	NA	
48+00	914.848	918.376	894	20.8	11.0	9.8	None	3.528	3.5	21	NA	
48+20	915.868	918.326	893.940	21.9	7.0	14.9	None	2.458	2.5	21	NA	
48+40	916.025	918.275	893.880	22.1	7.0	15.1	None	2.25	2.5	No Riprap	NA	
48+60	914.456	918.225	893.820	20.6	13.0	7.6	None	3.769	4	No Riprap	NA	
48+80	907.134	918.174	893.760	13.4	100.0	-86.6	None	11.04	11	No Riprap	NA	
49+00	905.743	918.123	893.700	12.0	100.0	-88.0	None	12.38	12.5	No Riprap	NA	
49+20	906.471	918.073	893.640	12.8	100.0	-87.2	None	11.602	11.5	No Riprap	NA	
49+40	906.045	918.022	893.580	12.5	100.0	-87.5	None	11.977	12	No Riprap	NA	
49+60	905.553	917.972	893.520	12.0	100.0	-88.0	None	12.419	12.5	No Riprap	NA	
49+80	905.546	917.921	893.460	12.1	100.0	-87.9	None	12.375	12.5	No Riprap	NA	
50+00	905.527	917.871	893.400	12.1	100.0	-87.9	None	12.344	12.5	No Riprap	NA	
50+20	905.344	917.82	893.340	12.0	100.0	-88.0	None	12.476	12.5	No Riprap	NA	
50+40	905.123	917.769	893.280	11.8	100.0	-88.2	None	12.646	12.5	No Riprap	NA	
50+60	905.809	917.719	893.220	12.6	100.0	-87.4	None	11.91	12	No Riprap	NA	
50+80	907.18	917.668	893.160	14.0	100.0	-86.0	None	10.488	10.5	No Riprap	NA	
51+00	907.134	917.618	893.100	14.0	100.0	-86.0	None	10.484	10.5	No Riprap	NA	
51+20	906.759	917.567	893.040	13.7	100.0	-86.3	None	10.808	11	No Riprap	NA	
51+40	906.355	917.517	892.980	13.4	100.0	-86.6	None	11.162	11	No Riprap	NA	
51+60	906.416	917.466	892.920	13.5	100.0	-86.5	None	11.05	11	No Riprap	NA	
51+80	906.974	917.415	892.860	14.1	100.0	-85.9	None	10.441	10.5	No Riprap	NA	
52+00	904.856	917.365	892.800	12.1	100.0	-87.9	None	12.509	12.5	No Riprap	NA	
52+20	907.707	917.314	892.740	15.0	100.0	-85.0	None	9.607	9.5	No Riprap	NA	
52+40	907.316	917.264	892.680	14.6	100.0	-85.4	None	9.948	10	No Riprap	NA	

Alignment	Existing Elev (ft)	Proposed Elev (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer	Levee, FW or None	Height of protection abv ground	Nearest 0.5	Riprap	Iwall Feasibility	Template Ranges (original)
52+60	906.752	917.213	892.620	14.1	100.0	-85.9	None	10.461	10.5	No Riprap	NA	
52+80	906.046	917.162	892.560	13.5	100.0	-86.5	None	11.116	11	No Riprap	NA	
53+00	905.763	917.112	892.500	13.3	100.0	-86.7	None	11.349	11.5	No Riprap	NA	
53+20	905.554	917.061	892.440	13.1	100.0	-86.9	None	11.507	11.5	No Riprap	NA	None
53+40	905.392	917.011	892.380	13.0	100.0	-87.0	None	11.619	11.5	No Riprap	NA	
53+60	905.032	916.96	892.320	12.7	100.0	-87.3	None	11.928	12	No Riprap	NA	
53+80	904.813	916.91	892.260	12.6	100.0	-87.4	None	12.097	12	No Riprap	NA	
54+00	904.943	916.859	892.200	12.7	100.0	-87.3	None	11.916	12	No Riprap	NA	
54+20	904.751	916.808	892.140	12.6	100.0	-87.4	None	12.057	12	No Riprap	NA	
54+40	904.353	916.758	892.080	12.3	100.0	-87.7	None	12.405	12.5	No Riprap	NA	
54+60	904.474	916.707	892.020	12.5	100.0	-87.5	None	12.233	12	No Riprap	NA	
54+80	904.525	916.657	891.960	12.6	100.0	-87.4	None	12.132	12	No Riprap	NA	
55+00	904.395	916.606	891.900	12.5	100.0	-87.5	None	12.211	12	No Riprap	NA	
55+20	904.036	916.556	891.840	12.2	100.0	-87.8	None	12.52	12.5	No Riprap	NA	
55+40	903.924	916.505	891.780	12.1	100.0	-87.9	None	12.581	12.5	No Riprap	NA	
55+60	903.816	916.454	891.720	12.1	100.0	-87.9	None	12.638	12.5	No Riprap	NA	
55+80	903.955	916.404	891.660	12.3	100.0	-87.7	None	12.449	12.5	No Riprap	NA	
56+00	903.992	916.353	891.600	12.4	100.0	-87.6	None	12.361	12.5	No Riprap	NA	
56+20	904.043	916.303	891.540	12.5	100.0	-87.5	None	12.26	12.5	No Riprap	NA	
56+40	904.182	916.252	891.480	12.7	100.0	-87.3	None	12.07	12	No Riprap	NA	
56+60	904.348	916.202	891.420	12.9	100.0	-87.1	None	11.854	12	No Riprap	NA	
56+80	904.469	916.151	891.360	13.1	100.0	-86.9	None	11.682	11.5	No Riprap	NA	
57+00	904.975	916.1	891.300	13.7	100.0	-86.3	None	11.125	11	No Riprap	NA	
57+20	906.479	916.05	891.240	15.2	100.0	-84.8	None	9.571	9.5	No Riprap	NA	
57+40	908.535	915.999	891.180	17.4	100.0	-82.6	None	7.464	7.5	No Riprap	NA	
57+60	910.42	915.949	891.120	19.3	16.0	3.3	None	5.529	5.5	No Riprap	NA	
57+80	910.995	915.898	891.060	19.9	15.0	4.9	None	4.903	5	No Riprap	NA	
58+00	911.417	915.848	891	20.4	14.0	6.4	None	4.431	4.5	No Riprap	NA	
58+20	912.031	915.797	891.181	20.9	13.0	7.9	None	3.766	4	No Riprap	NA	
58+40	912.139	915.746	891.362	20.8	11.0	9.8	None	3.607	3.5	No Riprap	NA	
58+60	912.198	915.696	891.542	20.7	11.0	9.7	None	3.498	3.5	No Riprap	NA	
58+80	912.26	915.645	891.723	20.5	11.0	9.5	None	3.385	3.5	No Riprap	NA	
59+00	912.266	915.595	891.904	20.4	11.0	9.4	None	3.329	3.5	No Riprap	NA	
59+20	912.276	915.544	892.085	20.2	11.0	9.2	None	3.268	3.5	No Riprap	NA	
59+40	912.017	915.494	892.265	19.8	11.0	8.8	None	3.477	3.5	No Riprap	NA	
59+60	911.602	915.443	892.446	19.2	13.0	6.2	None	3.841	4	No Riprap	NA	
59+80	911.12	915.392	892.627	18.5	14.0	4.5	None	4.272	4.5	No Riprap	NA	
60+00	912.12	915.342	892.808	19.3	9.0	10.3	None	3.222	3	No Riprap	NA	
60+20	913.118	915.291	892.988	20.1	6.0	14.1	None	2.173	2	No Riprap	NA	
60+40	911.984	915.241	893.169	18.8	11.0	7.8	None	3.257	3.5	No Riprap	NA	
60+60	910.681	915.19	893.350	17.3	14.0	3.3	None	4.509	4.5	No Riprap	NA	
60+80	910.381	915.14	893.531	16.9	15.0	1.9	None	4.759	5	No Riprap	NA	
61+00	910.23	915.089	893.712	16.5	15.0	1.5	None	4.859	5	No Riprap	NA	
61+20	910.077	915.038	893.892	16.2	15.0	1.2	None	4.961	5	No Riprap	NA	
61+40	909.71	914.988	894.073	15.6	16.0	-0.4	None	5.278	5.5	No Riprap	NA	
61+60	911.037	914.937	894.254	16.8	13.0	3.8	None	3.9	4	No Riprap	NA	
61+80	910.814	914.887	894.435	16.4	13.0	3.4	None	4.073	4	No Riprap	NA	
62+00	911.013	914.836	894.615	16.4	13.0	3.4	None	3.823	4	No Riprap	NA	
62+20	912.354	914.786	894.796	17.6	7.0	10.6	None	2.432	2.5	No Riprap	NA	
62+40	913.924	914.735	894.977	18.9	4.0	14.9	None	0.811	1	No Riprap	NA	
62+60	915.618	914.684	895.158	20.5	0.0	20.5	None	-0.934	0	No Riprap	NA	
62+80	916.129	914.634	895.338	20.8	0.0	20.8	None	-1.495	0	No Riprap	NA	
63+00	916.792	914.583	895.519	21.3	0.0	21.3	None	-2.209	0	No Riprap	NA	
63+20	918.966	914.533	895.7	23.3	0.0	23.3	None	-4.433	0	No Riprap	NA	
63+40	918.71	914.482	895.432	23.3	0.0	23.3	None	-4.228	0	No Riprap	NA	
63+60	920.661	914.432	895.164	25.5	0.0	25.5	None	-6.229	0	No Riprap	NA	
63+80	923.058	914.381	894.896	28.2	0.0	28.2	None	-8.677	0	No Riprap	NA	
64+00	918.948	914.33	894.628	24.3	0.0	24.3	None	-4.618	0	No Riprap	NA	
64+20	920.375	914.28	894.360	26.0	0.0	26.0	None	-6.095	0	No Riprap	NA	
64+40	918.036	914.229	894.092	23.9	0.0	23.9	None	-3.807	0	No Riprap	NA	
64+60	916.186	914.179	893.824	22.4	0.0	22.4	None	-2.007	0	No Riprap	NA	
64+80	912.713	914.128	893.556	19.2	5.0	14.2	Levee	1.415	1.5	No Riprap	NA	
65+00	910.075	914.077	893.288	16.8	13.0	3.8	Levee	4.002	4	No Riprap	NA	
65+20	909.323	914.027	893.020	16.3	14.0	2.3	Levee	4.704	4.5	No Riprap	NA	
65+40	908.902	913.976	892.752	16.2	15.0	1.1	Levee	5.074	5	No Riprap	NA	
65+60	909.035	913.926	892.484	16.6	15.0	1.6	Levee	4.891	5	No Riprap	NA	Levee w/3.25 RR 64+83.96 to 66+60.91
65+80	909.172	913.875	892.216	17.0	14.0	3.0	Levee	4.703	4.5	No Riprap	NA	
66+00	909.397	913.825	891.948	17.4	14.0	3.4	Levee	4.428	4.5	No Riprap	NA	
66+20	909.445	913.739	891.680	17.8	14.0	3.8	Levee	4.294	4.5	No Riprap	NA	
66+40	909.172	913.62	891.412	17.8	14.0	3.8	Levee	4.448	4.5	No Riprap	NA	
66+60	908.885	913.502	891.144	17.7	14.0	3.7	FW	4.617	4.5	No Riprap	Y	I wall
66+80	908.363	913.383	890.876	17.5	15.0	2.5	FW	5.02	5	No Riprap	Y	66+60.91 to 66+90.51
67+00	908.072	913.264	890.608	17.5	15.0	2.5	None	5.192	5	No Riprap	NA	
67+20	907.987	913.145	890.340	17.6	15.0	2.6	None	5.158	5	No Riprap	NA	
67+40	908.035	913.026	890.072	18.0	15.0	3.0	None	4.991	5	No Riprap	NA	Merriam Bridge
67+60	907.692	912.908	889.804	17.9	15.0	2.9	FW	5.216	5	No Riprap	Y	
67+80	908.356	912.789	889.536	18.8	14.0	4.8	FW	4.433	4.5	No Riprap	Y	
68+00	907.921	912.67	889.268	18.7	14.0	4.7	FW	4.749	4.5	No Riprap	Y	
68+20	907.969	912.584	889.0	19.0	14.0	5.0	FW	4.615	4.5	No Riprap	Y	
68+40	907.804	912.545	889.0	18.8	14.0	4.8	FW	4.741	4.5	No Riprap	Y	Iwall w/3.25 RR 67+56.18 to 69+19.66
68+60	907.927	912.505	889.0	18.9	14.0	4.9	FW	4.578	4.5	No Riprap	Y	
68+80	907.955	912.466	889.0	19.0	14.0	5.0	FW	4.511	4.5	No Riprap	Y	
69+00	907.912	912.427	889.0	18.9	14.0	4.9	FW	4.515	4.5	No Riprap	Y	
69+20	907.62	912.388	889.0	18.6	15.0	3.6	FW	4.768	5	No Riprap	Y	

Y	penetration available within 1 foot
G	grading or pre-drilling/excavation required
N	wall too tall or grading >3ft required
NA	Not floodwall

critical section analyzed

Alignment	Existing Elevation (ft)	Proposed Elevation (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer		Height of Protection above ground	Nearest 0.5	Riprap	Iwall Feasibility	
+	937.754	937.4	915.8	22.0	0	22.0	None	-0.354	0		NA	
+20	937.094	937.4	915.8	21.3	3	18.3	None	0.306	0.5		NA	
+40	936.441	937.4	915.8	20.6	4	16.6	None	0.959	1		NA	
+60	935.804	937.4	915.8	20.0	5	15.0	None	1.596	1.5		NA	
+80	935.291	937.4	915.8	19.5	6	13.5	None	2.109	2		NA	
1+00	934.967	937.4	915.8	19.2	7	12.2	None	2.433	2.5		NA	
1+20	934.606	937.4	915.8	18.8	9	9.8	None	2.794	3		NA	
1+40	934.181	937.4	915.8	18.4	9	9.4	None	3.219	3		NA	
1+60	934.091	937.375	915.8	18.3	11	7.3	None	3.284	3.5		NA	
1+80	933.859	937.08	915.8	18.1	9	9.1	None	3.221	3		NA	
2+00	933.864	936.786	915.8	18.1	9	9.1	None	2.922	3		NA	
2+20	933.953	936.491	915.8	18.2	7	11.2	None	2.538	2.5		NA	
2+40	934.514	936.197	915.8	18.7	5	13.7	None	1.683	1.5		NA	
2+60	934.756	935.902	915.8	19.0	4	15.0	None	1.146	1		NA	
2+80	934.908	935.608	915.8	19.1	3	16.1	None	0.7	0.5		NA	
3+00	934.978	935.313	915.8	19.2	3	16.2	None	0.335	0.5		NA	
3+20	934.165	935.019	915.8	18.4	4	14.4	None	0.854	1		NA	
3+40	932.196	934.725	915.8	16.4	7	9.4	None	2.529	2.5		NA	
3+60	933.522	934.43	915.8	17.7	4	13.7	None	0.908	1		NA	
3+80	932.426	934.136	915.8	16.6	5	11.6	None	1.71	1.5		NA	
4+00	932.192	933.841	915.8	16.4	5	11.4	None	1.649	1.5		NA	
4+20	932.361	933.547	915.8	16.6	4	12.6	None	1.186	1		NA	
4+40	930.938	933.252	915.8	15.1	7	8.1	None	2.314	2.5		NA	
4+60	929.414	933.04	915.8	13.6	11	2.6	None	3.626	3.5		NA	
4+80	929.898	932.844	915.8	14.1	9	5.1	None	2.946	3		NA	
5+00	930.892	932.649	915.8	15.1	6	9.1	None	1.757	2		NA	
5+20	931.519	932.454	915.8	15.7	4	11.7	None	0.935	1		NA	
5+40	930.93	932.259	915.8	15.1	5	10.1	None	1.329	1.5		NA	
5+60	929.475	932.063	**915.8**	13.7	7	6.7	FW	2.588	2.5	No Riprap	Y	
5+80	928.055	931.868	915.7	12.4	13	-0.6	FW	3.813	4	No Riprap	Y	6ft FW NO Riprap
6+00	928.101	931.673	915.6	12.5	11	1.5	FW	3.572	3.5	No Riprap	Y	5+71.53 to 6+40
6+20	927.227	931.478	915.5	11.7	14	-2.3	FW	4.251	4.5	No Riprap	G	
6+40	926.135	931.283	915.4	10.7	15	-4.3	FW	5.148	5	No Riprap	N	8ft FW NO Riprap
6+60	925.353	931.087	915.3	10.1	16	-5.9	FW	5.734	5.5	No Riprap	N	6+40 to 6+80
6+80	923.383	930.892	915.2	8.2	30	-21.8	FW	7.509	7.5	2.25	N	10ft FW w/21 Riprap
7+00	923.365	930.697	915.1	8.3	30	-21.7	FW	7.332	7.5	2.25	N	
7+20	923.391	930.502	915	8.4	20	-11.6	FW	7.111	7	2.25	N	6+80 to 7+60
7+40	923.457	930.306	914.9	8.6	20	-11.4	FW	6.849	7	2.25	N	
7+60	923.514	930.111	914.8	8.7	18	-9.3	FW	6.597	6.5	2.25	N	
7+80	923.66	929.874	914.7	9.0	17	-8.0	FW	6.214	6	2.25	N	
8+00	923.872	929.634	914.6	9.3	17	-7.7	FW	5.762	6	2.25	N	8ft FW w/21 Riprap
8+20	923.919	929.393	914.5	9.4	16	-6.6	FW	5.474	5.5	2.25	N	7+60 to 8+80
8+40	924.01	929.153	914.4	9.6	15	-5.4	FW	5.143	5	2.25	N	
8+60	924.093	928.913	914.3	9.8	15	-5.2	FW	4.82	5	2.25	N	
8+80	924.283	928.673	914.2	10.1	14	-3.9	FW	4.39	4.5	2.25	N	
9+00	923.94	928.433	914.1	9.8	14	-4.2	FW	4.493	4.5	2.25	N	
9+20	923.875	928.193	914	9.9	14	-4.1	FW	4.318	4.5	2.25	N	
9+40	923.573	927.953	913.9	9.7	14	-4.3	FW	4.38	4.5	2.25	N	
9+60	923.332	927.712	913.8	9.5	14	-4.5	FW	4.38	4.5	2.25	N	
9+80	923.356	927.472	913.7	9.7	13	-3.3	FW	4.116	4	2.25	N	
10+00	923.271	927.232	913.6	9.7	13	-3.3	FW	3.961	4	2.25	N	6 ft FW w/21 Riprap
10+20	923.084	926.992	913.5	9.6	13	-3.4	FW	3.908	4	2.25	N	
10+40	922.941	926.752	913.4	9.5	13	-3.5	FW	3.811	4	2.25	N	8+80 to 11+85.05
10+60	922.759	926.512	913.3	9.5	13	-3.5	FW	3.753	4	2.25	N	
10+80	922.393	926.342	913.2	9.2	13	-3.8	FW	3.949	4	2.25	N	
11+00	922.408	926.235	913.1	9.3	13	-3.7	FW	3.827	4	2.25	N	
11+20	922.331	926.127	913	9.3	13	-3.7	FW	3.796	4	2.25	N	
11+40	922.054	926.019	912.9	9.2	13	-3.8	FW	3.965	4	2.25	N	
11+60	922.632	925.912	912.8	9.8	11	-1.2	FW	3.28	3.5	2.25	G	
11+80	923.356	925.804	912.7	10.7	7	3.7	Levee	2.448	2.5	2.25	NA	
12+00	924.475	925.696	912.6	11.9	4	7.9	Levee	1.221	1	2.25	NA	
12+20	924.617	925.589	912.5	12.1	4	8.1	Levee	0.972	1	2.25	NA	
12+40	924.551	925.481	912.4	12.2	4	8.2	Levee	0.93	1	2.25	NA	
12+60	923.948	925.374	912.3	11.6	5	6.6	Levee	1.426	1.5	2.25	NA	
12+80	922.559	925.266	912.2	10.4	7	3.4	Levee	2.707	2.5	2.25	NA	
13+00	921.379	925.158	912.1	9.3	13	-3.7	Levee	3.779	4	2.25	NA	
13+20	920.845	925.051	912	8.8	13	-4.2	Levee	4.206	4	2.25	NA	
13+40	920.353	924.943	911.9	8.5	14	-5.5	Levee	4.59	4.5	2.25	NA	
13+60	921.02	924.835	911.8	9.2	13	-3.8	Levee	3.815	4	2.25	NA	
13+80	921.525	924.728	**911.7**	9.8	9	0.8	Levee	3.203	3	2.25	NA	
14+00	922.14	924.62	911.5952381	10.5	7	3.5	Levee	2.48	2.5	2.25	NA	
14+20	***921.838***	924.512	911.4904762	10.3	7	3.3	Levee	2.674	2.5	2.25	NA	
14+40	***921.838***	924.405	911.3857143	10.5	7	3.5	Levee	2.567	2.5	2.25	NA	Levee w/21 Riprap
14+60	***921.838***	924.297	911.2809524	10.6	7	3.6	Levee	2.459	2.5	2.25	NA	
14+80	***921.838***	924.189	911.1761905	10.7	7	3.7	Levee	2.351	2.5	2.25	NA	11+85.05 to 17+75
15+00	***921.838***	924.082	911.0714286	10.8	6	4.8	Levee	2.244	2	2.25	NA	
15+20	921.838	923.974	910.9666667	10.9	6	4.9	Levee	2.136	2	2.25	NA	

Alignment	Existing Elevation (ft)	Proposed Elevation (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer		Height of Protection above ground	Nearest 0.5	Riprap	Iwall Feasibility	
15+40	921.551	923.866	910.8619048	10.7	7	3.7	Levee	2.315	2.5	2.25	NA	
15+60	921.687	923.759	910.7571429	10.9	6	4.9	Levee	2.072	2	2.25	NA	
15+80	919.424	923.651	910.652381	8.8	13	-4.2	Levee	4.227	4	2.25	NA	
16+00	918.691	923.543	910.547619	8.1	15	-6.9	Levee	4.852	5	2.25	NA	
16+20	918.225	923.436	910.4428571	7.8	15	-7.2	Levee	5.211	5	2.25	NA	
16+40	917.906	923.328	910.3380952	7.6	16	-8.4	Levee	5.422	5.5	2.25	NA	
16+60	917.694	923.22	910.2333333	7.5	16	-8.5	Levee	5.526	5.5	2.25	NA	
16+80	917.718	923.113	910.1285714	7.6	16	-8.4	Levee	5.395	5.5	2.25	NA	
17+00	918.036	923.005	910.0238095	8.0	15	-7.0	Levee	4.969	5	2.25	NA	
17+20	918.181	922.897	909.9190476	8.3	14	-5.7	Levee	4.716	4.5	2.25	NA	
17+40	917.879	922.79	909.8142857	8.1	15	-6.9	Levee	4.911	5	2.25	NA	
17+60	917.514	922.682	909.7095238	7.8	15	-7.2	Levee	5.168	5	2.25	NA	
17+80	916.926	922.575	909.6047619	7.3	16	-8.7	Levee	5.649	5.5	No Riprap	NA	
18+00	915.479	922.484	**909.5**	6.0	20	-14.0	Levee	7.005	7	No Riprap	NA	
18+20	914.517	922.431	909.2230769	5.3	30	-24.7	Levee	7.914	8	No Riprap	NA	
18+40	915.082	922.378	908.9461538	6.1	30	-23.9	Levee	7.296	7.5	No Riprap	NA	Levee NO Riprap
18+60	915.367	922.325	908.6692308	6.7	20	-13.3	Levee	6.958	7	No Riprap	NA	17+75 to 19+45.28
18+80	915.538	922.272	908.3923077	7.1	18	-10.9	Levee	6.734	6.5	No Riprap	NA	
19+00	916.002	922.219	908.1153846	7.9	17	-9.1	Levee	6.217	6	No Riprap	NA	
19+20	916.649	922.166	907.8384615	8.8	16	-7.2	Levee	5.517	5.5	No Riprap	NA	
19+40	916.867	922.114	907.5615385	9.3	15	-5.7	FW	5.247	5	No Riprap	N	
19+60	917.313	922.061	907.2846154	10.0	14	-4.0	FW	4.748	4.5	No Riprap	N	
19+80	917.177	922.008	907.0076923	10.2	15	-4.8	FW	4.831	5	No Riprap	N	
20+00	917.263	921.955	906.7307692	10.5	14	-3.5	FW	4.692	4.5	No Riprap	N	
20+20	917.216	921.902	906.4538462	10.8	14	-3.2	FW	4.686	4.5	No Riprap	N	
20+40	916.902	921.849	906.1769231	10.7	15	-4.3	FW	4.947	5	No Riprap	N	8ft FW NO Riprap
20+60	916.699	921.796	905.9	10.8	15	-4.2	FW	5.097	5	No Riprap	N	19+45.28 to 21+59.23
20+80	916.576	921.743	905.6230769	11.0	15	-4.0	FW	5.167	5	No Riprap	N	
21+00	916.884	921.691	905.3461538	11.5	15	-3.5	FW	4.807	5	No Riprap	N	
21+20	917.138	921.638	905.0692308	12.1	14	-1.9	FW	4.5	4.5	No Riprap	G	
21+40	917.1	921.585	904.7923077	12.3	14	-1.7	FW	4.485	4.5	No Riprap	G	
21+60	917.041	921.532	904.5153846	12.5	14	-1.5	FW	4.491	4.5	33	G	
21+80	916.948	921.479	904.2384615	12.7	14	-1.3	FW	4.531	4.5	33	G	
22+00	917.026	921.426	903.9615385	13.1	14	-0.9	FW	4.4	4.5	33	Y	
22+20	916.882	921.373	903.6846154	13.2	14	-0.8	FW	4.491	4.5	33	Y	
22+40	916.721	921.32	903.4076923	13.3	14	-0.7	FW	4.599	4.5	33	Y	
22+60	915.949	921.267	903.1307692	12.8	16	-3.2	FW	5.318	5.5	33	N	10ft FW w/33 Riprap
22+80	914.641	921.215	902.8538462	11.8	18	-6.2	FW	6.574	6.5	33	N	
23+00	913.687	921.162	902.5769231	11.1	30	-18.9	FW	7.475	7.5	33	N	21+59.23 to 24+00
23+20	913.537	921.109	**902.3**	11.2	30	-18.8	FW	7.572	7.5	33	N	
23+40	913.603	921.056	902.1037037	11.5	30	-18.5	FW	7.453	7.5	33	N	
23+60	914.173	921.003	901.9074074	12.3	20	-7.7	FW	6.83	7	33	N	
23+80	914.693	920.95	901.7111111	13.0	18	-5.0	FW	6.257	6.5	33	N	
24+00	914.533	920.897	901.5148148	13.0	18	-5.0	FW	6.364	6.5	33	N	12 ft FW w/33 Riprap
24+20	914.541	920.844	901.3185185	13.2	18	-4.8	FW	6.303	6.5	33	N	24+00 to 24+40.47
24+40	915.8	920.792	901.1222222	14.7	15	-0.3	FW	4.992	5	33	Y	8ft FW w/33 Riprap
24+60	914.913	920.739	900.9259259	14.0	17	-3.0	FW	5.826	6	33	N	24+40.47 to 25+00.47
24+80	915.255	920.686	900.7296296	14.5	16	-1.5	FW	5.431	5.5	21	G	
25+00	915.398	920.633	900.5333333	14.9	15	-0.1	FW	5.235	5	21	Y	8ft FW/21 Riprap
25+20	918.178	920.58	900.337037	17.8	7	10.8	FW	2.402	2.5	21	Y	25+00.47 to 25+60.47
25+40	917.759	920.527	900.1407407	17.6	9	8.6	FW	2.768	3	21	Y	
25+60	0	920.474	899.9444444	-899.9	100	-999.9	FW	920.474	920.5	21	N	10ft FW/w 21 Riprap
25+80	913.63	920.421	899.7481481	13.9	20	-6.1	FW	6.791	7	21	N	25+60.47 to 26+20.48
26+00	913.976	920.369	899.5518519	14.4	18	-3.6	FW	6.393	6.5	21	N	
26+20	914.197	920.316	899.3555556	14.8	17	-2.2	FW	6.119	6	21	G	
26+40	915.064	920.263	899.1592593	15.9	15	0.9	FW	5.199	5	21	Y	
26+60	914.061	920.21	898.962963	15.1	17	-1.9	FW	6.149	6	21	G	8ft FW/w 21 Riprap
26+80	914.061	920.2	898.7666667	15.3	17	-1.7	FW	6.139	6	21	G	26+20.48 to 27+40.48
27+00	914.142	920.2	898.5703704	15.6	17	-1.4	FW	6.058	6	21	G	
27+20	914.175	920.2	898.3740741	15.8	17	-1.2	FW	6.025	6	21	G	
27+40	913.489	920.2	898.1777778	15.3	18	-2.7	FW	6.711	6.5	21	G	
27+60	913.273	920.2	897.9814815	15.3	20	-4.7	FW	6.927	7	21	N	10ft FW/w 21 Riprap
27+80	913.321	920.2	897.7851852	15.5	20	-4.5	FW	6.879	7	21	N	27+40.48 to 28+40.48
28+00	912.744	920.2	897.5888889	15.2	30	-14.8	FW	7.456	7.5	21	N	
28+20	911.818	920.2	897.3925926	14.4	30	-15.6	FW	8.382	8.5	21	N	
28+40	910.89	920.2	897.1962963	13.7	30	-16.3	FW	9.31	9.5	21	N	
28+60	911.352	920.2	**897**	14.4	30	-15.6	FW	8.848	9	21	N	
28+80	911.851	920.2	896.5272727	15.3	30	-14.7	FW	8.349	8.5	21	N	
29+00	911.984	920.2	896.0545455	15.9	30	-14.1	FW	8.216	8	21	N	
29+20	912.434	920.2	895.5818182	16.9	30	-13.1	FW	7.766	8	21	N	
29+40	911.419	920.2	895.1090909	16.3	30	-13.7	FW	8.781	9	21	N	
29+60	912.024	920.2	894.6363636	17.4	30	-12.6	FW	8.176	8	21	N	
29+80	911.644	920.2	894.1636364	17.5	30	-12.5	FW	8.556	8.5	21	N	
30+00	911.605	920.2	893.6909091	17.9	30	-12.1	FW	8.595	8.5	21	N	
30+20	911.493	920.2	893.2181818	18.3	30	-11.7	FW	8.707	8.5	21	N	
30+40	911.346	920.2	892.7454545	18.6	30	-11.4	FW	8.854	9	21	N	
30+60	911.279	920.2	892.2727273	19.0	30	-11.0	FW	8.921	9	21	N	12ft FW/w 21 Riprap
30+80	911.12	920.2	**891.8**	19.3	30	-10.7	FW	9.08	9	21	N	28+40.48 to 33+00.76
31+00	910.889	920.2	891.6846154	19.2	30	-10.8	FW	9.311	9.5	21	N	
31+20	910.998	920.2	891.5692308	19.4	30	-10.6	FW	9.202	9	21	N	
31+40	911.212	920.2	891.4538462	19.8	30	-10.2	FW	8.988	9	21	N	
31+60	911.29	920.2	891.3384615	20.0	30	-10.0	FW	8.91	9	21	N	
31+80	911.439	920.2	891.2230769	20.2	30	-9.8	FW	8.761	9	21	N	
32+00	911.617	920.2	891.1076923	20.5	30	-9.5	FW	8.583	8.5	21	N	

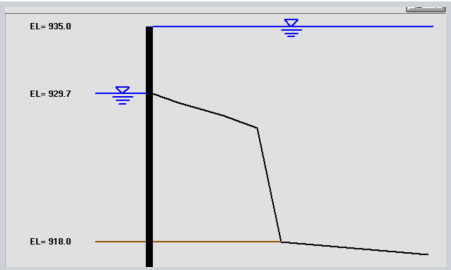
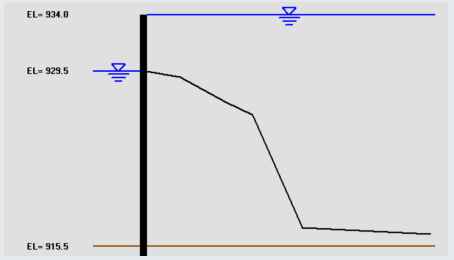
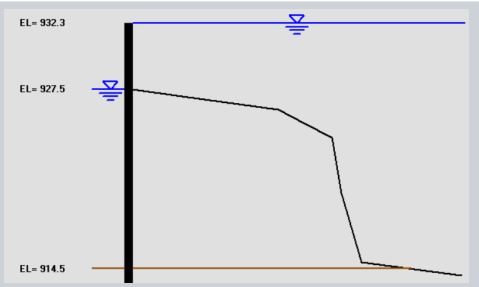
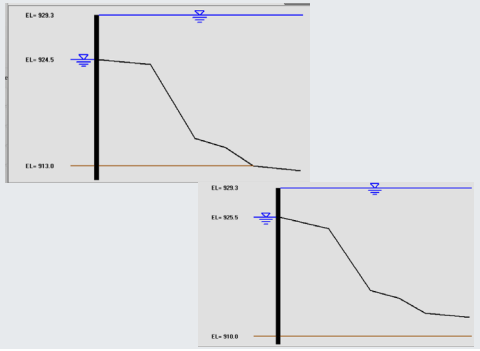
Alignment	Existing Elevation (ft)	Proposed Elevation (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer		Height of Protection above ground	Nearest 0.5	Riprap	Iwall Feasibility	
32+20	911.836	920.2	890.9923077	20.8	30	-9.2	FW	8.364	8.5	21	N	
32+40	912.051	920.2	890.8769231	21.2	30	-8.8	FW	8.149	8	21	N	
32+60	912.268	920.2	890.7615385	21.5	30	-8.5	FW	7.932	8	21	N	
32+80	912.561	920.184	890.6461538	21.9	30	-8.1	FW	7.623	7.5	21	N	
33+00	913.481	920.09	890.5307692	23.0	18	5.0	none	6.609	6.5	21	NA	
33+20	914.388	919.996	890.4153846	24.0	16	8.0	none	5.608	5.5	21	NA	Johnson Drive Bridge
33+40	914.746	919.902	890.3	24.4	15	9.4	none	5.156	5	21	NA	
33+60	914.51	919.808	890.1846154	24.3	16	8.3	FW	5.298	5.5	21	Y	
33+80	909.416	919.714	890.0692308	19.3	100	-80.7	FW	10.298	10.5	21	N	
34+00	911.35	919.62	889.9538462	21.4	30	-8.6	FW	8.27	8.5	21	N	
34+20	911.535	919.526	889.8384615	21.7	30	-8.3	FW	7.991	8	21	N	
34+40	910.66	919.431	889.7230769	20.9	30	-9.1	FW	8.771	9	21	N	
34+60	910.634	919.337	889.6076923	21.0	30	-9.0	FW	8.703	8.5	21	N	
34+80	910.081	919.243	889.4923077	20.6	30	-9.4	FW	9.162	9	21	N	
35+00	909.302	919.149	889.3769231	19.9	30	-10.1	FW	9.847	10	21	N	
35+20	909.091	919.055	889.2615385	19.8	30	-10.2	FW	9.964	10	21	N	12ft FW/w 21 Riprap
35+40	908.916	918.961	889.1461538	19.8	30	-10.2	FW	10.045	10	21	N	33+75.62 to 37+20.62
35+60	909.321	918.867	889.0307692	20.3	30	-9.7	FW	9.546	9.5	21	N	
35+80	909.289	918.772	888.9153846	20.4	30	-9.6	FW	9.483	9.5	21	N	
36+00	909.527	918.678	**888.8**	20.7	30	-9.3	FW	9.151	9	21	N	
36+20	909.24	918.584	888.7458333	20.5	30	-9.5	FW	9.344	9.5	21	N	
36+40	909.85	918.49	888.6916667	21.2	30	-8.8	FW	8.64	8.5	21	N	
36+60	910.066	918.396	888.6375	21.4	30	-8.6	FW	8.33	8.5	21	N	
36+80	0	918.302	888.5833333	-888.6	100	-988.6	FW	918.302	918.5	21	N	
37+00	909.763	918.208	888.5291667	21.2	30	-8.8	FW	8.445	8.5	21	N	
37+20	909.894	918.114	888.475	21.4	30	-8.6	Levee	8.22	8	21	NA	
37+40	909.833	918.019	888.4208333	21.4	30	-8.6	Levee	8.186	8	21	NA	
37+60	909.757	917.925	888.3666667	21.4	30	-8.6	Levee	8.168	8	21	NA	
37+80	909.682	917.831	888.3125	21.4	30	-8.6	Levee	8.149	8	21	NA	
38+00	909.644	917.737	888.2583333	21.4	30	-8.6	Levee	8.093	8	21	NA	
38+20	909.591	917.7	888.2041667	21.4	30	-8.6	Levee	8.109	8	21	NA	
38+40	909.557	917.7	888.15	21.4	30	-8.6	Levee	8.143	8	21	NA	Levee w/21 Riprap
38+60	909.64	917.7	888.0958333	21.5	30	-8.5	Levee	8.06	8	21	NA	
38+80	0	917.7	888.0416667	-888.0	100	-988.0	Levee	917.7	917.5	21	NA	37+20.62 to 40+00.53
39+00	0	917.7	887.9875	-888.0	100	-988.0	Levee	917.7	917.5	21	NA	
39+20	910.993	917.7	887.9333333	23.1	18	5.1	Levee	6.707	6.5	21	NA	
39+40	909.429	917.7	887.8791667	21.5	30	-8.5	Levee	8.271	8.5	21	NA	
39+60	909.336	917.7	887.825	21.5	30	-8.5	Levee	8.364	8.5	21	NA	
39+80	909.01	917.7	887.7708333	21.2	30	-8.8	Levee	8.69	8.5	21	NA	
40+00	910.057	917.7	887.7166667	22.3	30	-7.7	Levee	7.643	7.5	No Riprap	NA	
40+20	909.853	917.7	887.6625	22.2	30	-7.8	Levee	7.847	8	No Riprap	NA	
40+40	909.677	917.7	887.6083333	22.1	30	-7.9	Levee	8.023	8	No Riprap	NA	
40+60	909.306	917.7	887.5541667	21.8	30	-8.2	Levee	8.394	8.5	No Riprap	NA	
40+80	909.862	917.7	**887.5**	22.4	30	-7.6	Levee	7.838	8	No Riprap	NA	
41+00	908.779	917.7	887.71875	21.1	30	-8.9	Levee	8.921	9	No Riprap	NA	
41+20	908.212	917.7	887.9375	20.3	30	-9.7	Levee	9.488	9.5	No Riprap	NA	
41+40	907.592	917.7	888.15625	19.4	30	-10.6	Levee	10.108	10	No Riprap	NA	
41+60	907.722	917.7	888.375	19.3	30	-10.7	Levee	9.978	10	No Riprap	NA	
41+80	907.985	917.7	888.59375	19.4	30	-10.6	Levee	9.715	9.5	No Riprap	NA	
42+00	908.255	917.7	888.8125	19.4	30	-10.6	Levee	9.445	9.5	No Riprap	NA	
42+20	908.97	917.7	889.03125	19.9	30	-10.1	Levee	8.73	8.5	No Riprap	NA	Levee NO RIPRAP
42+40	909.242	917.7	889.25	20.0	30	-10.0	Levee	8.458	8.5	No Riprap	NA	
42+60	911.549	917.7	889.46875	22.1	17	5.1	Levee	6.151	6	No Riprap	NA	40+00.53 to 44+80
42+80	911.148	917.647	889.6875	21.5	18	3.5	Levee	6.499	6.5	No Riprap	NA	
43+00	912.442	917.569	889.90625	22.5	15	7.5	Levee	5.127	5	No Riprap	NA	
43+20	912.103	917.491	890.125	22.0	16	6.0	Levee	5.388	5.5	No Riprap	NA	
43+40	910.611	917.414	890.34375	20.3	20	0.3	Levee	6.803	7	No Riprap	NA	
43+60	909.302	917.336	890.5625	18.7	30	-11.3	Levee	8.034	8	No Riprap	NA	
43+80	909.087	917.258	890.78125	18.3	30	-11.7	Levee	8.171	8	No Riprap	NA	
44+00	909.034	917.18	**891**	18.0	30	-12.0	Levee	8.146	8	No Riprap	NA	
44+20	909.069	917.102	890.974359	18.1	30	-11.9	Levee	8.033	8	No Riprap	NA	
44+40	909.103	917.025	890.9487179	18.2	30	-11.8	Levee	7.922	8	No Riprap	NA	
44+60	908.783	916.947	890.9230769	17.9	30	-12.1	Levee	8.164	8	No Riprap	NA	
44+80	907.053	916.869	890.8974359	16.2	30	-13.8	Levee	9.816	10	21	NA	
45+00	906.856	916.791	890.8717949	16.0	30	-14.0	Levee	9.935	10	21	NA	
45+20	907.089	916.713	890.8461538	16.2	30	-13.8	Levee	9.624	9.5	21	NA	
45+40	907.131	916.636	890.8205128	16.3	30	-13.7	Levee	9.505	9.5	21	NA	
45+60	907.222	916.558	890.7948718	16.4	30	-13.6	Levee	9.336	9.5	21	NA	
45+80	908.082	916.465	890.7692308	17.3	30	-12.7	Levee	8.383	8.5	21	NA	
46+00	910.638	916.328	890.7435897	19.9	16	3.9	Levee	5.69	5.5	21	NA	
46+20	911.242	916.19	890.7179487	20.5	15	5.5	Levee	4.948	5	21	NA	Levee w/21 Riprap
46+40	911.097	916.053	890.6923077	20.4	15	5.4	Levee	4.956	5	21	NA	
46+60	911.012	915.916	890.6666667	20.3	15	5.3	Levee	4.904	5	21	NA	44+80 to 48+19.31
46+80	910.943	915.779	890.6410256	20.3	15	5.3	Levee	4.836	5	21	NA	
47+00	910.929	915.642	890.6153846	20.3	14	6.3	Levee	4.713	4.5	21	NA	
47+20	910.558	915.504	890.5897436	20.0	15	5.0	Levee	4.946	5	21	NA	
47+40	910.563	915.367	890.5641026	20.0	15	5.0	Levee	4.804	5	21	NA	
47+60	911.051	915.23	890.5384615	20.5	13	7.5	Levee	4.179	4	21	NA	
47+80	910.71	915.093	890.5128205	20.2	14	6.2	Levee	4.383	4.5	21	NA	
48+00	910.629	914.956	890.4871795	20.1	14	6.1	Levee	4.327	4.5	21	NA	
48+20	910.548	914.818	890.4615385	20.1	14	6.1	Levee	4.27	4.5	33	NA	
48+40	910.453	914.681	890.4358974	20.0	13	7.0	Levee	4.228	4	33	NA	Levee w/33 Riprap
48+60	910.509	914.544	890.4102564	20.1	13	7.1	Levee	4.035	4	33	NA	
48+80	910.572	914.407	890.3846154	20.2	13	7.2	Levee	3.835	4	33	NA	48+19.31 to 49+15.34

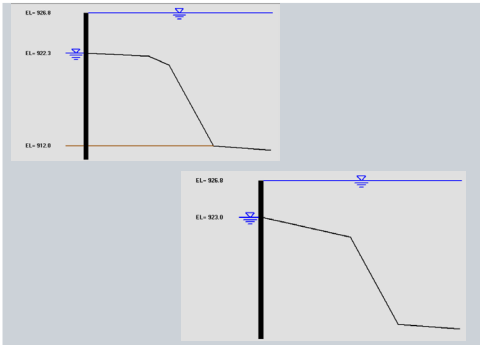
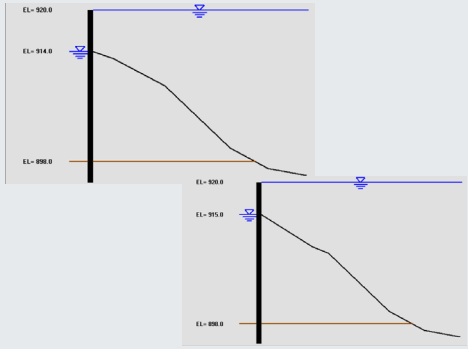
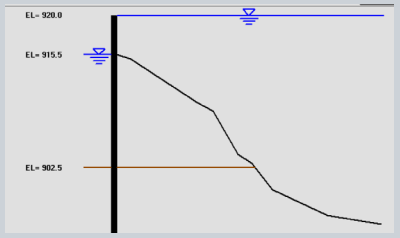
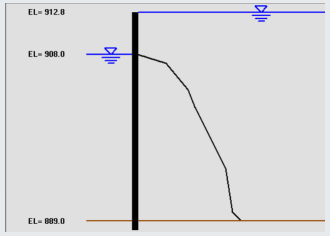
Alignment	Existing Elevation (ft)	Proposed Elevation (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	Buffer		Height of Protection above ground	Nearest 0.5	Riprap	Iwall Feasibility	
49+00	910.503	914.27	890.3589744	20.1	13	7.1	Levee	3.767	4	33	NA	
49+20	910.594	914.132	890.3333333	20.3	11	9.3	FW	3.538	3.5	33	Y	
49+40	910.5	913.995	890.3076923	20.2	11	9.2	FW	3.495	3.5	33	Y	
49+60	910.781	913.898	890.2820513	20.5	9	11.5	FW	3.117	3	33	Y	
49+80	911.104	913.893	890.2564103	20.8	9	11.8	FW	2.789	3	33	Y	**IWALL** w/33 Riprap
50+00	910.761	913.887	890.2307692	20.5	9	11.5	FW	3.126	3	33	Y	
50+20	910.734	913.881	890.2051282	20.5	9	11.5	FW	3.147	3	33	Y	49+15.34 to 51+00
50+40	910.711	913.875	890.1794872	20.5	9	11.5	FW	3.164	3	33	Y	
50+60	910.483	913.87	890.1538462	20.3	11	9.3	FW	3.387	3.5	33	Y	
50+80	910.372	913.864	890.1282051	20.2	11	9.2	FW	3.492	3.5	33	Y	
51+00	910.304	913.858	890.1025641	20.2	11	9.2	FW	3.554	3.5	33	Y	
51+20	910.195	913.852	890.0769231	20.1	11	9.1	FW	3.657	3.5	21	Y	IWALL w/21 Riprap
51+40	910.13	913.847	890.0512821	20.1	11	9.1	FW	3.717	3.5	21	Y	
51+60	910.12	913.841	890.025641	20.1	11	9.1	FW	3.721	3.5	21	Y	51+00 to 52+23
51+80	910.031	913.835	**890**	20.0	13	7.0	FW	3.804	4	21	Y	
52+00	909.555	913.83	889.9333333	19.6	14	5.6	FW	4.275	4.5	21	Y	
52+20	907.976	913.824	889.8666667	18.1	17	1.1	FW	5.848	6	21	Y	12 ft FW w/21 Riprap
52+40	906.536	913.818	889.8	16.7	30	-13.3	FW	7.282	7.5	21	N	52+23 t 53+18.00
52+60	904.963	913.812	889.7333333	15.2	30	-14.8	FW	8.849	9	21	N	
52+80	904.73	913.807	889.6666667	15.1	30	-14.9	FW	9.077	9	21	N	
53+00	906.372	913.801	889.6	16.8	30	-13.2	none	7.429	7.5	21	NA	
53+20	907.996	913.696	889.5333333	18.5	16	2.5	none	5.7	5.5		NA	Merriam Drive Bridge
53+40	907.68	913.573	889.4666667	18.2	17	1.2	none	5.893	6		NA	
53+60	908.005	913.451	889.4	18.6	16	2.6	none	5.446	5.5		NA	
53+80	907.702	913.329	889.3333333	18.4	16	2.4	fw	5.627	5.5		Y	
54+00	907.727	913.204	889.2666667	18.5	16	2.5	FW	5.477	5.5	21	Y	8ft FW w/21 Riprap
54+20	907.878	913.079	889.2	18.7	15	3.7	FW	5.201	5	21	Y	53+92.36 to 54+60.86
54+40	908.113	912.953	889.1333333	19.0	15	4.0	FW	4.84	5	21	Y	
54+60	908.057	912.828	889.0666667	19.0	15	4.0	FW	4.771	5	42	Y	8ft FW w/3.25 RR
54+80	907.931	912.694	889	18.9	15	3.9	FW	4.763	5	42	Y	54+60.86 to 55+00
55+00	907.972	912.557	888.9333333	19.0	14	5.0	FW	4.585	4.5	42	Y	
55+20	908.031	912.42	888.8666667	19.2	14	5.2	FW	4.389	4.5	42	Y	
55+40	908.107	912.283	888.8	19.3	13	6.3	FW	4.176	4	42	Y	
55+60	908.179	912.146	888.7333333	19.4	13	6.4	FW	3.967	4	42	Y	
55+80	908.2	912.01	888.6666667	19.5	13	6.5	FW	3.81	4	42	Y	
56+00	907.556	911.873	888.6	19.0	14	5.0	FW	4.317	4.5	42	Y	
56+20	907.612	911.736	888.5333333	19.1	13	6.1	FW	4.124	4	42	Y	
56+40	908.384	911.599	888.4666667	19.9	9	10.9	FW	3.215	3	42	Y	Iwall w/3.25 RR
56+60	908.743	911.5	888.4	20.3	9	11.3	FW	2.757	3	42	Y	55+00 to 57+19.13
56+80	908.942	911.5	888.3333333	20.6	7	13.6	FW	2.558	2.5	42	Y	
57+00	909.608	911.5	888.2666667	21.3	6	15.3	FW	1.892	2	42	Y	
57+20	909.936	911.5	888.2	21.7	5	16.7	FW	1.564	1.5	42	Y	
57+40	910.671	911.5	888.1333333	22.5	4	18.5	FW	0.829	1	42	Y	
57+60	911.17	911.5	888.0666667	23.1	3	20.1	FW	0.33	0.5	42	Y	
57+80	911.413	911.5	**888**	23.4	0	23.4	FW	0.087	0	42	Y	

Analysis of Critical Sections

Alignment	Approx. UTC Station	Existing Elev (ft)	Proposed Elev (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	CWALSHT modified penetration	Height of protection abv ground	Nearest 0.5	Riprap	Template Ranges (original)	comments
1+40		938.775	939	920.2	18.6	0.0		0.225	0	None		tieback reach not analyzed
1+60		938.718	939	920.2	18.5	3.0		0.282	0.5			
1+80		938.104	939	920.2	17.9	4.0		0.896	1			
2+00		937.546	939	920.2	17.3	5.0		1.454	1.5			
2+20		937.774	939	920.2	17.6	4.0		1.226	1		I wall	
2+40		937.509	939	920.2	17.3	5.0		1.491	1.5		1+40 to 3+20	
2+60		936.644	939	920.2	16.4	7.0		2.356	2.5			
2+80		934.205	939	920.2	14.0	15.0		4.795	5	21		
3+00		934.39	939	920.2	14.2	14.0		4.61	4.5	21		
3+20		934.191	939	920.2	14.0	15.0		4.809	5	21		
12+00	11+50	929.665	935.116	916.209	13.5	16.0	13.3	5.451	5.5	33		good
12+20		929.609	934.909	915.987	13.6	16.0		5.3	5.5	33	8 ft Twall	
12+40		929.752	934.701	915.765	14.0	15.0		4.949	5	33	11+60 to 12+80	
12+60		929.374	934.494	915.543	13.8	15.0		5.12	5	33		
12+80		929.534	934.286	915.322	14.2	15.0		4.752	5	33		
13+00	12+60	929.609	934.078	915.1	14.5	14.0	13.5	4.469	4.5	33		good
13+20		929.704	933.871	915.030	14.7	13.0		4.167	4	33		
13+40		929.921	933.663	914.960	15.0	11.0		3.742	3.5	33	6 ft Twall w/3.25 RR	
13+60		930.088	933.456	914.890	15.2	11.0		3.368	3.5	33	12+80 to 14+20	
13+80		930.635	933.248	914.820	15.8	7.0		2.613	2.5	33		
14+00		930.63	933.021	914.750	15.9	7.0		2.391	2.5	33		
14+20		929.613	932.789	914.680	14.9	9.0		3.176	3	33		
14+40		927.886	932.557	914.610	13.3	14.0		4.671	4.5	33	8 ft Twall	
14+60	14+40	927.505	932.324	914.540	13.0	15.0	12.9	4.819	5	None	14+20 to 14+80	good
14+80		927.515	932.092	914.470	13.0	14.0		4.577	4.5	None		
15+00		927.862	931.859	914.400	13.5	13.0		3.997	4	None	6 ft Twall	
15+20		928.273	931.627	914.330	13.9	11.0		3.354	3.5	None	14+80 to 15+40	
15+40		928.493	931.394	914.260	14.2	9.0		2.901	3	None		
15+60		928.488	931.162	914.190	14.3	7.0		2.674	2.5	21		
15+80		928.383	930.93	914.120	14.3	7.0		2.547	2.5	21		
16+00		928.145	930.697	914.050	14.1	7.0		2.552	2.5	21		
16+20		927.654	930.465	913.980	13.7	9.0		2.811	3	21	6 ft Twall w/2.25 RR	
16+40		927.064	930.232	913.910	13.2	9.0		3.168	3	21	15+40 to 17+00	
16+60		926.287	930	913.840	12.4	11.0		3.713	3.5	21		
16+80		925.78	929.771	913.770	12.0	13.0		3.991	4	21		
17+00		925.215	929.545	913.700	11.5	14.0		4.33	4.5	21		
17+20	16+80	**925.5**	929.319	**913**	12.5	13.0	10.7 (regraded)	3.819	4	21	8 ft Twall w/2.25 RR	works when regraded (+1 ft)
17+40		924.516	929.093	913.560	11.0	14.0		4.577	4.5	21	17+00 to 17+40	
17+60		924.625	928.867	913.490	11.1	13.0		4.242	4	21		
17+80		924.778	928.641	913.420	11.4	13.0		3.863	4	21		
18+00		924.822	928.415	913.350	11.5	11.0		3.593	3.5	21		
18+20		924.685	928.189	913.280	11.4	11.0		3.504	3.5	21	6 ft Twallw/2.25 RR	
18+40		924.777	927.963	913.210	11.6	9.0		3.186	3	21	17+40 to 19+30.37	
18+60		924.398	927.738	913.140	11.3	11.0		3.34	3.5	21		
18+80		923.991	927.512	913.070	10.9	11.0		3.521	3.5	21		
19+00		923.677	927.286	913.000	10.7	11.0		3.609	3.5	21		
19+20		922.765	927.06	912.930	9.8	14.0		4.295	4.5	None	6 ft Twall	
19+40	19+00	**923**	926.834	**912**	11.0	13.0	10.7 (regraded)	3.834	4	None	19+30.37 to 19.54.88	works when regraded (+1 ft)
19+60		922.624	926.608	912.790	9.8	13.0		3.984	4	None		
19+80		922.514	926.394	912.720	9.8	13.0		3.88	4	None		
20+00		922.497	926.315	912.650	9.8	13.0		3.818	4	None		
20+20		922.019	926.236	912.580	9.4	13.0		4.217	4	None	4 ft Twall	
20+40		923.058	926.157	912.510	10.5	9.0		3.099	3	21	19+54.88 to 21+00	
20+60		923.921	926.078	912.440	11.5	6.0		2.157	2	21		
20+80		924.457	925.998	912.370	12.1	5.0		1.541	1.5	21		
21+00		924.985	925.919	912.3	12.7	4.0		0.934	1	21		
40+00		915.006	920.056	900.560	14.4	15.0		5.05	5	No Riprap	Iwall	
40+20		914.814	920.049	900.256	14.6	15.0		5.235	5	No Riprap	40+10.91 to 40+30.91	
40+40		914.632	920.042	899.952	14.7	16.0		5.41	5.5	No Riprap		
40+60	39+90	**915**	920.035	**898**	17.0	15.0	14.3 (regraded)	5.035	5	No Riprap	8 ft Twall	works when regraded (+1 ft)
40+80		914.171	920.028	899.344	14.8	17.0		5.857	6	No Riprap	40+30.91 to 41+10.91	
41+00		914.631	920.02	899.040	15.6	16.0		5.389	5.5	No Riprap		
41+20	40+40	915.369	920.013	898.736	16.6	14.0	12.6	4.644	4.5	No Riprap	Iwall	good
41+40		916.684	920.006	898.432	18.3	11.0		3.322	3.5	No Riprap	41+10.91 to 41+79.08	
41+60		919.056	919.995	898.128	20.9	4.0		0.939	1	No Riprap		
66+60		908.885	913.502	891.144	17.7	14.0		4.617	4.5	No Riprap	I wall	
66+80		908.363	913.383	890.876	17.5	15.0		5.02	5	No Riprap	66+60.91 to 66+90.51	
67+60		907.692	912.908	889.804	17.9	15.0		5.216	5	No Riprap		
67+80	63+80	908.356	912.789	889.536	18.8	14.0	14.2	4.433	4.5	No Riprap		good
68+00		907.921	912.67	889.268	18.7	14.0		4.749	4.5	No Riprap		
68+20		907.969	912.584	889.0	19.0	14.0		4.615	4.5	No Riprap		
68+40		907.804	912.545	889.0	18.8	14.0		4.741	4.5	No Riprap	Iwall w/3.25 RR	
68+60		907.927	912.505	889.0	18.9	14.0		4.578	4.5	No Riprap	67+56.18 to 69+19.66	
68+80		907.955	912.466	889.0	19.0	14.0		4.511	4.5	No Riprap		
69+00		907.912	912.427	889.0	18.9	14.0		4.515	4.5	No Riprap		
69+20		907.62	912.388	889.0	18.6	15.0		4.768	5	No Riprap		

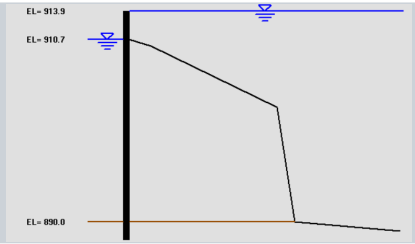
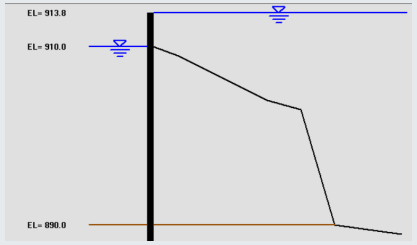
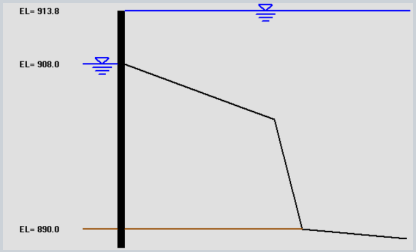
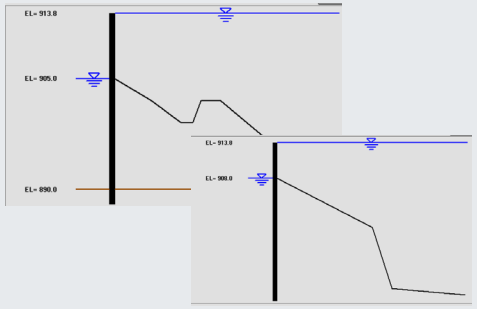
K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\6.0 Engineering-Design\6.2 Discipline\6.2.1 Geotechnical\I-Wall Design\Rotational Stability\CWALSHT runs\Jan 2026 Alignment Runs

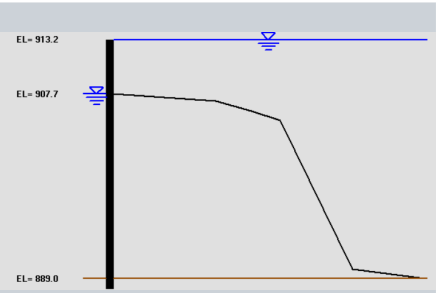
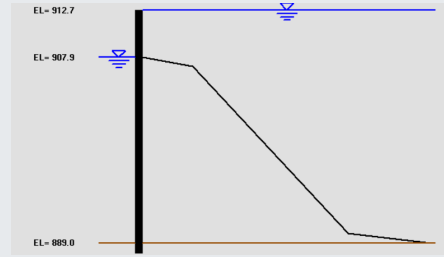
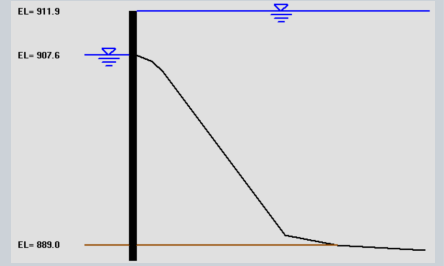
I-wall Sta (Jan2026 Alignment)	Wall Height (ft)	Bedrock Depth (ft)	Needed Penetration (ft)	Elevation Profile from CWALSHT
LB 12+00 UTC 11+50	5.5	12	13.34	
LB 13+00 UTC 12+60	4.4	14	13.54	
LB 14+60 UTC 14+40	4.8	13	12.92	
LB 17+20 UTC 16+80	4.8 3.8	11.5 12.5	12.58 13.94 (no bedrock) 10.7 (set bedrock to elev 910 -would not run otherwise)	

I-wall Sta	Wall Height (ft)	Bedrock Depth (ft)	Needed Penetration (ft)	Elevation Profile from CWALSHT
LB 19+40 UTC 19+00	4.5 3.8	10 11	11.70 10.68 (no bedrock)	
LB 40+60 UTC 39+90	6 5	16 17	17.11 (no bedrock) 14.32 (regraded, 915 GS)	
LB 41+20 UTC 40+40	4.3	13	12.64	
LB 67+80 UTC 63+80	4.5	18.8	14.2	

Alignment	UTC sta	Existing Elevation (ft)	Proposed Elevation (ft)	Assumed Bedrock Elev (ft)	Available Penetration (ft)	Needed Penetration (ft)	CWALSHT modified penetration	Height of Protection above ground	Nearest 0.5	Riprap	Original Template Ranges	comments
49+20		910.594	914.132	890.333	20.3	11		3.538	3.5	33		
49+40		910.5	913.995	890.308	20.2	11		3.495	3.5	33		
49+60		910.781	913.898	890.282	20.5	9		3.117	3	33		
49+80		911.104	913.893	890.256	20.8	9		2.789	3	33		
50+00		910.761	913.887	890.231	20.5	9		3.126	3	33	**IWALL** w/33 Riprap 49+15.34 to 51+00	
50+20		910.734	913.881	890.205	20.5	9		3.147	3	33		
50+40	59+40	910.711	913.875	890.179	20.5	9	9.2	3.164	3	33		good
50+60		910.483	913.87	890.154	20.3	11		3.387	3.5	33		
50+80		910.372	913.864	890.128	20.2	11		3.492	3.5	33		
51+00		910.304	913.858	890.103	20.2	11		3.554	3.5	33		
51+20		910.195	913.852	890.077	20.1	11		3.657	3.5	21		
51+40		910.13	913.847	890.051	20.1	11		3.717	3.5	21	IWALL w/21 Riprap 51+00 to 52+23	
51+60		910.12	913.841	890.026	20.1	11		3.721	3.5	21		
51+80	60+60	910.031	913.835	890	20.0	13	12	3.804	4	21		good
52+00		909.555	913.83	889.933	19.6	14		4.275	4.5	21		
52+20	61+30	907.976	913.824	889.867	18.1	17	16.5	5.848	6	21		good
52+40		906.536	913.818	889.800	16.7	#N/A		7.282	7.5	21	12 ft FW w/21 Riprap 52+23 t 53+18.00	
52+60	61+60	908	913.812	889.733	18.3	17	17.5 (regraded)	5.812	6	21		works when regraded +3 ft
52+80		904.73	913.807	889.667	15.1	#N/A		9.077	9	21		
53+80		907.702	913.329	889.333	18.4	16		5.627	5.5			
54+00	64+00	907.727	913.204	889.267	18.5	16	15.4	5.477	5.5	21	8ft FW w/21 Riprap 53+92.36 to 54+60.86	good
54+20		907.878	913.079	889.200	18.7	15		5.201	5	21		
54+40		908.113	912.953	889.133	19.0	15		4.84	5	21		
54+60		908.057	912.828	889.067	19.0	15		4.771	5	42	8ft FW w/3.25 RR 54+60.86 to 55+00	
54+80	64+50	907.931	912.694	889.000	18.9	15	14.9	4.763	5	42		good
55+00		907.972	912.557	888.933	19.0	14		4.585	4.5	42		
55+20		908.031	912.42	888.867	19.2	14		4.389	4.5	42		
55+40		908.107	912.283	888.800	19.3	13		4.176	4	42		
55+60		908.179	912.146	888.733	19.4	13		3.967	4	42		
55+80		908.2	912.01	888.667	19.5	13		3.81	4	42		
56+00	65+40	907.556	911.873	888.600	19.0	14	13.6	4.317	4.5	42		good
56+20		907.612	911.736	888.533	19.1	13		4.124	4	42		
56+40		908.384	911.599	888.467	19.9	9		3.215	3	42	Iwall w/3.25 RR 55+00 to 57+19.13	
56+60		908.743	911.5	888.400	20.3	9		2.757	3	42		
56+80		908.942	911.5	888.333	20.6	7		2.558	2.5	42		
57+00		909.608	911.5	888.267	21.3	6		1.892	2	42		
57+20		909.936	911.5	888.200	21.7	5		1.564	1.5	42		
57+40		910.671	911.5	888.133	22.5	4		0.829	1	42		
57+60		911.17	911.5	888.067	23.1	3		0.33	0.5	42		
57+80		911.413	911.5	888	23.4	0		0.087	0	42		

K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\6.0 Engineering-Design\6.2 Discipline\6.2.1 Geotechnical\I-Wall Design\Rotational Stability\CWALSHT runs\Jan 2026 Alignment Runs

I-wall Section	I-wall Height (ft)	Bedrock Depth (ft)	Needed Penetration (ft)	Elevation Profile
RB 50+40 UTC 59+40	3.2	20.5	9.20	
RB 51+80 UTC 60+60	3.7	27	11.99	
RB 52+20 UTC 61+30	5.8	18	16.51	
RB 52+60 UTC 61+60	9 6	15 18	20.27 17.53 (had to remove bedrock to solve)	

I-wall Section	Height (ft)	Bedrock Depth (ft)	Needed Penetration (ft)	Elevation Profile
RB 54+00 UTC 64+00	5.5	18.5	15.40	
RB 54+80 UTC 64+50	4.8	19	14.86	
RB 56+00 UTC 65+40	4.3	19	13.62	

CWALSHT Example Output
LB 12+00 (UTC 11+50)

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                        BY CLASSICAL METHODS
DATE: 29-JANUARY-2026                                        TIME: 16:11:40

                          ****************
                          *  INPUT DATA  *
                          ****************

     I.--HEADING
     'UTC I-WALL ANALYSIS

    II.--CONTROL
       CANTILEVER WALL DESIGN
       FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
       FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

   III.--WALL DATA
       ELEVATION AT TOP OF WALL    = 935.00 FT.

    IV.--SURFACE POINT DATA

         IV.A.--RIGHTSIDE
            DIST. FROM     ELEVATION
            WALL (FT)         (FT)
                 0.00        929.70
                 6.00        929.00
                15.00        928.00
                22.00        927.00
                27.00        918.00
                57.00        917.00

         IV.B.--LEFTSIDE
            DIST. FROM     ELEVATION
            WALL (FT)         (FT)
                 0.00        929.70

     V.--SOIL LAYER DATA

        V.A.--RIGHTSIDE
            LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
            LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

               ANGLE OF         ANGLE OF                          <-SAFETY->
  SAT.   MOIST INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.  WGHT. FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)  (PCF)   (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
115.00 115.00   28.00    0.00    14.00    0.00 918.00   0.00   DEF  DEF
140.00 140.00   35.00  500.00    17.50  250.00                 DEF  DEF

         V.B.--LEFTSIDE
            LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
            LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

               ANGLE OF         ANGLE OF                          <-SAFETY->
  SAT.   MOIST INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.  WGHT. FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)  (PCF)   (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
115.00 115.00   28.00    0.00    14.00    0.00 918.00   0.00   DEF  DEF
140.00 140.00   35.00  500.00    17.50  250.00                 DEF  DEF

    VI.--WATER DATA
       UNIT WEIGHT         = 62.40 (PCF)
       RIGHTSIDE ELEVATION = 935.00 (FT)
       LEFTSIDE ELEVATION  = 929.70 (FT)
       SEEPAGE ELEVATION   = 929.70 (FT)
       SEEPAGE GRADIENT    = AUTOMATIC

   VII.--VERTICAL SURCHARGE LOADS
       NONE

  VIII.--HORIZONTAL LOADS
       NONE
```

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                        BY CLASSICAL METHODS
DATE: 29-JANUARY-2026                                        TIME: 16:12:09

                          **************************
                          *   SOIL PRESSURES FOR   *
                          * CANTILEVER WALL DESIGN *
                          **************************

     I.--HEADING
     'UTC I-WALL ANALYSIS

    II.--SOIL PRESSURES

         RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

         LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.
```

SOIL PRESSURES ARE REPORTED FOR A SEEPAGE GRADIENT = 0.0001
AND MAY CHANGE WITH AUTOMATIC ADJUSTMENT OF THE GRADIENT.

ELEV. (FT)	NET WATER (PSF)	<---LEFTSIDE---> PASSIVE (PSF)	ACTIVE (PSF)	<------NET------> (SOIL + WATER) ACTIVE (PSF)	PASSIVE (PSF)	<--RIGHTSIDE---> ACTIVE (PSF)	PASSIVE (PSF)
935.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
934.0	62.4	0.0	0.0	62.4	62.4	0.0	0.0
933.0	124.8	0.0	0.0	124.8	124.8	0.0	0.0
932.0	187.2	0.0	0.0	187.2	187.2	0.0	0.0
931.0	249.6	0.0	0.0	249.6	249.6	0.0	0.0
930.0	312.0	0.0	0.0	312.0	312.0	0.0	0.0
929.7	330.7	0.0	0.0	330.7	330.7	0.0	0.0
929.0	330.7	92.4	11.7	249.2	392.3	10.8	73.3
928.7	330.7	132.0	16.6	214.2	418.7	15.5	104.7
928.0	330.7	224.4	28.3	132.6	480.3	26.3	178.0
927.0	330.7	356.3	45.0	16.1	568.6	41.8	282.8
926.9	330.7	374.6	47.3	0.0	580.8	43.9	297.4
926.0	330.7	488.3	61.6	-100.4	657.3	57.2	388.2
925.0	330.7	620.3	78.3	-216.9	746.5	72.7	494.1
924.0	330.6	752.2	94.9	-333.4	835.7	88.2	599.9
923.0	330.6	884.2	111.5	-450.0	924.7	103.6	705.7
922.0	330.6	1016.2	128.2	-566.5	1009.7	119.1	807.3
921.0	330.6	1148.2	144.8	-683.0	1091.4	134.6	905.6
920.0	330.6	1280.1	161.5	-799.5	679.4	150.0	510.3*
919.0	330.6	1412.1	178.1	-916.0	715.3	165.5	562.9*
918.1+	330.6	1528.2	192.8	-1197.6	4174.6	0.0	4287.8
918.1-	330.6	1528.2	192.8	-1197.6	4174.6	0.0	3785.9
918.0	330.6	2075.8	151.1	-1745.2	4467.3	0.0	4287.8
917.6+	330.6	6497.5	0.0	-4872.3	5685.9	0.0	5355.3
917.6-	330.6	3908.3	0.0	-4872.3	5685.9	0.0	5355.3
917.0	330.6	6497.5	0.0	-6167.0	7194.3	0.0	6863.7
916.0	330.5	6573.8	0.0	-6243.2	6223.4	0.0	5892.9
915.0	330.5	6862.2	0.0	-6531.7	6296.8	0.0	5966.3
914.0	330.5	7146.5	0.0	-6816.0	6379.9	0.0	6049.4
913.0	330.5	7427.6	0.0	-7097.1	6455.8	0.0	6125.3
912.0	330.5	7706.2	0.0	-7375.7	6518.1	0.0	6187.6
911.0	330.5	7982.7	0.0	-7652.2	5247.7	0.0	4917.2
910.0	330.5	8257.5	0.0	-7927.0	4380.4	0.0	4049.9
909.0	330.5	8531.0	0.0	-8200.5	4946.7	0.0	4616.2
908.0	330.4	8803.3	0.0	-8472.9	5167.0	0.0	4836.6
907.0	330.4	9074.8	0.0	-8744.3	5304.2	0.0	4973.7
906.0	330.4	9345.4	0.0	-9015.0	5487.7	0.0	5157.3
905.0	330.4	9615.4	0.0	-9285.0	5748.7	0.0	5418.3
904.0	330.4	9884.8	0.0	-9554.4	6009.8	0.0	5679.4
903.0	330.4	10153.8	0.0	-9823.4	6177.0	0.0	5846.6
902.0	330.4	10422.3	0.0	-10091.9	6324.8	0.0	5994.4
901.0	330.4	10690.5	0.0	-10360.1	6562.6	0.0	6232.3
900.0	330.3	10958.3	0.0	-10627.9	6865.6	0.0	6535.2
899.0	330.3	11225.9	0.0	-10895.5	7216.4	0.0	6886.1

* STANDARD WEDGE SOLUTION DOES NOT EXIST FOR INDICATED PRESSURE
FOR THIS ELEVATION.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 29-JANUARY-2026 TIME: 16:12:10

****
* SUMMARY OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'UTC I-WALL ANALYSIS

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

*****WARNING: STANDARD WEDGE SOLUTION DOES NOT EXIST
AT ALL ELEVATIONS. SEE COMPLETE OUTPUT.

WALL BOTTOM ELEV. (FT) : 916.36
PENETRATION (FT) : 13.34

MAX. BEND. MOMENT (LB-FT) : 1.0059E+04
AT ELEVATION (FT) : 921.49

MAX. SCALED DEFL. (LB-IN^3): 1.8524E+09
AT ELEVATION (FT) : 935.00

SEEPAGE GRADIENT : 0.1983

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELLASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 29-JANUARY-2026 TIME: 16:12:10

****
* COMPLETE OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'UTC I-WALL ANALYSIS

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
935.00	0.0000E+00	0.	1.8524E+09	0.00
934.00	1.0400E+01	31.	1.6999E+09	62.40
933.00	8.3200E+01	125.	1.5474E+09	124.80
932.00	2.8080E+02	281.	1.3952E+09	187.20
931.00	6.6560E+02	499.	1.2434E+09	249.60
930.00	1.3000E+03	780.	1.0928E+09	312.00
929.70	1.5483E+03	876.	1.0480E+09	330.72
929.00	2.2367E+03	1082.	9.4451E+08	256.11
928.70	2.5723E+03	1154.	9.0069E+08	224.14
928.00	3.4288E+03	1285.	8.0011E+08	149.53
927.00	4.7704E+03	1381.	6.6167E+08	42.94
926.60	5.3291E+03	1390.	6.0808E+08	0.00
926.00	6.1550E+03	1371.	5.3147E+08	-63.64
925.00	7.4759E+03	1254.	4.1190E+08	-170.22
924.00	8.6266E+03	1030.	3.0522E+08	-276.81
923.00	9.5005E+03	700.	2.1341E+08	-383.39
922.00	9.9910E+03	263.	1.3796E+08	-489.97
921.00	9.9915E+03	-280.	7.9706E+07	-596.56
920.00	9.3955E+03	-930.	3.8631E+07	-703.13
919.00	8.0964E+03	-1686.	1.3690E+07	-809.68
918.60	7.2708E+03	-2643.	7.7576E+06	-3962.56
918.00	5.1407E+03	-4194.	2.5252E+06	-1215.39
917.91	4.7766E+03	-4281.	2.0706E+06	-821.63
917.84	4.4733E+03	-4328.	1.7433E+06	-498.63
917.00	1.1034E+03	-3116.	7.0432E+04	3370.32
916.36	0.0000E+00	0.	0.0000E+00	6319.89

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELLASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

III.--WATER AND SOIL PRESSURES

		<----------SOIL PRESSURES---------->			
	WATER	<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
935.00	0.	0.	0.	0.	0.
934.00	62.	0.	0.	0.	0.
933.00	125.	0.	0.	0.	0.
932.00	187.	0.	0.	0.	0.
931.00	250.	0.	0.	0.	0.
930.00	312.	0.	0.	0.	0.
929.70	331.	0.	0.	0.	0.
929.00	313.	71.	9.	13.	91.
928.70	306.	101.	13.	19.	129.
928.00	289.	172.	22.	32.	220.
927.00	264.	273.	34.	52.	349.
926.60	254.	313.	40.	59.	402.
926.00	239.	373.	47.	71.	480.
925.00	214.	474.	60.	90.	610.
924.00	190.	575.	73.	109.	741.
923.00	165.	676.	85.	128.	872.
922.00	140.	777.	98.	147.	997.
921.00	115.	878.	111.	166.	1119.
920.00	91.	979.	124.	185.*	630.
919.00	66.	1080.	136.	204.*	695.
918.60	56.	4148.	105.	130.	2332.
918.00	41.	8733.	58.	18.	4778.
917.91+	39.	9065.	22.	0.	7245.
917.91-	39.	9065.	22.	0.	4990.
917.84+	37.	5764.	0.	0.	5164.
917.84-	37.	8810.	0.	0.	5164.
917.00	16.	5764.	0.	0.	7245.
916.36	1.	5936.	0.	0.	6290.
916.00	0.	6029.	0.	0.	6285.

* STANDARD WEDGE SOLUTION DOES NOT EXIST FOR INDICATED PRESSURE
AT THIS ELEVATION.

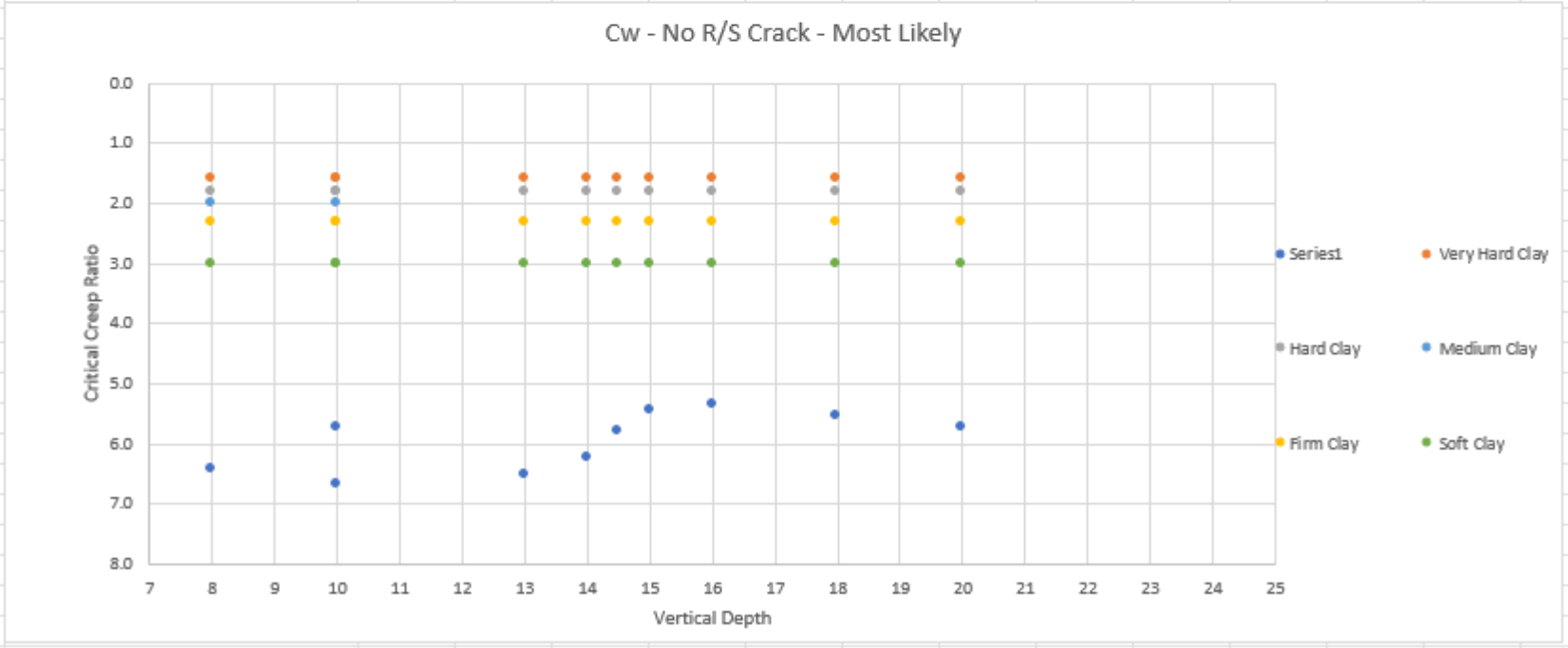
I-wall Line of Creep Analysis

No Crack vs. Vertical Crack

Using Lane's Theory

L_V for no crack scenario equals 2 x sheet pile depth (seepage has to travel full distance down and up the sheet pile)

Lane's Weighted Creep Ratio (No Crack)												
Dimension	Unit	A	B	C	D	E	F	G	H	I	J	K
Thickness of sheet pile	Feet	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125
Depth of sheet pile	Feet	6	8	10	10	13	14	14.5	15	16	18	20
Concrete - Beyond Sheet Pile Side	Feet	0	0	0	0	0	0	0	0	0	0	0
Concrete - Below Sheet Pile	feet	0	0	0	0	0	0	0	0	0	0	0
Horizontal Length	L	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125
Vertical Depth (penetration depth from CWALSHT analysis)	Feet	6	8	10	10	13	14	14.5	15	16	18	20
Head (I-wall height)	Feet	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Cw	H/L	6.0	6.4	6.7	5.7	6.5	6.2	5.8	5.5	5.3	5.5	5.7
Critical Creep Ratios												
Very Hard Clay		1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6
Hard Clay		1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8
Medium Clay		2	2	2	2	2	2	2	2	2	2	2
Firm Clay		2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
Soft Clay		3	3	3	3	3	3	3	3	3	3	3



4.16

Lane's theory resolves itself into the following equation:

$$C_w = \frac{\Sigma L_H + 3\Sigma L_V}{3H} \qquad 4.7$$

where C_w = weighted creep ratio
L_H = horizontal or flat contact distances
L_V = vertical or steep contact distances
H = head between headwater and tailwater

Material	C_w
Clean gravel	5.0
Clean sand or sand and gravel mixture	6.5
Very fine sands and silts	8.5
Well-graded mixture of sand, silt, and less than 15 percent clay	5.5
Well-graded mixture of sand, silt, and more than 15 percent clay	4.0
Firm clay	2.3
Hard clay	1.8

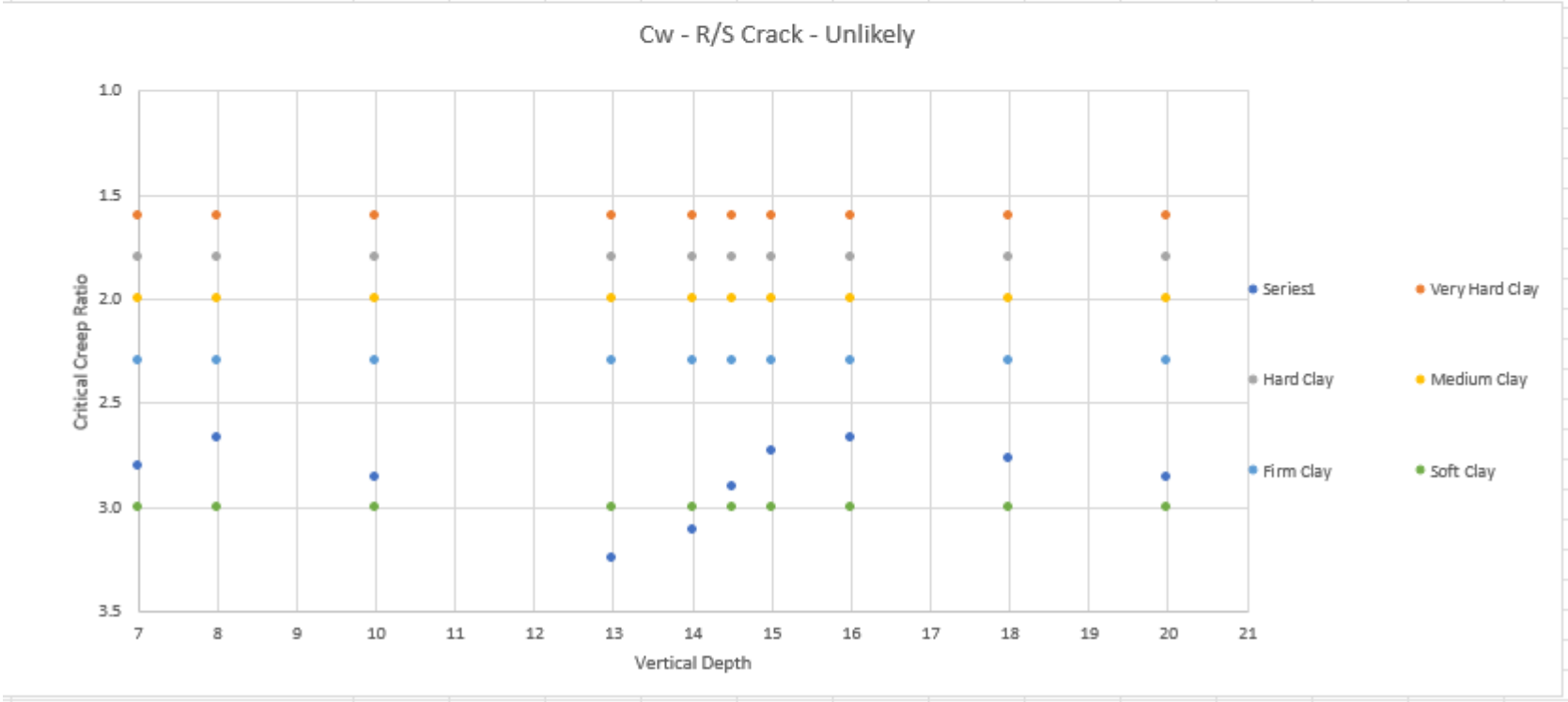
TABLE 4.3

Material	C_w
Very fine sand and silt	8.5
Fine sand	7.0
Medium sand	6.0
Course sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Course gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

Sheet pile depths required for rotational stability will satisfy creep ratio requirements

L_V for crack scenario equals 1 x sheet pile depth (seepage has half the travel distance due to the crack)

Lane's Weighted Creep Ratio (With a Crack)												
Dimension	Unit	A	B	C	D	E	F	G	H	I	J	K
Thickness of sheet pile	Feet	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125
Depth of sheet pile	Feet	6	7	8	10	13	14	14.5	15	16	18	20
Concrete - Beyond Sheet Pile Side	Feet	0	0	0	0	0	0	0	0	0	0	0
Concrete - Below Sheet Pile	feet	0	0	0	0	0	0	0	0	0	0	0
Horizontal Length	L	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125	0.03125
Vertical Depth (penetration depth from CWALSHT analysis)	Feet	6	7	8	10	13	14	14.5	15	16	18	20
Head (I-wall height)	Feet	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Cw	H/L	3.0	2.8	2.7	2.9	3.3	3.1	2.9	2.7	2.7	2.8	2.9
Critical Creep Ratios												
Very Hard Clay		1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6
Hard Clay		1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8
Medium Clay		2	2	2	2	2	2	2	2	2	2	2
Firm Clay		2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
Soft Clay		3	3	3	3	3	3	3	3	3	3	3



4.16

Lane's theory resolves itself into the following equation:

$$C_w = \frac{\Sigma L_H + 3\Sigma L_V}{3H} \qquad 4.7$$

where C_w = weighted creep ratio
L_H = horizontal or flat contact distances
L_V = vertical or steep contact distances
H = head between headwater and tailwater

Material	C_w
Clean gravel	5.0
Clean sand or sand and gravel mixture	6.5
Very fine sands and silts	8.5
Well-graded mixture of sand, silt, and less than 15 percent clay	5.5
Well-graded mixture of sand, silt, and more than 15 percent clay	4.0
Firm clay	2.3
Hard clay	1.8

TABLE 4.3

Material	C_w
Very fine sand and silt	8.5
Fine sand	7.0
Medium sand	6.0
Course sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Course gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

Sheet pile depths required for rotational stability will satisfy creep ratio requirements

APPENDIX B.4 – FLOODWALL BEARING CAPACITY

Bearing Capacity

Bearing capacity for floodwall footings was determined for drained and undrained cases, using a friction angle of 28 degrees for the drained case (see 6.9.2) and a cohesion of 1000 psf for the undrained case (see 6.9.3). A groundwater depth of 10.8 feet was used (see 6.3 and this appendix).

Floodwall continuous footing widths ranged from 7 feet to 20 feet and depth is 3 feet below adjacent finished grade (see this appendix). For bearing capacity, for drained conditions, the smallest footing size results in smallest bearing capacity for Terzaghi and Vesic methods. For bearing capacity, for undrained conditions, Terazaghi method result does not change with footing size and Vesic bearing capacity goes down as footing size increases. Showed footing widths of 7 feet for drained and 20 feet for undrained.

The undrained case controlled. An allowable bearing capacity of 2000 psf was selected for design.

Drained

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date March 23, 2026
Identification UTC Floodwalls Drained

Input

Units of Measurement
E SI or E

Foundation Information
Shape CO SQ, CI, CO, or RE
B = 7 ft
L = ft
D = 3 ft

Soil Information
c = 0 lb/ft^2
phi = 28 deg
gamma = 115 lb/ft^3
Dw = 10.8 ft

Factor of Safety
F = 3

Copyright 2000 by Donald P. Coduto

Results

	Terzaghi	Vesic
Bearing Capacity		
q ult =	12,016 lb/ft^2	12,458 lb/ft^2
q a =	4,005 lb/ft^2	4,153 lb/ft^2
Allowable Wall Load		
P/b =	28 k/ft	29 k/ft

Unit conve	1000
Gamma w	62.4
phi (radiar	0.48869
Terzaghi Computations	
a theta =	3.07372
Nc =	31.61
Nq =	17.81
N gamma	14.59
gamma' =	115
coefficient	1
coefficient	0.5
sigma zD'	345
Vesic Computation	
Nc =	25.80
sc =	1.00
dc =	1.17
Nq =	14.72
sq =	1.00
dq =	1.13
N gamma	16.72
s gamma	1.00
d gamma	1.00
B/L =	0
k =	0.42857
W sub f	0

Bearing Capacity

Undrained

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date March 23, 2026
Identification UTC Floodwalls Undrained

Input

Units of Measurement
E SI or E

Foundation Information
Shape CO SQ, CI, CO, or RE
B = 20 ft
L = ft
D = 3 ft

Soil Information
c = 1000 lb/ft^2
phi = 0 deg
gamma = 115 lb/ft^3
Dw = 10.8 ft

Factor of Safety
F = 3

Copyright 2000 by Donald P. Coduto

Results

	Terzaghi	Vesic
Bearing Capacity		
q ult =	6,045 lb/ft^2	5,793 lb/ft^2
q a =	2,015 lb/ft^2	1,931 lb/ft^2
Allowable Wall Load		
P/b =	40 k/ft	39 k/ft

Unit conve	1000
Gamma w	62.4
phi (radian	0
Terzaghi Computations	
a theta =	1
Nc =	5.70
Nq =	1.00
N gamma	0.00
gamma' =	76.936
coefficient	1
coefficient	0.5
sigma zD'	345
Vesic Computation	
Nc =	5.14
sc =	1.00
dc =	1.06
Nq =	1.00
sq =	1.00
dq =	1.00
N gamma	0.00
s gamma	1.00
d gamma	1.00
B/L =	0
k =	0.15
W sub f	0

Bearing Capacity

Groundwater Depth use 10.8 ft

Unit Weight use 115 pcf

Groundwater Depth

Boring	Ground Surface EL	Groundwater Depth	Groundwater EL	Bedrock Depth	Bedrock EL	Groundwater above Bedrock
196	910.5	21	889.5	27.8	882.7	6.8
197	909	18.5	890.5	20	889	1.5
198	909	NE		21.5	887.5	
199	910	17.9	892.1	20	890	2.1
200	910.6	10	900.6	18.8	891.8	8.8
201	913	14.7	898.3	16.5	896.5	1.8
202	913	10.3	902.7	10.3	902.7	0
203	916	NE		6.5	909.5	
204	921.4	NE		9.7	911.7	
205	928.8	10.5	918.3	13	915.8	2.5
206	931.4	NE		11.2	920.2	
207	930	NE		9.8	920.2	
208	921.8	NE		8.7	913.1	
209	913.5	NE		10	903.5	
210	914.7	NE		11.1	903.6	
211	907.7	NE		18.5	889.2	
212	906	NE		14	892	

	7 of 17 w Groundwater
	21
	18.5
	17.9
	10
	14.7
	10.3
	10.5
average	14.7
std dev	3.9
average - std dev	10.8

Sample	Dry Unit Weight (pcf)	Water Content (%)	Moist Unit Weight (pcf)
ADU-197, 5'-7'	113.1	14.6	129.6
ADU-197, 10'-12'	95.8	26	120.7
~~AD-198, 10'-12'~~	~~124.2~~	~~28.9~~	~~160.1~~
ADUC-202, 5'-7'	96.6	23.2	119
ADU-204, 2'-4'	99.7	22.2	121.8
ADU-206, 5'-7'	103.1	18.1	121.8
ADU-207, 5'-7'	92.5	25	115.6
ADU-208, 1'-3'	94.3	25.4	118.3
Average	99.3	22.1	121.0
Standard Deviation	6.5	3.9	4.4
Avg - Std Dev	92.8	18.2	116.6

Bearing Capacity

Footing Width

Widths vary from 7’ to 20’

Bank Statio	Creek Cent	WALL PROFILE	
STATION	STATION	BWIDTH	TWIDTH
		Total foundation width from front of key to back of toe	Width from back of toe to landside face of stem wall
11+80	0+20	7.00	3.00
13+00	0+29	9.00	4.00
10+20	1+90	9.00	4.00
9+00	3+33	7.00	3.00
7+80	4+52	9.00	4.00
19+00	5+77	10.00	5.50
5+20	7+32	10.00	5.00
23+40	10+29	14.00	5.00
24+20	11+16	10.00	5.50
25+60	12+67	9.00	4.00
6+20	14+98	9.00	4.00
6+60	15+34	10.00	5.50
7+00	15+50	14.00	5.00
7+40	16+28	10.00	5.50
8+60	17+12	9.00	4.00
22+60	31+05	10.00	5.50
24+40	32+61	18.00	10.00
24+40	32+61	14.00	6.50
24+40	32+61	14.00	5.00
45+00	32+67	14.00	5.00
25+00	33+21	14.00	6.50
25+20	33+40	9.00	4.00
46+20	33+90	10.00	5.50
26+00	34+17	14.00	5.00
27+20	35+55	10.00	5.50
28+40	36+75	14.00	5.00
31+00	39+14	14.00	6.50
31+80	39+86	14.00	5.00
32+20	40+24	20.00	8.00
32+80	40+77	10.00	5.00
53+00	41+00	#REF!	#REF!
35+40	43+05	20.00	8.00
49+00	57+91	9.00	4.00
49+20	58+07	10.00	5.00
52+00	60+86	9.00	4.00
52+40	61+33	10.00	5.00
52+80	61+92	14.00	5.00
79+40	64+15	10.00	5.00
54+20	64+09	10.00	5.00
54+80	64+25	9.00	4.00
79+80	64+61	9.00	4.00
56+60	65+80	7.00	3.00

"K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\6.0 Engineering-Design\6.2 Discipline\6.2.1 Geotechnical\LB & RB station ranges and elevations_20260116.xlsx"

Bearing Capacity

Footing Depth

SHEET ID

S-001

Depth is 3’

IV. FOUNDATION & SOIL DATA

A. THE FOUNDATION SYSTEM CONSISTS OF CONTINUOUS WALL FOOTINGS. THE MINIMUM DEPTH TO THE BOTTOM OF THE FOOTINGS IS 3'-0" BELOW ADJACENT FINISH GRADE,

APPENDIX B.5 – LEVEE AND FLOODWALL SETTLEMENT

Settlement Notes

Levee Settlement

Evaluated settlement of underlying soil due to loading from new levee embankment using:

- Settle3 software
- Spreadsheet using Das textbook methods

Assumed settlement of compacted levee fill will be insignificant

Floodwall Settlement

Evaluated settlement of underlying soil due to loading from new floodwall using:

- Settle3 software
- Coduto Spreadsheet

Consolidation Parameters

Compression Index Cc used average plus one standard deviation
Recompression Index Cr used average plus one standard deviation
Initial Void Ratio Eo used average minus one standard deviation

Sample	USCS	Preconsolidation Pressure (tons-force per square foot [tsf])	Overburden (tsf)	OCR	C_c	C_r	e_o
ADU-197, 5 feet–7 feet	Gravelly CL	3.5	0.39	8.9	0.11	0.01	0.503
AD-198, 10 feet–12 feet	CL	3.2	0.88	3.6	0.23	0.03	0.876
ADU-204, 2 feet–4 feet	GC w sand	2.9	0.18	16.1	0.15	0.02	0.639
ADU-206, 5 feet–7 feet	CL w sand	4.1	0.37	11.1	0.16	0.03	0.667
ADU-208, 1 foot–3 feet	CL	2.3	0.12	19.2	0.11	0.01	0.791
Average					0.15	0.02	0.695
Average plus Std Deviation for C_c and C_r, Average minus Std. Deviation for e_o					0.20	0.03	0.57

$Cc/1+e = 0.20/1.57 = 0.1274$
$Cr/1+e = 0.03/1.57 = 0.0191$

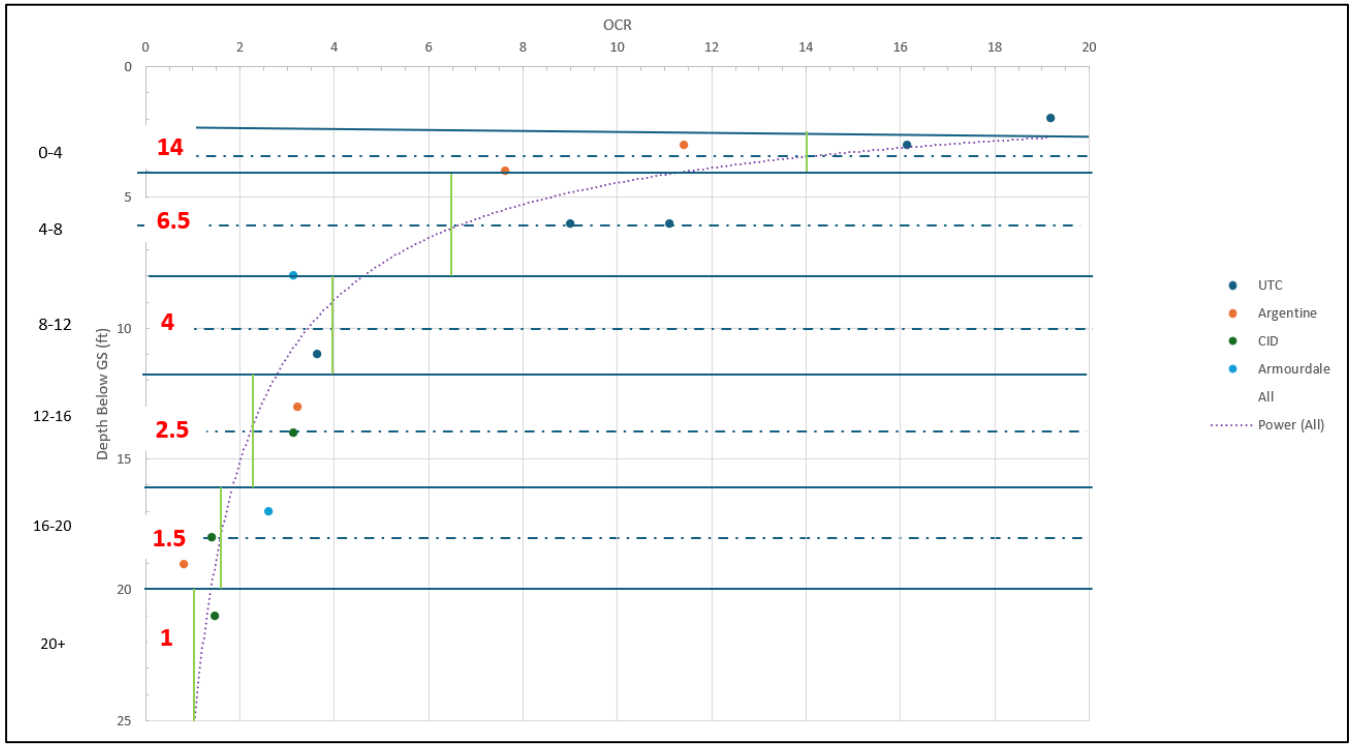
Consolidation Parameters

Boring ID	Top Depth	Bottom Depth	avg depth	Overburden tsf	Past Pressure P_c (tsf)	OCR	Overconsol Margin (psf)
ADU-208	1	3	2	0.12	2.3	19	4360
ADU-204	2	4	3	0.18	2.9	16	5440
ADU-206	5	7	6	0.37	4.1	11	7460
ADU-197	5	7	6	0.39	3.5	9	6220
AD-198	10	12	11	0.88	3.2	4	4640
Average							5624
Std Dev							1125
Avg - std dev							4499
Use							4500

Overconsolidation Ratio (OCR) = Preconsolidation Pressure/Overburden Pressure

Overconsolidation Margin = Preconsolidation Pressure – Overburden Pressure

An overconsolidation margin of 4500 psf was used in the Coduto spreadsheet.



The design overconsolidation ratio was varied by depth incorporating the consolidation testing results from UTC and similar soils at KC Levees with a power trendline

Design OCR vs Depth	
Depth	**OCR**
0-4	14
4-8	6.5
8-12	4
12-16	2.5
16-20	1.5
20+	1

Settle3 Soil Properties

Soil Property 1

Name: Soil Property 1 Color: Hatch:
Unit Weight (kips/ft3) 0.115 Sat. Unit Wt. (kips/ft3) 0.115
Poisson Ratio: 0.2 K0: 1
Immediate Settlement | Primary Consolidation | Secondary Consolidation | Datum Dependency | Stage Factors
Primary Consolidation
Material Type: Non-Linear
Cc: 0.2 Pc (ksf): 2.1
Cr: 0.03 OCR: 14
e0: 0.57 OCM (ksf): 0

Soil Property 2

Name: Soil Property 2 Color: Hatch:
Unit Weight (kips/ft3) 0.115 Sat. Unit Wt. (kips/ft3) 0.115
Poisson Ratio: 0.2 K0: 1
Immediate Settlement | Primary Consolidation | Secondary Consolidation | Datum Dependency | Stage Factors
Primary Consolidation
Material Type: Non-Linear
Cc: 0.2 Pc (ksf): 2.1
Cr: 0.03 OCR: 6.5
e0: 0.57 OCM (ksf): 0

Soil Property 3

Name: Soil Property 3 Color: Hatch:
Unit Weight (kips/ft3) 0.115 Sat. Unit Wt. (kips/ft3) 0.115
Poisson Ratio: 0.2 K0: 1
Immediate Settlement | Primary Consolidation | Secondary Consolidation | Datum Dependency | Stage Factors
Primary Consolidation
Material Type: Non-Linear
Cc: 0.2 Pc (ksf): 2.1
Cr: 0.03 OCR: 4
e0: 0.57 OCM (ksf): 0

Soil Property 4

Name: Soil Property 4 Color: Hatch:
Unit Weight (kips/ft3) 0.115 Sat. Unit Wt. (kips/ft3) 0.115
Poisson Ratio: 0.2 K0: 1
Immediate Settlement | Primary Consolidation | Secondary Consolidation | Datum Dependency | Stage Factors
Primary Consolidation
Material Type: Non-Linear
Cc: 0.2 Pc (ksf): 2.1
Cr: 0.03 OCR: 2.5
e0: 0.57 OCM (ksf): 0

Soil Property 5

Name: Soil Property 5 Color: Hatch:
Unit Weight (kips/ft3) 0.115 Sat. Unit Wt. (kips/ft3) 0.115
Poisson Ratio: 0.2 K0: 1
Immediate Settlement | Primary Consolidation | Secondary Consolidation | Datum Dependency | Stage Factors
Primary Consolidation
Material Type: Non-Linear
Cc: 0.2 Pc (ksf): 2.1
Cr: 0.03 OCR: 1.5
e0: 0.57 OCM (ksf): 0

Soil Property 6

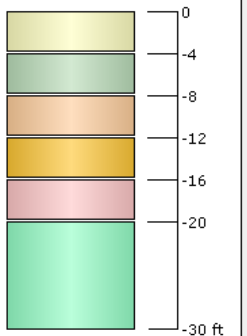
Name: Soil Property 6 Color: Hatch:
Unit Weight (kips/ft3) 0.115 Sat. Unit Wt. (kips/ft3) 0.115
Poisson Ratio: 0.2 K0: 1
Immediate Settlement | Primary Consolidation | Secondary Consolidation | Datum Dependency | Stage Factors
Primary Consolidation
Material Type: Non-Linear
Cc: 0.2 Pc (ksf): 2.1
Cr: 0.03 OCR: 1
e0: 0.57 OCM (ksf): 0

Soil Layers

Top Elevation: 0 Define Layers By Thickness

#	Name	Thickness	Top Elevation	Bottom Elevation
1	Soil Property 1	4	0	-4
2	Soil Property 2	4	-4	-8
3	Soil Property 3	4	-8	-12
4	Soil Property 4	4	-12	-16
5	Soil Property 5	4	-16	-20
6	Soil Property 6	10	-20	-30

Soil Layer Column



Insert Layer Above | Insert Layer Below | Delete Layer | OK | Cancel

Layer 6 only used for the 12’ wall in an area with 30’ of compressible soil above bedrock. In other areas, bedrock was found at 20’.

Assumed no groundwater/groundwater at creek/bedrock elevation. According to Settle3 documentation, groundwater is not needed for calculations that are not time-dependent.

- If you are NOT modelling time-dependent consolidation, then you do not necessarily have to define groundwater (e.g. if you are only interested in immediate settlement or long-term consolidation).

https://www.rocscience.com/help/settle3/documentation/project-settings/groundwater

Bedrock Depth

The majority of the borings showed compressible clay soil down to bedrock. Some borings had clayey gravel or weathered rock above bedrock. Bedrock depth varied from 6.5’ to 27.8’ below grade, generally deeper on the downstream (north) end of the project.

Boring	Bedrock Depth (ft)	Bedrock EL
AD-211	18.7	889
DC-4	12.5	895.7
ADUC-196	27.8	882.7
ADU-198	21.5	887.5
ADUC-199	21.2	888.8
ADU-200	18.8	891.8
ADU-210	11.1	903.6
AD-209	11.3	902.2
ADUC-202	10.7	902.3
AD-203	6.5	909.5
ADU-204	9.7	911.7
ADU-208	9.5	912.3
ADUC-205	13	915.8
DC-5	12.9	915.1
ADU-206	11.2	920.2
ADU-207	9.8	920.2
AVERAGE	14.1	903.0
STD DEV	5.7	12.5
PLUS STD DEV	19.7	
MINUS STD DEV		890.5

The area around ADUC-196 (UTC RB 60+70) has a floodwall planned, 9.5’ max above grade (12’ total stem length).

Settlement Checks

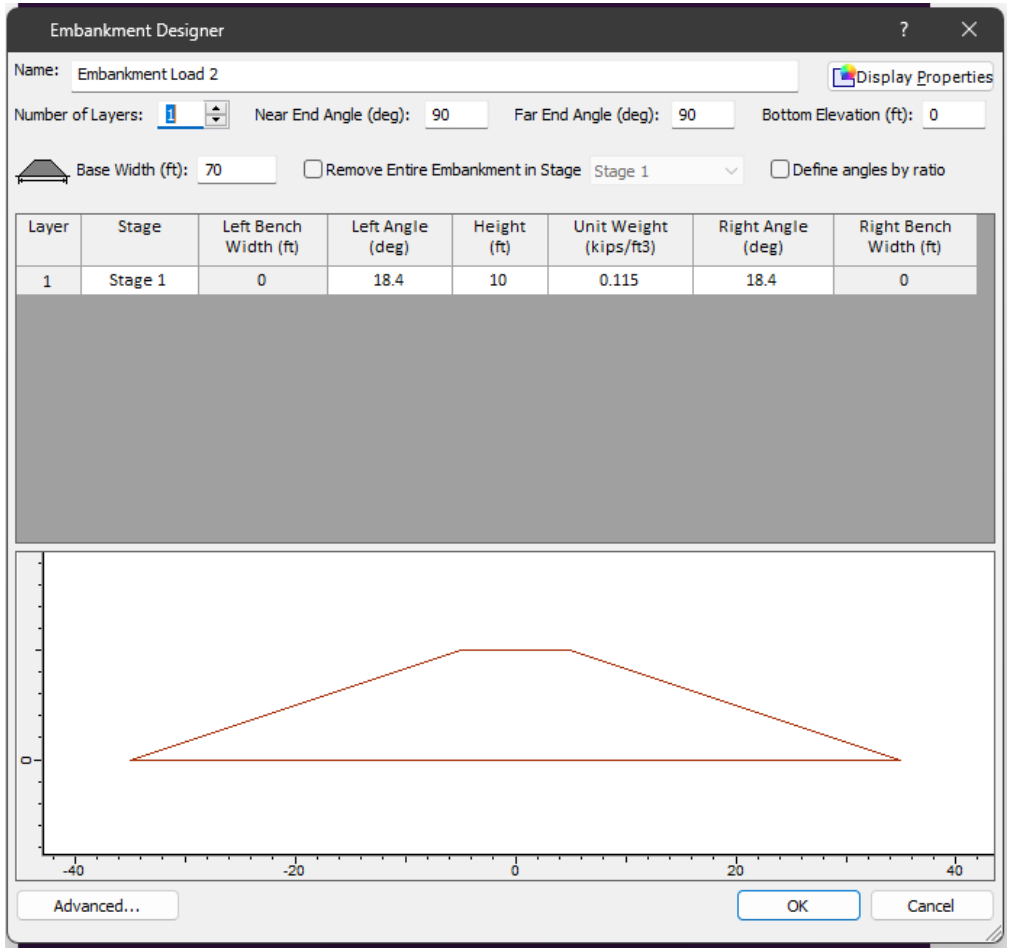
- The area around ADUC-196 (UTC RB 60+70) has a floodwall planned, 9.5’ max above grade.
- Checked settlement for 9.5’ floodwall with 30’ of compressible fill
- Checked other floodwall and levee settlements with 20’ of compressible fill

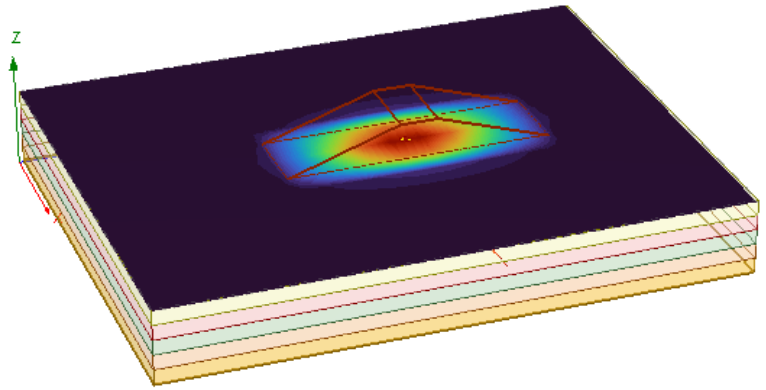
Levee Settlement

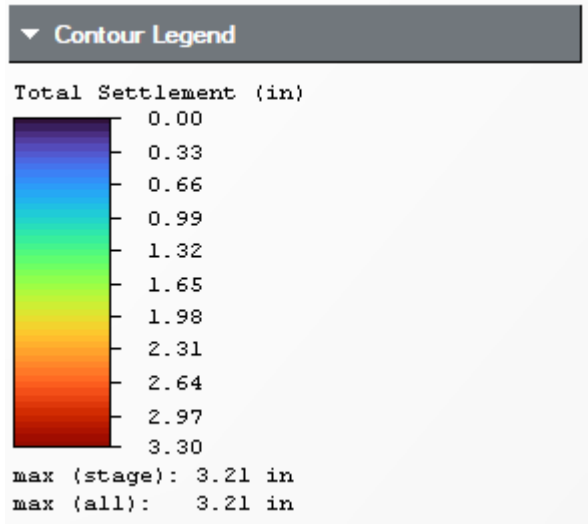
The maximum levee settlement (for 10’ tall levee with 20’ of compressible overburden soils) is expected to be around 3 inches

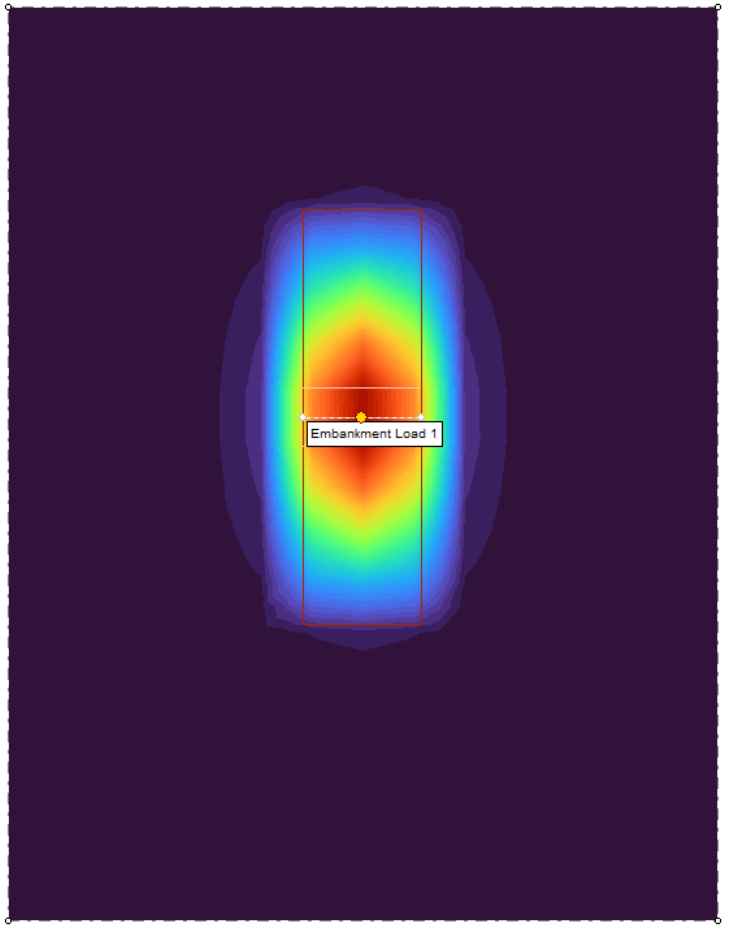
1V:3H slopes for 10’ tall levee = 30’ slope on each side
30’(2) + 10’ crest width = 70’ base width

$\tan^{-1}$ (1/3) = 18.4 degrees









Levee Settlement

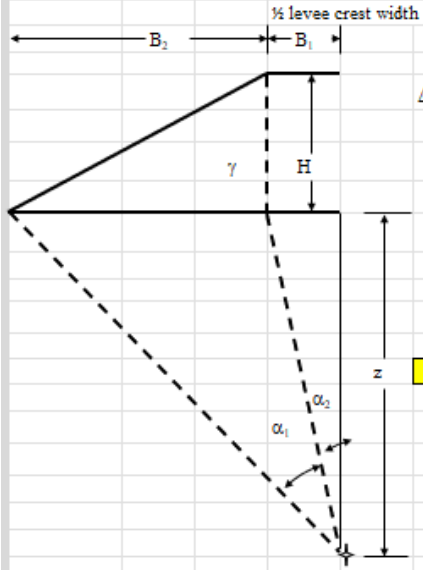
Spreadsheet results

Find stress due to new embankment

STRESS INCREASE UNDER AN EMBANKMENT - CENTERLINE

Das Foundations, 1999 (pp 233-234)

Project:	UTC 49+50 - Reach 4 RB	
Designer :	PNB/DHJ	3/26/2026
Checker :		



$\Delta p = (q_o/\pi\ [((B_1 + B_2)/B_2)(\alpha_1 + \alpha_2) - (B_1/B_2)(\alpha_2)])$ (eq. 4.20)

$\alpha 1 = \tan^{-1}((B_1 + B_2) / z) - \tan^{-1}(B_1/z)$ radians (eq. 4.21)

$\alpha 2 = \tan^{-1}(B_1/z)$ radians (eq. 4.21)

H (feet)	γ (pcf)	q_o (psf)	z (feet)	B_1 (feet)	B_2 (feet)
10.1	115	1161.5	2.5	5	30.3

α_1 = 0.3929 radians

α_2 = 1.1071 radians

Δp = 578.58 (psf)	CL due to One Side
Δp = 1157.2 psf	CL due to Both Sides

Left Side

H (feet)	γ (pcf)	q_o (psf)	z (feet)	B_1 (feet)	B_2 (feet)	α_1 (rad)	α_2 (rad)	Only from one side Δp (psf)	Include Both Δp (psf)
10.1	115	1161.5	2	5	30.3	0.3239	1.1903	579.59	**1159.17**
10.1	115	1161.5	6	5	30.3	0.7077	0.6947	561.7	**1123.36**
10.1	115	1161.5	10	5	30.3	0.8311	0.4636	529.4	**1058.79**
10.1	115	1161.5	14	5	30.3	0.8502	0.343	493.0	**986.05**
10.1	115	1161.5	18	5	30.3	0.8283	0.2709	456.9	**913.89**

Top Levee	917.7	
Ground	907.6	
Bedrock	888	in ADU-198
(Not Modified)		
	using 20' compressible zone	

Levee Settlement

2.76” spreadsheet vs 3.21” Settle3

Consolidation Settlement	
Project:	UTC Reach 4 Right Bank
Location:	Merriam, Kansas
Comments:	Settlement of clay blanket due to new levee

C_r =	0.03
C_c =	0.2
e_o =	0.57
$\gamma_{moist/sat}$ =	115 pcf

^from consol test results

Designer :	PNB/DHJ	3/26/2026
Checker :		

Compressible Sublayer Depth below Grade			Depth below LS Grade	Sublayer Thickness	Depth to Water	Effective Overburden	OCR based on depth below LS Grade	Max past pressure (overburden * OCR)	Stress Increase	Final Pressure	Rebound Curve $C_r/(1+e)$	Virgin Curve $C_c/(1+e)$	Test For Overconsol (P_c>initial)	Test For OC1 (P_{final}<P_c), rebound only	recompress for OC1	recompress portion of OC2	virgin compress for OC2	virgin compress for NC	Strain			Settlement		
Top	Bottom	Average	Average	H_o	h_w	P_o'		P_c adjusted	ΔP	P_f	Cεr	Cεc	P_c>(P_o')	P_f<P_c	ε_{rz}	ε_{rz}	ε_{cz}	ε_{cz}	Recompress Sum εrz	Virgin Compress Sum εcz	Total εz	εz*Ho	Per Layer	Total
ft	ft	ft	ft	ft	ft	psf		psf	psf	psf					Y/Y	Y/N	Y/N	N/N	ft/ft	ft/ft	ft/ft	feet	inches	inches
0	4	2	2	4	10.8	230.00	14	3220.00	1159.17	1389	0.0191	0.1274	Yes	Yes	0.0149241	NA	NA	NA	0.0149	0.0000	0.0149	0.060	0.716	0.72
4	8	6	6	4	10.8	690.00	6.5	4485.00	1123.36	1813	0.0191	0.1274	Yes	Yes	0.0080185	NA	NA	NA	0.0080	0.0000	0.0080	0.032	0.385	1.10
8	12	10	10	4	10.8	526.00	4	2104.00	1058.79	1585	0.0191	0.1274	Yes	Yes	0.0091526	NA	NA	NA	0.0092	0.0000	0.0092	0.037	0.439	1.54
12	16	14	14	4	10.8	736.40	2.5	1841.00	986.05	1722	0.0191	0.1274	Yes	Yes	0.0070516	NA	NA	NA	0.0071	0.0000	0.0071	0.028	0.338	1.88
16	20	18	18	4	10.8	946.80	1.5	1420.20	913.89	1861	0.0191	0.1274	Yes	No	NA	0.0033648	0.0149	NA	0.0034	0.0149	0.0183	0.073	0.879	2.76
																					Settlement =		2.76	inches

NC: Normally Consolidated

$$S_c = C_c H_c/(1+e_o) \log ((p_o + \Delta p)/p_o)$$

uses virgin compression curve

OC1: Overconsolidated but not loaded beyond Preconsolidation Pressure

$$S_c = C_s H_c/(1+e_o) \log ((p_o + \Delta p)/p_o)$$

uses recompression curve

OC2: Overconsolidated and loaded beyond Preconsolidation Pressure

$$S_c = C_s H_c/(1+e_o) \log p_c/p_o + C_c H_c/(1+e_o) \log ((p_o + \Delta p)/p_o)$$

uses both curves

Definitions

P_c	Max past pressure based on overburden*OCR
P_o'	Effective Overburden Pressure, γ x z
ΔP	Stress Increase from Fill or Footing Pressure
P_f	Final Pressure at given depth, P_o' + Δp
εrz	Recompressive Strain
εcz	Compressive Strain
C_r	Recompression Index
C_c	Virgin Compression Index
H_o	Sublayer Thickness
e_o	Initial Void Ratio

Preconsolidation Margin

$P_m' = P_c - P_o'$

P_o' for sample at depth of sample

for Normally Consolidated, $P_m' = 0$

Adjusted Preconsolidation Pressure

P_c adjusted = $P_m' + P_o'$ at depth being analyzed

Floodwall Settlement

The maximum floodwall settlement (for 12’ tall wall [10.5’ above grade] with 30’ of compressible overburden soils) is expected to be 0.93 inches

Max total allowable settlement is 1” based on waterstops and structural considerations

For the 8’ tall wall [6.5’ above grade]: 1.3 yd^3 of concrete
based on EDD-S spreadsheet on 1/8/26
Concrete volume = 1.3*27 = 35.1 ft^3
Concrete weight = 35.1ft^3*150 pcf = 5265 lb/ft wall length
Check w 20’ compressible soil

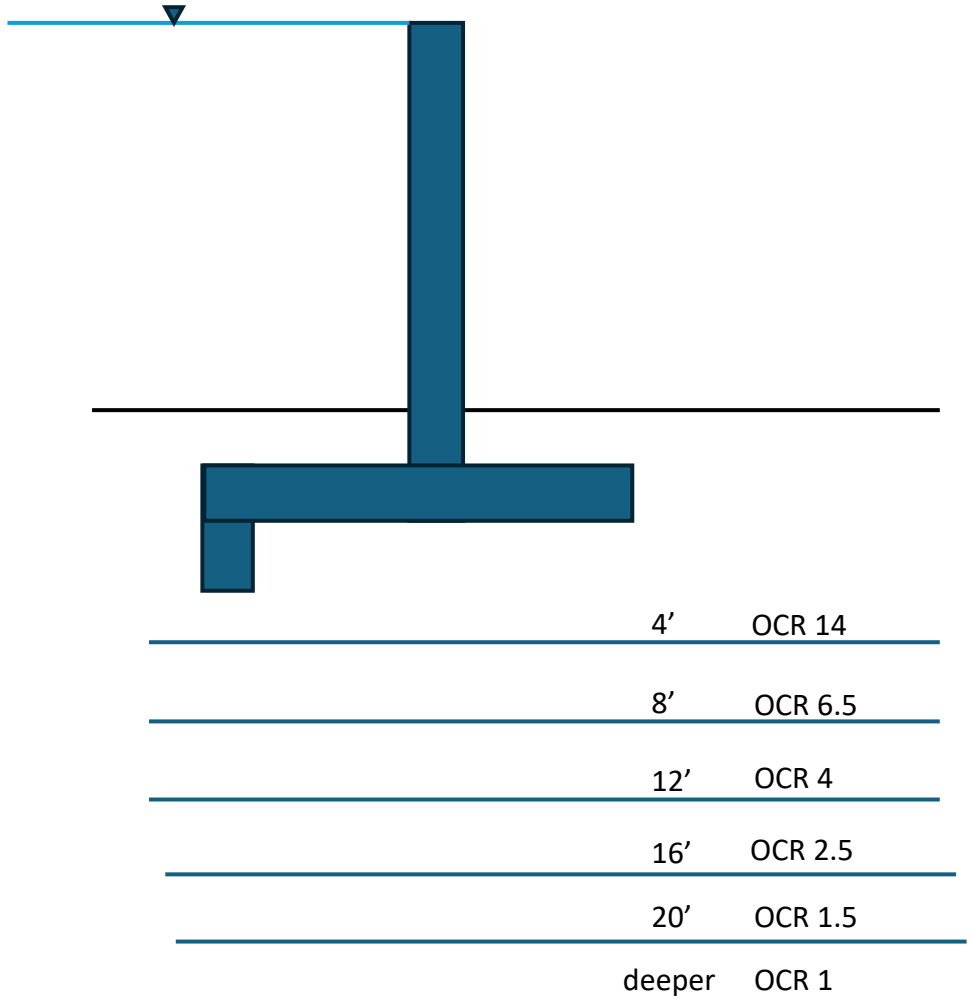
For the 12’ tall wall [10.5’ above grade]: 2.22 yd^3 of concrete
based on EDD-S spreadsheet on 1/8/26
Concrete volume = 2.22*27 = 59.94 ft^3
Concrete weight = 59.94ft^3*150 pcf = 8991 lb/ft wall length
Check with 30’ compressible soil

For the 14’ tall wall [12.5’ above grade]: 2.51 yd^3 of concrete
based on EDD-S spreadsheet on 1/8/26
Concrete volume = 2.51*27 = 67.77 ft^3
Concrete weight = 67.77ft^3*150 pcf = 10,165.5 lb/ft wall length
Check with 20’ compressible soil

WALL PROFILE		
HTS	BWIDTH	
Height of wall	Total foundation width from front of key to back of toe	Volume of Conc
4	7.00	0.61
6	9.00	0.83
6	9.00	0.83
4	7.00	0.61
6	9.00	0.83
8	10.00	1.3
8	10.00	1.3
10	14.00	1.73
8	10.00	1.3
6	9.00	0.83
6	9.00	0.83
8	10.00	1.3
10	14.00	1.73
8	10.00	1.3
6	9.00	0.83
8	10.00	1.3
14	18.00	2.36
12	14.00	2.22
10	14.00	1.73
10	14.00	1.73
12	14.00	2.22
6	9.00	0.83
8	10.00	1.3
10	14.00	1.73
8	10.00	1.3
10	14.00	1.73
12	14.00	2.22
10	14.00	1.73
14	20.00	2.51
8	10.00	1.3
3+00 LB	#REF!	#REF!
14	20.00	2.51
6	9.00	0.83
8	10.00	1.3
6	9.00	0.83
8	10.00	1.3
10	14.00	1.73
8	10.00	1.3
8	10.00	1.3
6	9.00	0.83
6	9.00	0.83
4	7.00	0.61

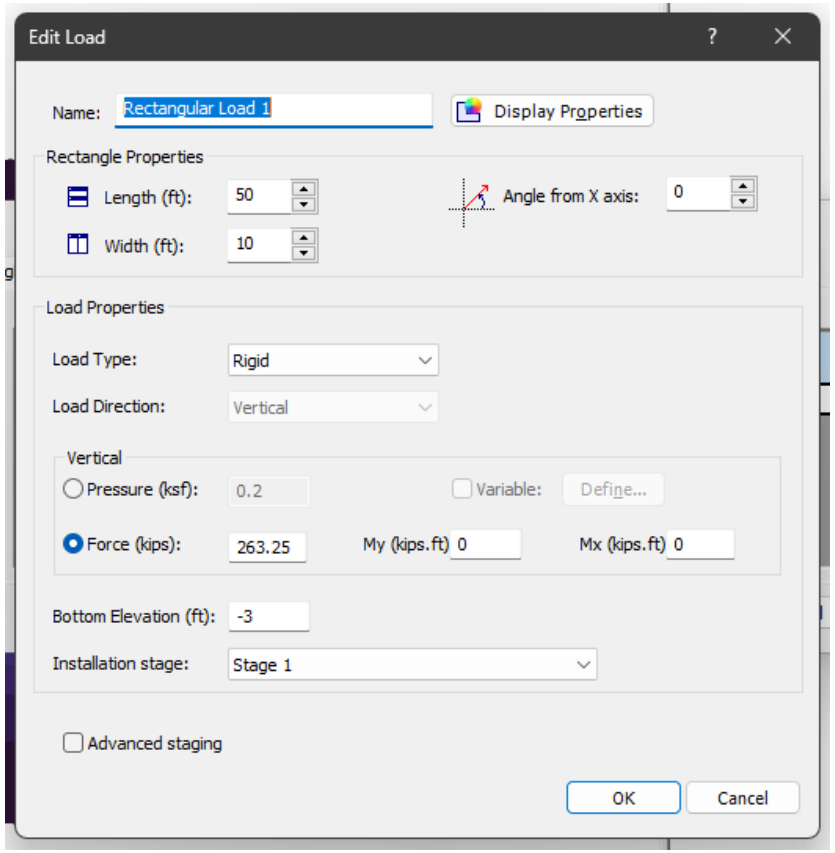
Foundation widths and concrete volumes from EDD-S

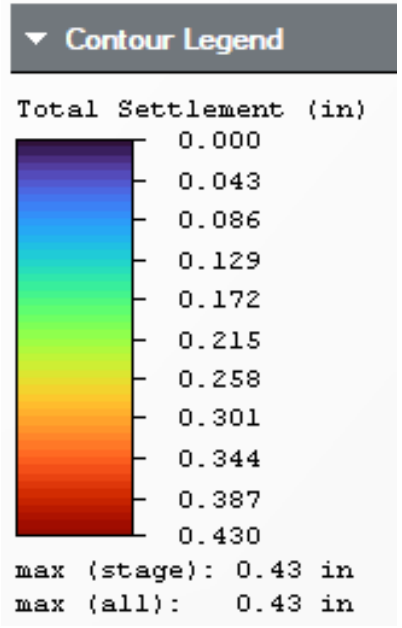
K:\MissionProjects\civ\turkeycr\505229 Upper Turkey Creek CG\6.0 Engineering-Design\6.2 Discipline\6.2.4 Structures\6.2.4.2 T-Wall\CT-Wall Files\DRAFT LB RB station ranges and elevations for structures_20250708.xlsx

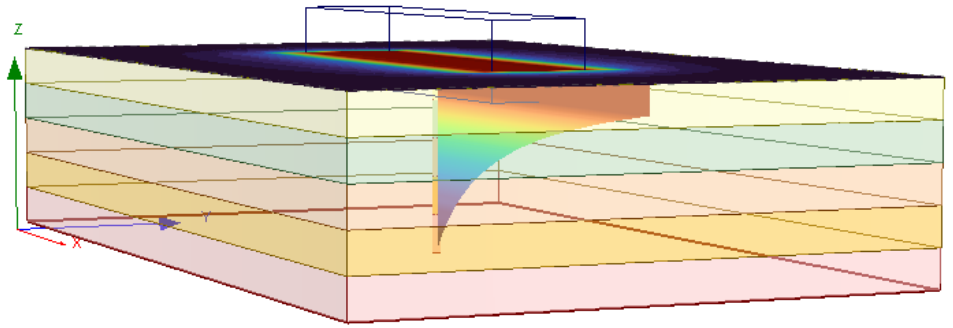


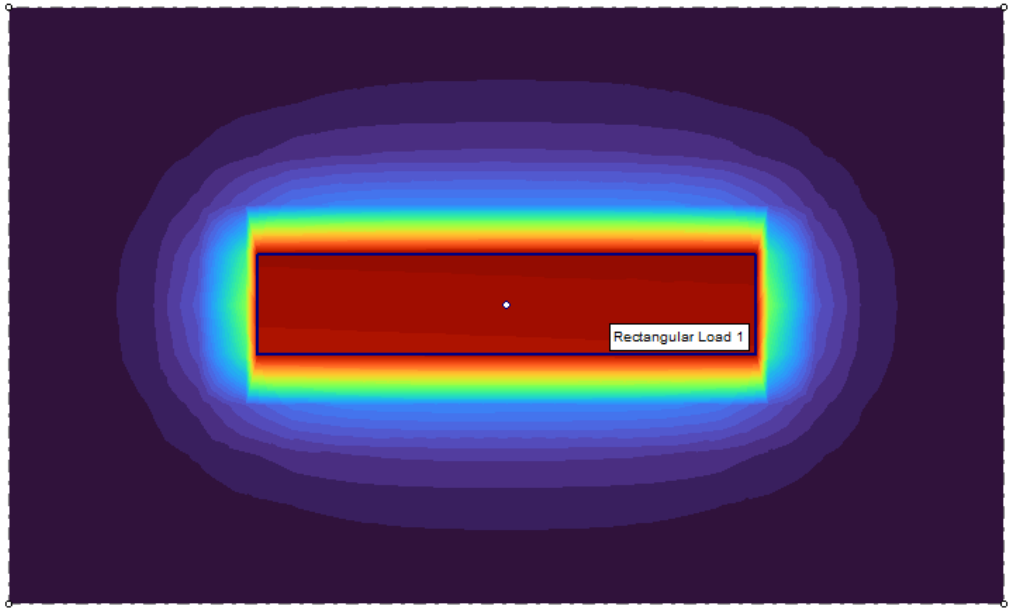
Floodwall Settlement 8’ wall

20’ compressible soil layer









5265 lb/ft of wall length*50’ representative length = 263,250 lb = 263.25 kips

Floodwall Settlement

Coduto Spreadsheet Results

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date: March 26, 2026
Identification: 8' tall wall

Input

Units	E	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	10	ft
L =	1	ft
D =	3	ft
P =	5.3	k/ft
Dw =	20	ft
r =	0.85	

Results

q =	980	lb/ft^2
delta =	0.52	in

Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)
0.0	3.0				115		
3.0	3.5	0.1274	0.0191	4500	115	0.25	4874

0.52” Coduto & 0.43” Settle3

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date: March 26, 2026
Identification: 8' tall wall

Input

Units	E	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	10	ft
L =	1	ft
D =	3	ft
P =	5.3	k/ft
Dw =	20	ft
r =	0.85	

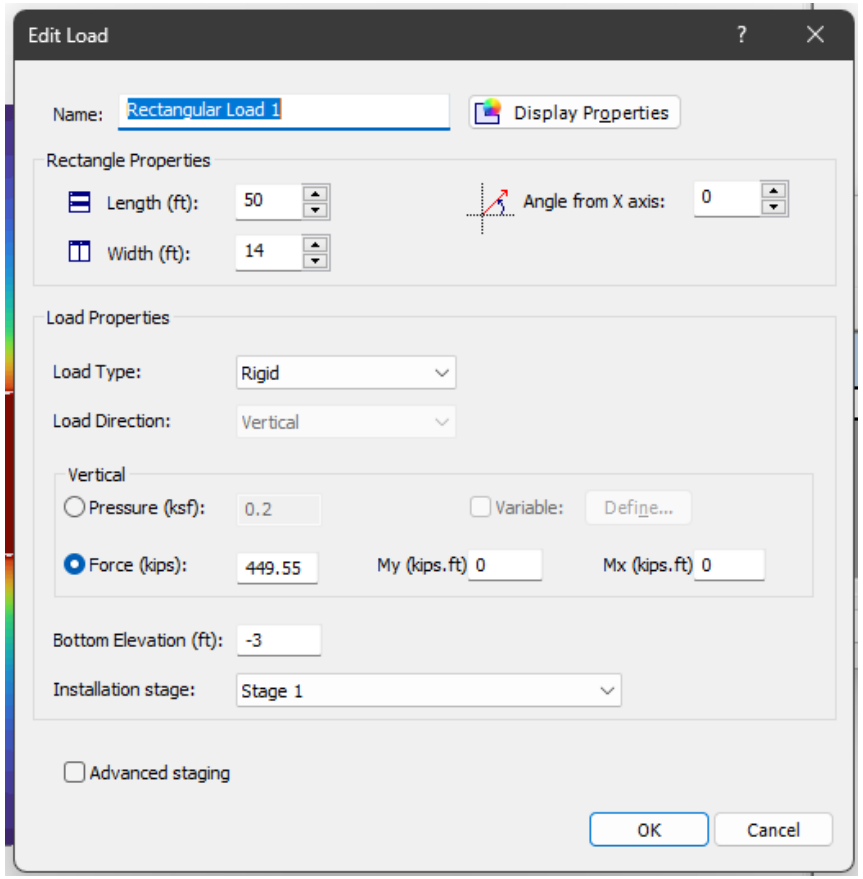
Results

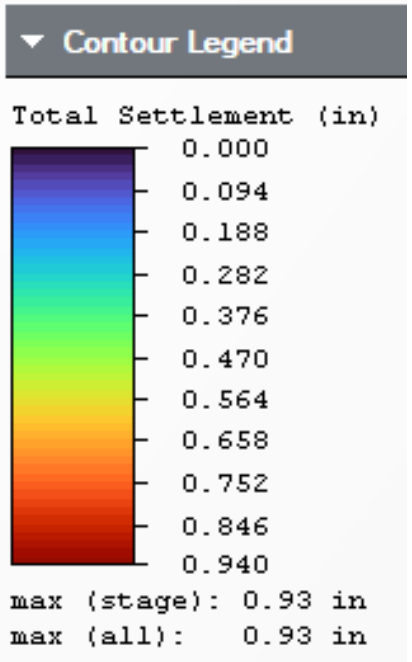
q =	980	lb/ft^2
delta =	0.51	in

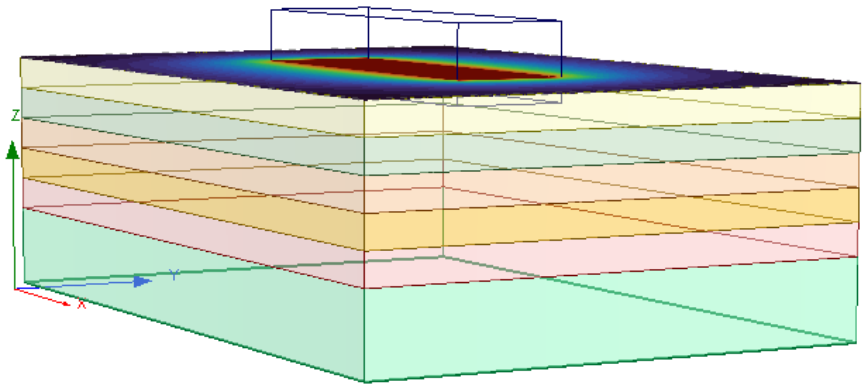
Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)	sigma zo' (lb/ft^2)	delta sigma (lb/ft^2)	sigma zf' (lb/ft^2)	strain (%)	delta (in)
0.0	3.0				115							
3.0	3.5	0.1274	0.0191	4500	115	0.25	4874	374	635	1009	0.70	0.042
3.5	4.0	0.1274	0.0191	4500	115	0.75	4931	431	634	1065	0.64	0.038
4.0	4.5	0.1274	0.0191	4500	115	1.25	4989	489	631	1120	0.58	0.035
4.5	5.0	0.1274	0.0191	4500	115	1.75	5046	546	625	1171	0.54	0.032
5.0	5.5	0.1274	0.0191	4500	115	2.25	5104	604	615	1219	0.50	0.030
5.5	6.0	0.1274	0.0191	4500	115	2.75	5161	661	602	1264	0.46	0.027
6.0	6.5	0.1274	0.0191	4500	115	3.25	5219	719	587	1305	0.42	0.025
6.5	7.0	0.1274	0.0191	4500	115	3.75	5276	776	569	1345	0.39	0.023
7.0	7.5	0.1274	0.0191	4500	115	4.25	5334	834	550	1383	0.36	0.021
7.5	8.0	0.1274	0.0191	4500	115	4.75	5391	891	530	1421	0.33	0.020
8.0	8.5	0.1274	0.0191	4500	115	5.25	5449	949	510	1458	0.30	0.018
8.5	9.0	0.1274	0.0191	4500	115	5.75	5506	1006	490	1496	0.28	0.017
9.0	9.5	0.1274	0.0191	4500	115	6.25	5564	1064	470	1534	0.26	0.015
9.5	10.0	0.1274	0.0191	4500	115	6.75	5621	1121	451	1572	0.24	0.014
10.0	10.5	0.1274	0.0191	4500	115	7.25	5679	1179	433	1612	0.22	0.013
10.5	11.0	0.1274	0.0191	4500	115	7.75	5736	1236	416	1652	0.20	0.012
11.0	11.5	0.1274	0.0191	4500	115	8.25	5794	1294	399	1693	0.19	0.011
11.5	12.0	0.1274	0.0191	4500	115	8.75	5851	1351	384	1735	0.18	0.011
12.0	12.5	0.1274	0.0191	4500	115	9.25	5909	1409	369	1778	0.16	0.010
12.5	13.0	0.1274	0.0191	4500	115	9.75	5966	1466	356	1822	0.15	0.009
13.0	13.5	0.1274	0.0191	4500	115	10.25	6024	1524	343	1867	0.14	0.009
13.5	14.0	0.1274	0.0191	4500	115	10.75	6081	1581	331	1912	0.13	0.008
14.0	14.5	0.1274	0.0191	4500	115	11.25	6139	1639	319	1958	0.13	0.008
14.5	15.0	0.1274	0.0191	4500	115	11.75	6196	1696	308	2005	0.12	0.007
15.0	15.5	0.1274	0.0191	4500	115	12.25	6254	1754	298	2052	0.11	0.007
15.5	16.0	0.1274	0.0191	4500	115	12.75	6311	1811	288	2100	0.10	0.006
16.0	16.5	0.1274	0.0191	4500	115	13.25	6369	1869	279	2148	0.10	0.006
16.5	17.0	0.1274	0.0191	4500	115	13.75	6426	1926	271	2197	0.09	0.006
17.0	17.5	0.1274	0.0191	4500	115	14.25	6484	1984	263	2246	0.09	0.005
17.5	18.0	0.1274	0.0191	4500	115	14.75	6541	2041	255	2296	0.08	0.005
18.0	18.5	0.1274	0.0191	4500	115	15.25	6599	2099	248	2347	0.08	0.005
18.5	19.0	0.1274	0.0191	4500	115	15.75	6656	2156	241	2397	0.07	0.004
19.0	19.5	0.1274	0.0191	4500	115	16.25	6714	2214	234	2448	0.07	0.004
19.5	20.0	0.1274	0.0191	4500	115	16.75	6771	2271	228	2499	0.07	0.004
20.0	20.5				115	17.25	2313	2313	222	2535	0.00	0.000

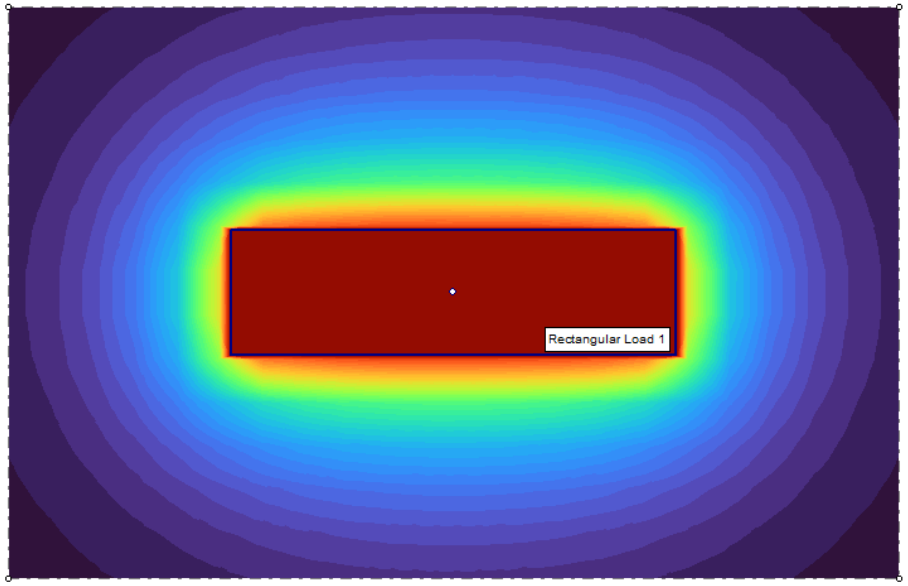
Floodwall Settlement 12’ wall

30’ compressible soil layer









8991 lb/ft of wall length*50’
representative length = 449,550 lb = 449.55 kips

Floodwall Settlement

Coduto Spreadsheet Results

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date: March 26, 2026
Identification: 12' tall wall w 30' compressible zone

Input

Units	E	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	14	ft
L =	1	ft
D =	3	ft
P =	8.9	k/ft
Dw =	30	ft
r =	0.85	

Results

q =	1086	lb/ft^2
delta =	0.71	in

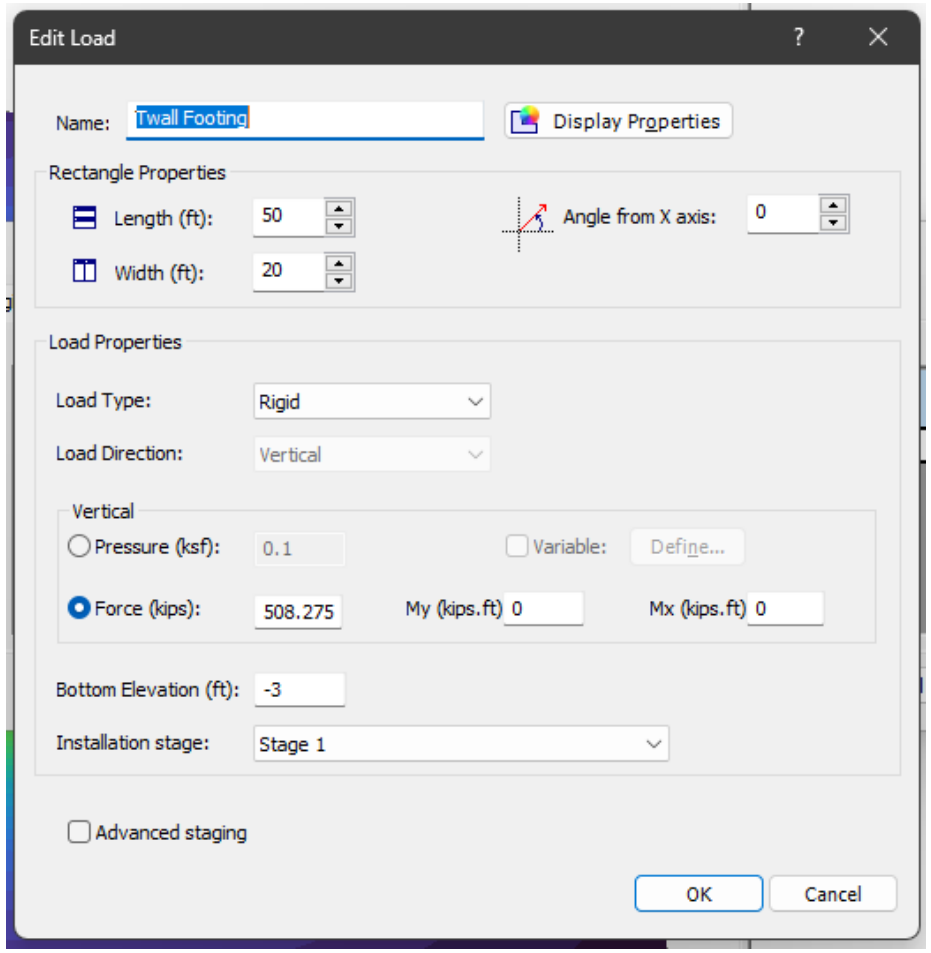
Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)	sigma zo' (lb/ft^2)	delta sigma (lb/ft^2)	sigma zf' (lb/ft^2)	strain (%)	delta (in)
0.0	3.0				115							
3.0	3.5	0.1274	0.0191	4500	115	0.25	4874	374	741	1114	0.77	0.046
3.5	4.0	0.1274	0.0191	4500	115	0.75	4931	431	740	1172	0.70	0.042

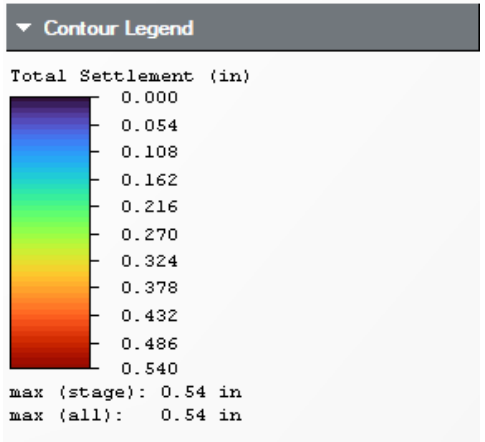
Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)	sigma zo' (lb/ft^2)	delta sigma (lb/ft^2)	sigma zf' (lb/ft^2)	strain (%)	delta (in)
0.0	3.0				115							
3.0	3.5	0.1274	0.0191	4500	115	0.25	4874	374	741	1114	0.77	0.046
3.5	4.0	0.1274	0.0191	4500	115	0.75	4931	431	740	1172	0.70	0.042
4.0	4.5	0.1274	0.0191	4500	115	1.25	4989	489	739	1228	0.65	0.039
4.5	5.0	0.1274	0.0191	4500	115	1.75	5046	546	736	1282	0.60	0.036
5.0	5.5	0.1274	0.0191	4500	115	2.25	5104	604	731	1335	0.56	0.034
5.5	6.0	0.1274	0.0191	4500	115	2.75	5161	661	725	1386	0.52	0.031
6.0	6.5	0.1274	0.0191	4500	115	3.25	5219	719	716	1435	0.49	0.029
6.5	7.0	0.1274	0.0191	4500	115	3.75	5276	776	705	1481	0.46	0.027
7.0	7.5	0.1274	0.0191	4500	115	4.25	5334	834	693	1526	0.43	0.026
7.5	8.0	0.1274	0.0191	4500	115	4.75	5391	891	679	1570	0.40	0.024
8.0	8.5	0.1274	0.0191	4500	115	5.25	5449	949	664	1612	0.37	0.022
8.5	9.0	0.1274	0.0191	4500	115	5.75	5506	1006	648	1654	0.35	0.021
9.0	9.5	0.1274	0.0191	4500	115	6.25	5564	1064	631	1695	0.33	0.020
9.5	10.0	0.1274	0.0191	4500	115	6.75	5621	1121	615	1736	0.31	0.018
10.0	10.5	0.1274	0.0191	4500	115	7.25	5679	1179	598	1776	0.29	0.017
10.5	11.0	0.1274	0.0191	4500	115	7.75	5736	1236	581	1817	0.27	0.016
11.0	11.5	0.1274	0.0191	4500	115	8.25	5794	1294	564	1858	0.26	0.015
11.5	12.0	0.1274	0.0191	4500	115	8.75	5851	1351	548	1899	0.24	0.014
12.0	12.5	0.1274	0.0191	4500	115	9.25	5909	1409	532	1941	0.23	0.014
12.5	13.0	0.1274	0.0191	4500	115	9.75	5966	1466	517	1983	0.21	0.013
13.0	13.5	0.1274	0.0191	4500	115	10.25	6024	1524	502	2026	0.20	0.012
13.5	14.0	0.1274	0.0191	4500	115	10.75	6081	1581	488	2069	0.19	0.011
14.0	14.5	0.1274	0.0191	4500	115	11.25	6139	1639	474	2113	0.18	0.011
14.5	15.0	0.1274	0.0191	4500	115	11.75	6196	1696	461	2157	0.17	0.010
15.0	15.5	0.1274	0.0191	4500	115	12.25	6254	1754	448	2202	0.16	0.010
15.5	16.0	0.1274	0.0191	4500	115	12.75	6311	1811	436	2247	0.15	0.009
16.0	16.5	0.1274	0.0191	4500	115	13.25	6369	1869	424	2293	0.14	0.009
16.5	17.0	0.1274	0.0191	4500	115	13.75	6426	1926	413	2339	0.14	0.008
17.0	17.5	0.1274	0.0191	4500	115	14.25	6484	1984	402	2386	0.13	0.008
17.5	18.0	0.1274	0.0191	4500	115	14.75	6541	2041	392	2433	0.12	0.007
18.0	18.5	0.1274	0.0191	4500	115	15.25	6599	2099	382	2480	0.12	0.007
18.5	19.0	0.1274	0.0191	4500	115	15.75	6656	2156	372	2528	0.11	0.007
19.0	19.5	0.1274	0.0191	4500	115	16.25	6714	2214	363	2577	0.11	0.006
19.5	20.0	0.1274	0.0191	4500	115	16.75	6771	2271	354	2626	0.10	0.006
20.0	20.5	0.1274	0.0191	4500	115	17.25	6829	2329	346	2675	0.10	0.006
20.5	21.0	0.1274	0.0191	4500	115	17.75	6886	2386	338	2724	0.09	0.006
21.0	21.5	0.1274	0.0191	4500	115	18.25	6944	2444	330	2774	0.09	0.005
21.5	22.0	0.1274	0.0191	4500	115	18.75	7001	2501	323	2824	0.09	0.005
22.0	22.5	0.1274	0.0191	4500	115	19.25	7059	2559	316	2875	0.08	0.005
22.5	23.0	0.1274	0.0191	4500	115	19.75	7116	2616	309	2925	0.08	0.005
23.0	23.5	0.1274	0.0191	4500	115	20.25	7174	2674	303	2976	0.08	0.005
23.5	24.0	0.1274	0.0191	4500	115	20.75	7231	2731	296	3027	0.07	0.004
24.0	24.5	0.1274	0.0191	4500	115	21.25	7289	2789	290	3079	0.07	0.004
24.5	25.0	0.1274	0.0191	4500	115	21.75	7346	2846	284	3131	0.07	0.004
25.0	25.5	0.1274	0.0191	4500	115	22.25	7404	2904	279	3182	0.06	0.004
25.5	26.0	0.1274	0.0191	4500	115	22.75	7461	2961	273	3235	0.06	0.004
26.0	26.5	0.1274	0.0191	4500	115	23.25	7519	3019	268	3287	0.06	0.004
26.5	27.0	0.1274	0.0191	4500	115	23.75	7576	3076	263	3339	0.06	0.003
27.0	27.5	0.1274	0.0191	4500	115	24.25	7634	3134	258	3392	0.06	0.003
27.5	28.0	0.1274	0.0191	4500	115	24.75	7691	3191	253	3445	0.05	0.003
28.0	28.5	0.1274	0.0191	4500	115	25.25	7749	3249	249	3498	0.05	0.003
28.5	29.0	0.1274	0.0191	4500	115	25.75	7806	3306	245	3551	0.05	0.003
29.0	29.5	0.1274	0.0191	4500	115	26.25	7864	3364	240	3604	0.05	0.003
29.5	30.0	0.1274	0.0191	4500	115	26.75	7921	3421	236	3657	0.05	0.003
30.0	30.5				120	27.25	3464	3464	232	3697	0.00	0.000

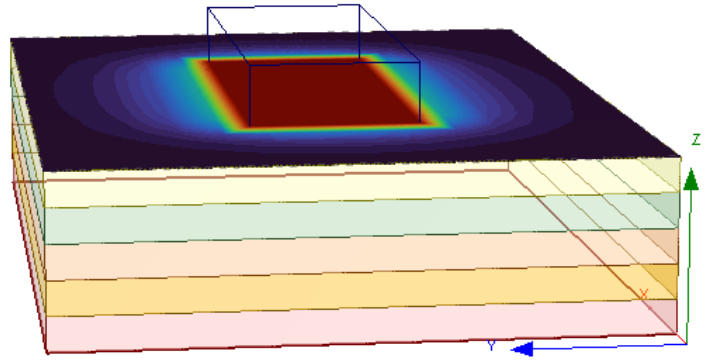
0.71” Coduto & 0.93” on Settle3 (OCR on Settle3 varies w depth leading to some areas w different settlement equations, no normally consolidated in this case for Coduto)

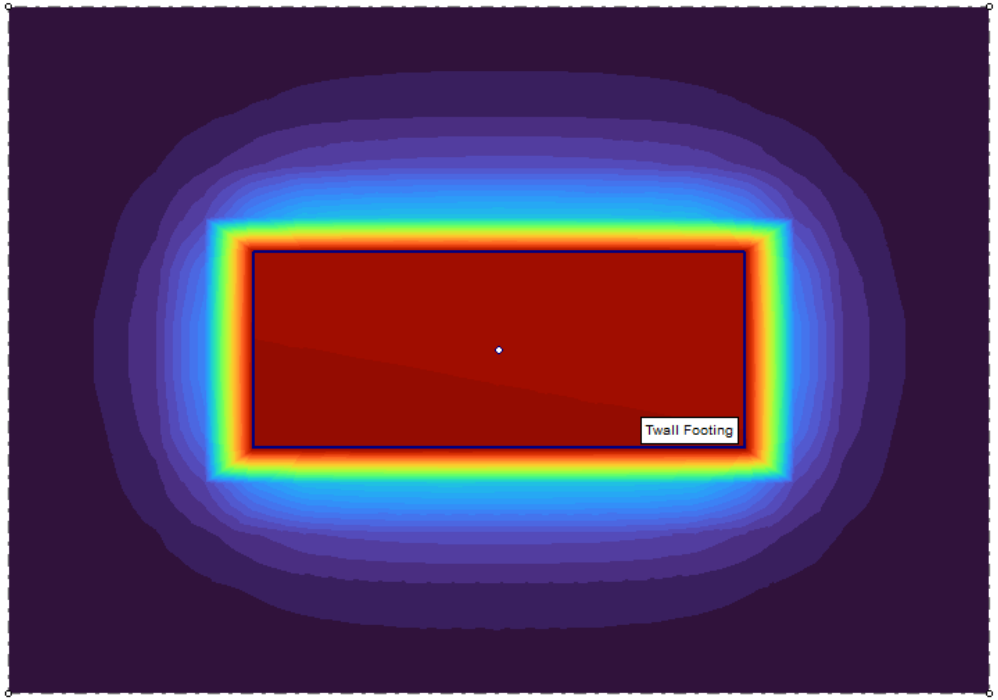
Floodwall Settlement 14’ wall

20’ compressible soil layer









10165.5 lb/ft of wall length*50’ representative length = 508,275 lb = 508.275 kips

Floodwall Settlement

Coduto Spreadsheet Results

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date: March 26, 2026
Identification: 14' tall wall

Input

Units	E	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	20	ft
L =	1	ft
D =	3	ft
P =	10.2	k/ft
Dw =	20	ft
r =	0.85	

Results

q =	960	lb/ft^2
delta =	0.58	in

Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)
0.0	3.0				115		
3.0	3.5	0.1274	0.0191	4500	115	0.25	4874

0.58” Coduto & 0.54” on Settle3

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date: March 26, 2026
Identification: 14' tall wall

Input

Units	E	E or SI
Shape	CO	SQ, CI, CO, or RE
B =	20	ft
L =	1	ft
D =	3	ft
P =	10.2	k/ft
Dw =	20	ft
r =	0.85	

Results

q =	960	lb/ft^2
delta =	0.58	in

Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)	sigma zo' (lb/ft^2)	delta sigma (lb/ft^2)	sigma zf' (lb/ft^2)	strain (%)	delta (in)
0.0	3.0				115							
3.0	3.5	0.1274	0.0191	4500	115	0.25	4874	374	615	989	0.69	0.041
3.5	4.0	0.1274	0.0191	4500	115	0.75	4931	431	615	1046	0.62	0.037
4.0	4.5	0.1274	0.0191	4500	115	1.25	4989	489	614	1103	0.57	0.034
4.5	5.0	0.1274	0.0191	4500	115	1.75	5046	546	614	1160	0.53	0.032
5.0	5.5	0.1274	0.0191	4500	115	2.25	5104	604	612	1216	0.49	0.030
5.5	6.0	0.1274	0.0191	4500	115	2.75	5161	661	610	1271	0.46	0.028
6.0	6.5	0.1274	0.0191	4500	115	3.25	5219	719	607	1326	0.43	0.026
6.5	7.0	0.1274	0.0191	4500	115	3.75	5276	776	603	1380	0.41	0.024
7.0	7.5	0.1274	0.0191	4500	115	4.25	5334	834	599	1432	0.38	0.023
7.5	8.0	0.1274	0.0191	4500	115	4.75	5391	891	593	1484	0.36	0.022
8.0	8.5	0.1274	0.0191	4500	115	5.25	5449	949	587	1536	0.34	0.020
8.5	9.0	0.1274	0.0191	4500	115	5.75	5506	1006	580	1586	0.32	0.019
9.0	9.5	0.1274	0.0191	4500	115	6.25	5564	1064	572	1636	0.30	0.018
9.5	10.0	0.1274	0.0191	4500	115	6.75	5621	1121	564	1685	0.29	0.017
10.0	10.5	0.1274	0.0191	4500	115	7.25	5679	1179	555	1734	0.27	0.016
10.5	11.0	0.1274	0.0191	4500	115	7.75	5736	1236	546	1783	0.26	0.015
11.0	11.5	0.1274	0.0191	4500	115	8.25	5794	1294	537	1831	0.24	0.015
11.5	12.0	0.1274	0.0191	4500	115	8.75	5851	1351	528	1879	0.23	0.014
12.0	12.5	0.1274	0.0191	4500	115	9.25	5909	1409	518	1927	0.22	0.013
12.5	13.0	0.1274	0.0191	4500	115	9.75	5966	1466	508	1974	0.21	0.013
13.0	13.5	0.1274	0.0191	4500	115	10.25	6024	1524	498	2022	0.20	0.012
13.5	14.0	0.1274	0.0191	4500	115	10.75	6081	1581	489	2070	0.19	0.011
14.0	14.5	0.1274	0.0191	4500	115	11.25	6139	1639	479	2118	0.18	0.011
14.5	15.0	0.1274	0.0191	4500	115	11.75	6196	1696	469	2166	0.17	0.010
15.0	15.5	0.1274	0.0191	4500	115	12.25	6254	1754	460	2214	0.16	0.010
15.5	16.0	0.1274	0.0191	4500	115	12.75	6311	1811	451	2262	0.16	0.009
16.0	16.5	0.1274	0.0191	4500	115	13.25	6369	1869	441	2310	0.15	0.009
16.5	17.0	0.1274	0.0191	4500	115	13.75	6426	1926	432	2359	0.14	0.009
17.0	17.5	0.1274	0.0191	4500	115	14.25	6484	1984	424	2407	0.14	0.008
17.5	18.0	0.1274	0.0191	4500	115	14.75	6541	2041	415	2456	0.13	0.008
18.0	18.5	0.1274	0.0191	4500	115	15.25	6599	2099	407	2506	0.12	0.007
18.5	19.0	0.1274	0.0191	4500	115	15.75	6656	2156	399	2555	0.12	0.007
19.0	19.5	0.1274	0.0191	4500	115	16.25	6714	2214	391	2604	0.11	0.007
19.5	20.0	0.1274	0.0191	4500	115	16.75	6771	2271	383	2654	0.11	0.007
20.0	20.5				115	17.25	2313	2313	376	2689	0.00	0.000

APPENDIX B.6 – RIPRAP AND BEDDING DESIGN

UTC Riprap Design

D100 max riprap sizes needed for UTC channel velocities were provided by H&H/Civil for various locations in the system using FS of 1.5, with results of 42”, 33”, and 21”

The 42” riprap was needed for one reach downstream of Merriam Bridge (right bank UTC Sta 64+00 to 65+75).
As 42” riprap is not locally available, rather than import rock from a nonlocal source, H&H determined it was okay to use a reduced FS of 1.3 for this area (1.1 is the minimum required), resulting in a D100 of 36”



From Chapter 4 of DDR:

A total of six unique maximum riprap diameters were determined from the HEC-RAS tool and then condensed down to three to reduce design complexity. The largest riprap required has a D_{100} of 42 inches. This is located downstream of the Merriam Drive bridge on the right bank which is a sharp outside bend. A 36-inch diameter was selected for this area due to the lack of local availability of 42-inch riprap.

A D_{100} of 36 inches was deemed suitable as it retained a factor of safety of 1.3 which is still above the minimum factor of safety (1.1) specified in EM 1110-2-1601. Additionally, this section of the flood protection system is lower risk given the riprap is located at the downstream end of the project. Any floodwall failure at this location would result in backwater flooding which would act as a bathtub until river stages drop and water drains out of the interior through the breach. The bathub effect occurs when backwater enters the downstream portion of a protection system and slowly fills rather than conveying through the leveed area. This presents less life safety risk than breaches that occur upstream and have hydraulic head to flow through the leveed area.

UTC Riprap Design

Table 3-1
Gradations for Riprap Placement in the Dry, Low-Turbulence Zones

	Limits of Stone Weight, lb[1], for Percent Lighter by Weight							
	D_{100}(max)	100		50		15		D_{30}(min)	D_{90}(min)	
	in.	Max	Min	Max[2]	Min	Max[2]	Min	ft	ft	
	Specific Weight = 155 pcf									
*	9	34	14	10	7	5	2	0.37	0.53	*
	12	81	32	24	16	12	5	0.48	0.70	
	15	159	63	47	32	23	10	0.61	0.88	
	18	274	110	81	55	41	17	0.73	1.06	
	21	435	174	129	87	64	27	0.85	1.23	
	24	649	260	192	130	96	41	0.97	1.40	
	27	924	370	274	185	137	58	1.10	1.59	
	30	1,268	507	376	254	188	79	1.22	1.77	
	33	1,688	675	500	338	250	105	1.34	1.94	
	36	2,191	877	649	438	325	137	1.46	2.11	
	42	3,480	1,392	1,031	696	516	217	1.70	2.47	
	48	5,194	2,078	1,539	1,039	769	325	1.95	2.82	
	54	7,396	2,958	2,191	1,479	1,096	462	2.19	3.17	
	Specific Weight = 165 pcf									
*	9	36	15	11	7	5	2	0.37	0.53	*
	12	86	35	26	17	13	5	0.48	0.70	
	15	169	67	50	34	25	11	0.61	0.88	
	18	292	117	86	58	43	18	0.73	1.06	
	21	463	185	137	93	69	29	0.85	1.23	
	24	691	276	205	138	102	43	0.97	1.40	
	27	984	394	292	197	146	62	1.10	1.59	
	30	1,350	540	400	270	200	84	1.22	1.77	
	33	1,797	719	532	359	266	112	1.34	1.96	
	36	2,331	933	691	467	346	146	1.46	2.11	
	42	3,704	1,482	1,098	741	549	232	1.70	2.47	
	48	5,529	2,212	1,638	1,106	819	346	1.95	2.82	
	54	7,873	3,149	2,335	1,575	1,168	492	2.19	3.17	
	Specific Weight = 175 pcf									
*	9	39	15	11	8	6	2	0.37	0.53	*
	12	92	37	27	18	14	5	0.48	0.70	
	15	179	72	53	36	27	11	0.61	0.88	
	18	309	124	92	62	46	19	0.73	1.06	
	21	491	196	146	98	73	31	0.85	1.23	
	24	733	293	217	147	109	46	0.97	1.40	
	27	1,044	417	309	209	155	65	1.10	1.59	
	30	1,432	573	424	286	212	89	1.22	1.77	
	33	1,906	762	565	381	282	119	1.34	1.94	
	36	2,474	990	733	495	367	155	1.46	2.11	
	42	3,929	1,571	1,164	786	582	246	1.70	2.47	
	48	5,864	2,346	1,738	1,173	869	367	1.95	2.82	
	54	8,350	3,340	2,474	1,670	1,237	522	2.19	3.17	

Notes:
1. Stone weight limit data from ETL 1110-2-120 (HQUSACE, 1971 (14 May), "Additional Guidance for Riprap Channel Protection, Ch 1," US Government Printing Office, Washington, DC). Relationship between diameter and weight is based on the shape of a sphere.
2. The maximum limits at the W_{50} and W_{15} sizes can be increased as in the Lower Mississippi Valley Division Standardized Gradations shown in Appendix F.

Table 3-1 in EM 1110-2-1601 (Hydraulic Design of Flood Channels) was used to develop a range of weights for W100, W50, and W15 for the different specific weight options. EDG-G stated that the 155 and 165 pcf specific weights should be used for rock in this area.

- For 36” riprap, W100 of 877-2331 lb, W50 of 438-691 lb, and W15 of 137-346 lb
- For 33” riprap, W100 of 675-1797 lb, W50 of 338-532 lb, and W15 of 105-266 lb
- For 21” riprap, W100 of 174-463 lb, W50 of 87-137 lb, and W15 of 27-69 lb

UTC Riprap Design

Layer Thickness

EM 1110-2-1601
Change 1
30 Jun 94

Section 3-2e:

e. Layer thickness. All stones should be contained within the riprap layer thickness to provide maximum resistance against erosive forces. Oversize stones, even in isolated spots, may result in riprap failure by precluding mutual support and interlock between individual stones, causing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller size stone. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness:

(1) It should not be less than the spherical diameter of the upper limit W_{100} stone or less than 1.5 times the spherical diameter of the upper limit W_{50} stone, whichever results in the greater thickness.

(2) The thickness determined by (1) above should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement. At one location in the US Army Engineer Division, Missouri River, divers and sonic sounders were used to reduce the underwater thickness to 1.25 times the dry placement thickness.

Layer thickness should not be less than spherical diameter of the upper limit W100 stone

36” for 36” D100 riprap

Layer thickness should not be less than 1.5 times the spherical diameter of the upper limit W50 stone

Largest W50 would be 691 lb
691lb/165pcf = 4.19 ft^3
$4/3\pi r^3 = 4.19\ ft^3$, r = 1 ft, d = 2 ft
1.5 x 24” = 36”

Layer thickness for 36” riprap should be at least 36”

UTC Bedding Design

No specific guidance is available for levees. General requirements for dams were used.

The gradation of the bedding material should provide for retention of bedding particles by the overlying riprap layer and for retention of the material underlying the bedding layer. Bedding material should generally conform to Equations C-1 through C-3 from EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams (2004).

In addition, the bedding must not be so broadly graded as to be susceptible to internal instability. USBR guidance recommends bedding material be considered internally unstable when its coefficient of uniformity is less than 10 (Design Standards No. 13 Embankment Dams, Chapter 7 Riprap Slope Protection, Section 7.3.7.1, dated May 2014).

EM 1110-2-2300 states that bedding materials over erosion-resistant clay materials need not be designed to meet the criteria of Equation C-1 or C-2 but must still satisfy Equation C-3.

$$D_{15B} > 5D_{15E} \quad \text{(C-1)}$$

$$D_{15B} < 5D_{85E} \quad \text{(C-2)}$$

$$D_{85B} > D_{15R}/5 \quad \text{(C-3)}$$

where

D_{15B} = the 15 percent passing the size of the bedding

D_{85B} = the 85 percent passing the size of the bedding

D_{15E} = the 15 percent passing the size of the material to be protected

D_{85E} = the 85 percent passing the size of the material to be protected

D_{15R} = the 15 percent passing the size of the riprap

An intermediate filter layer may be required between the bedding and riprap to prevent washout of the bedding. Bedding layers over erosion-resistant clay materials need not be designed to meet the criteria of Equation C-1 or Equation C-2 but must still satisfy Equation C-3. Each design should produce a specification that defines material sources, gradations, and layer thickness to economically provide the riprap and bedding layers required to protect the embankment.

UTC Bedding Design

Riprap will be placed over impervious fill on the riverside levee slope. Levee construction specifications define impervious fill as lean clay, low plasticity silt, or borderline clay and silt in accordance with ASTM D2487.

The in-place soils at Upper Turkey Creek are predominantly lean clays, so the existing bank slope and embankment fill material obtained from required excavations will likely be lean clay. Borrow soils are expected to be lean clay soils. The levee cross section is shown as homogenous impervious fill in plans.

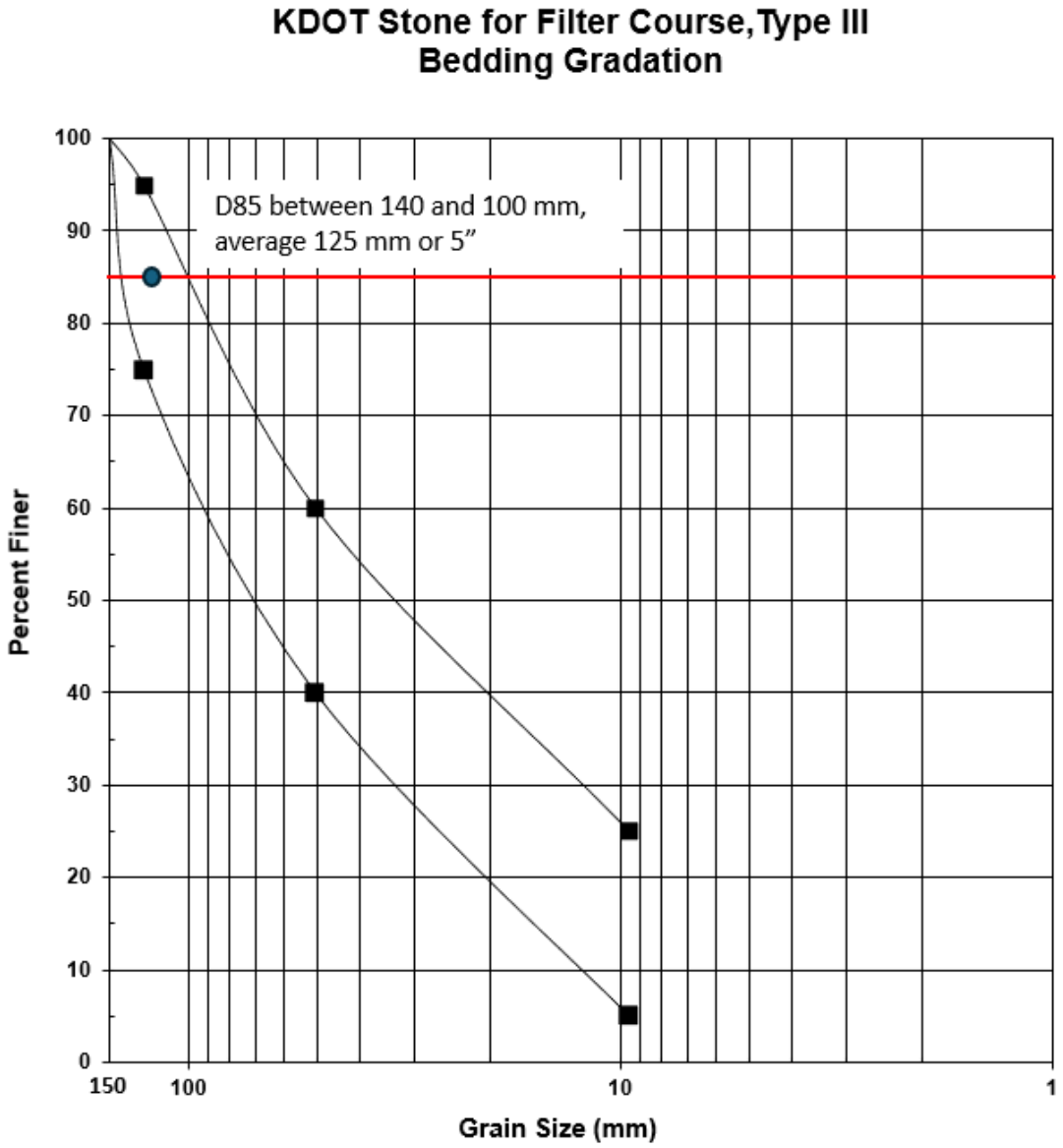
As the riverside levee slope will consist of erosion-resistant clay materials, bedding is being designed to satisfy Equation C-3 but not C-1 and C-2. Retention of the material underlying bedding is less important for levees which are loaded irregularly for short durations as compared to dams.

It is recommended that bedding material for the riprap overlay meet the gradation requirements of Kansas Department of Transportation (KDOT) Stone for Filter Course, Type III material.

TABLE 1114-3: STONE FOR FILTER COURSE									
Material	Percent Retained on Sieve Size								
	6”	5”	4”	3”	2”	1”	½”	3/8”	No. 4
Type I		0	0-5		10-40	25-60		55-85	70-95
Type II			0	0-5			50-90		
Type III	0	5-25			40-60			75-95	

Sieve/Size	Percent Finer
6” (152.4 mm)	100
5” (125 mm)	75-95
2” (50.8 mm)	40-60
3/8” (9.525 mm)	5-25

UTC Bedding Design



UTC Bedding Design

Retention of Bedding Material by Riprap per Equation C-3

$$D_{85B} > D_{15R}/5$$

Determined D_{15R} for the largest 36" riprap using the largest W_{15R} for the 165 pcf specific weight. The results are the same using the largest W_{15R} with the 155 pcf specific weight.

For the 36" riprap, the largest W_{15} for that range is 346 lb. 346 lb/165 pcf = 2.10 ft^3 volume. Using volume of sphere of $4/3\pi r^3$, r = 0.794 ft and d = 1.58 ft or 19.06".

$D_{15R}/5$ = 19.06"/5= 3.81"
D_{85B} ~ 5" supposed to be >$D_{15R}/5$ ~ 3.81"
For the largest riprap, Equation C-3 is satisfied

The average W_{15} for 36" riprap is 246 lb. 246 lb/165 pcf = 1.49 ft^3 volume. Using volume of sphere of $4/3\pi r^3$, r = 0.709 ft and d = 1.42 ft or 17.01".

$D_{15R}/5$ = 17.01"/5= 3.40"
D_{85B} ~ 5" >D_{15R} ~ 3.4"
For the average size 36" riprap, Equation C-3 is satisfied

For the smaller 33" and 21" riprap used over the majority of the project, the D_{15R} is smaller than for the 36" riprap and Equation C-3 is satisfied.

Internal Stability of Bedding

The approximate coefficient of uniformity C_u for the recommended bedding gradation D_{60}/D_{10} is 50.8 mm/9.525 mm or 5.33, meeting USBR guidance to be less than 10 for internal stability.

It is recommended that the gradation requirements for KDOT Stone for Filter Course, Type III material be used for bedding for all three riprap sizes

UTC Bedding Design

Bedding Thickness

There is no specific guidance for bedding thickness for levees. The levee design EM and flood channel EM do not specify.

Thickness guidance for riprap in flood channels specified thickness of not less than the spherical diameter of the W100 stone or 1.5 times the spherical diameter of the W50 stone (which roughly equals the spherical diameter of the W100 stone). D100 for the specified bedding gradation is 6”.

6” bedding thickness was used for KC Levees design and that is typical for levees in NWK. As levees have performed well for slope protection during flood loadings, 6” bedding thickness is recommended for Upper Turkey Creek

APPENDIX B.7 – FLOODWALL TOE DRAIN DESIGN

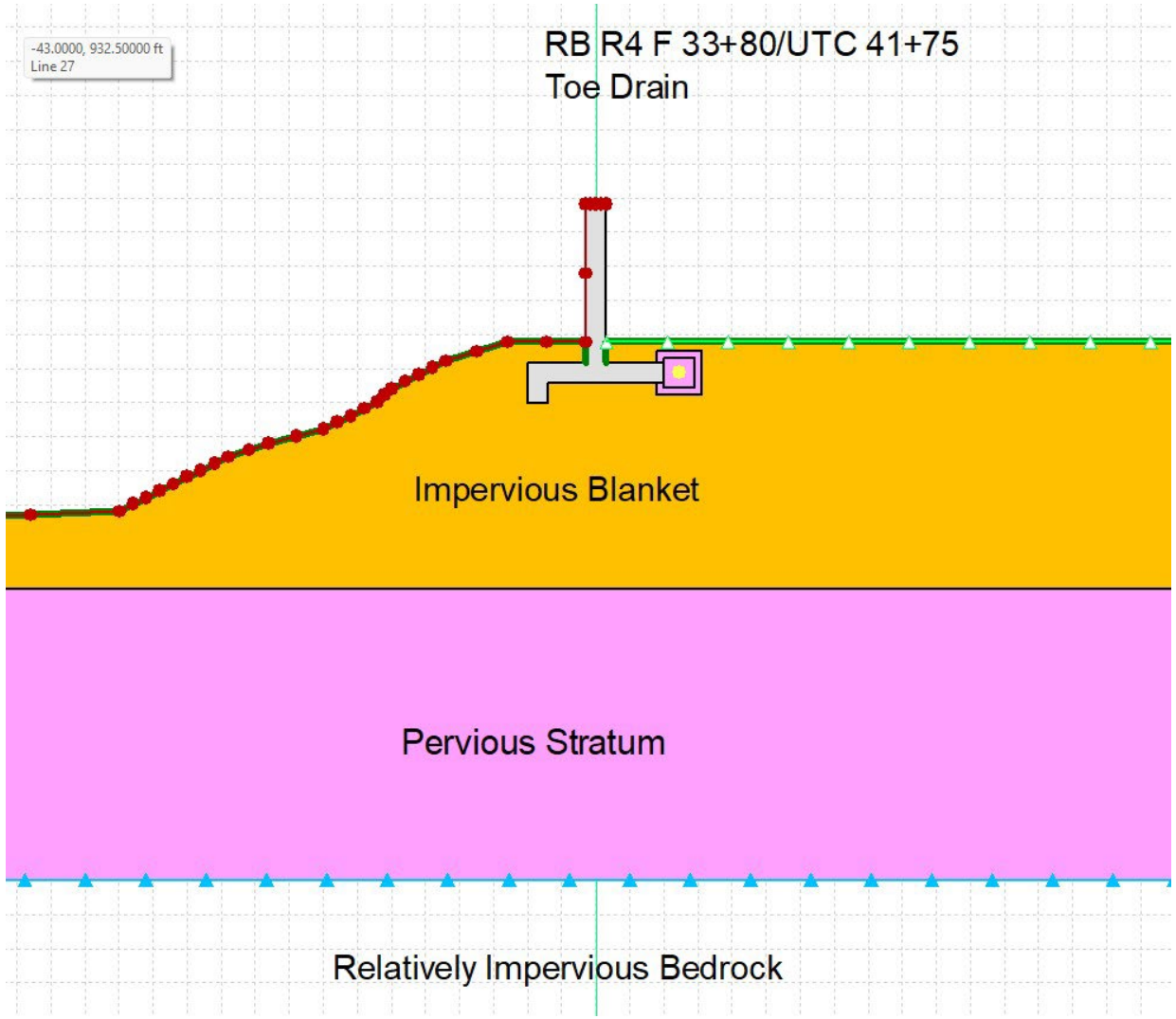
Floodwall Toe Drain

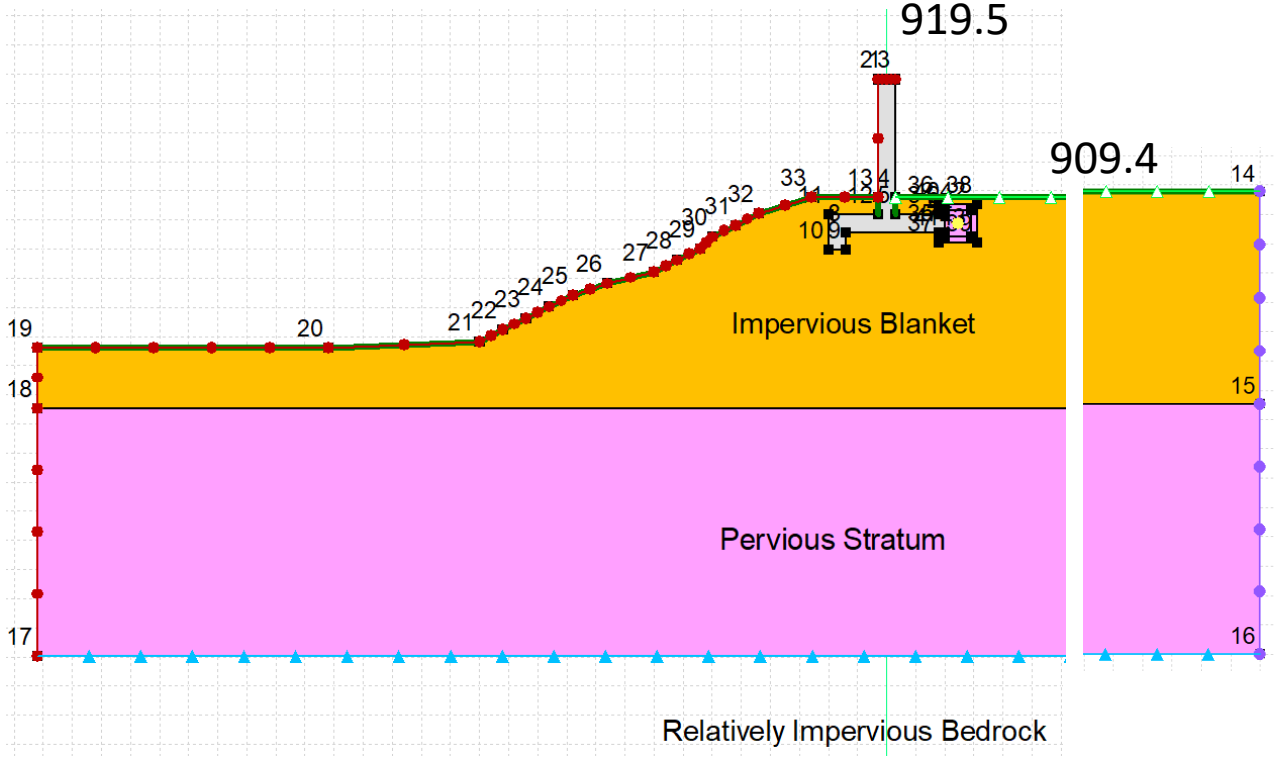
Used the example of a buried collector at Argentine and modeled the toe drain as a point in GeoStudio with a boundary condition of water total head equal to the ground surface. With loading to top of wall, I got a flow of approx. 3e-5 cfs/ft of wall length.

At Argentine, flow was two orders of magnitude higher, at 3e-3 cfs/ft of wall length. For that project, civil found that the 15” dia perforated pipe provided laminar flow and had sufficient capacity.

Civil subsequently checked expected flow for this floodwall toe drain (see Appendix C.3) and found that flow is laminar and pipe has sufficient capacity.

Derek





Name: Toe Drain
Kind: Water Total Head
Boundary Condition Type
Constant: 909.4 ft
Function:

Name: Landside Slope - Potential Seepage Face
Kind: Water Rate
Boundary Condition Type
Constant: 0 ft³/sec
Function:
Potential Seepage Face Review

Name: Tailwater Head
Kind: Water Total Head
Boundary Condition Type
Constant: 909.4 ft
Function:

Name: Bedrock - No Flow
Kind: Water Rate
Boundary Condition Type
Constant: 0 ft³/sec
Function:

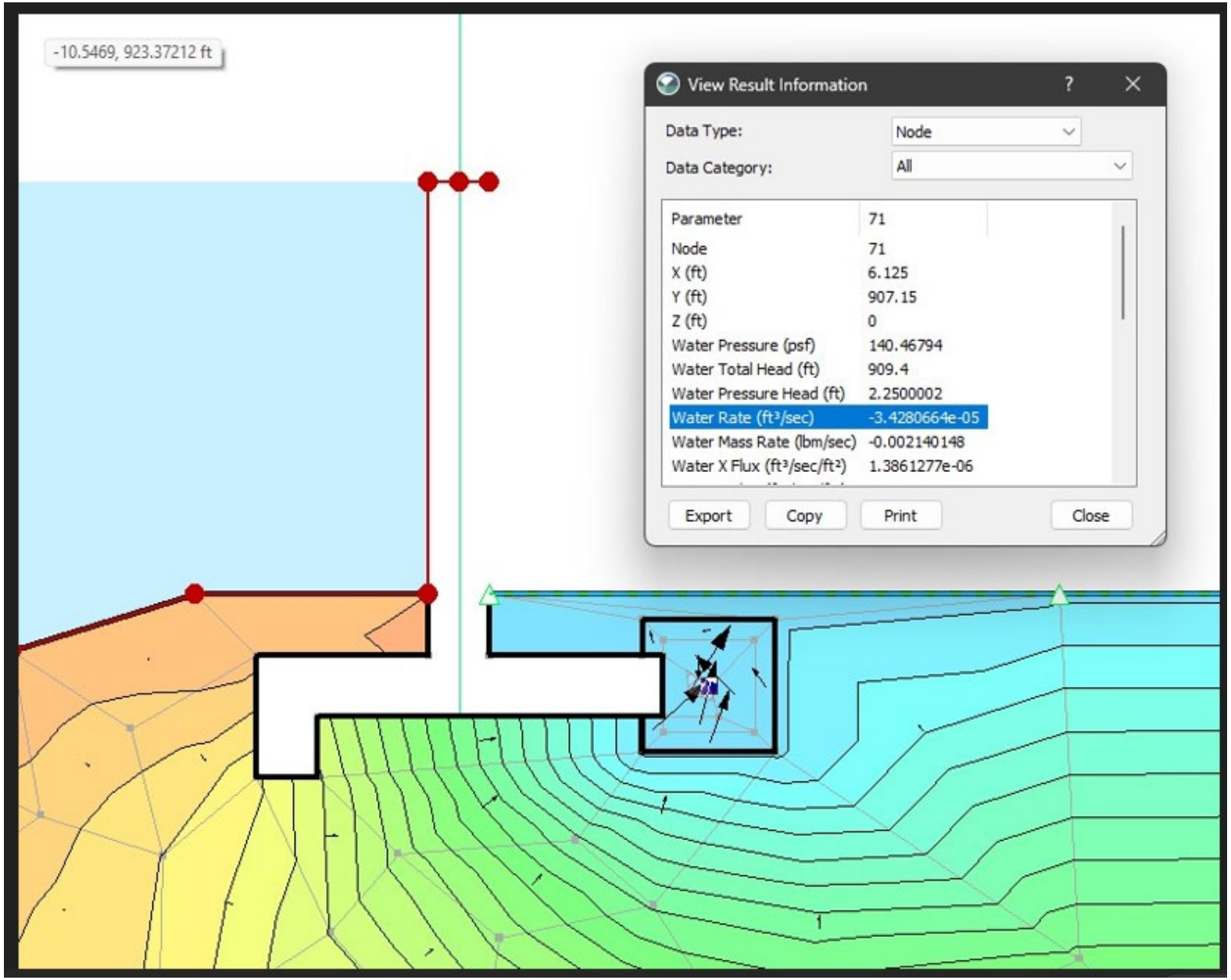
Name: Full Head
Kind: Water Total Head
Boundary Condition Type
Constant: 919.5 ft
Function:

Define Boundary Conditions

BC Category: All

All Boundary Conditions

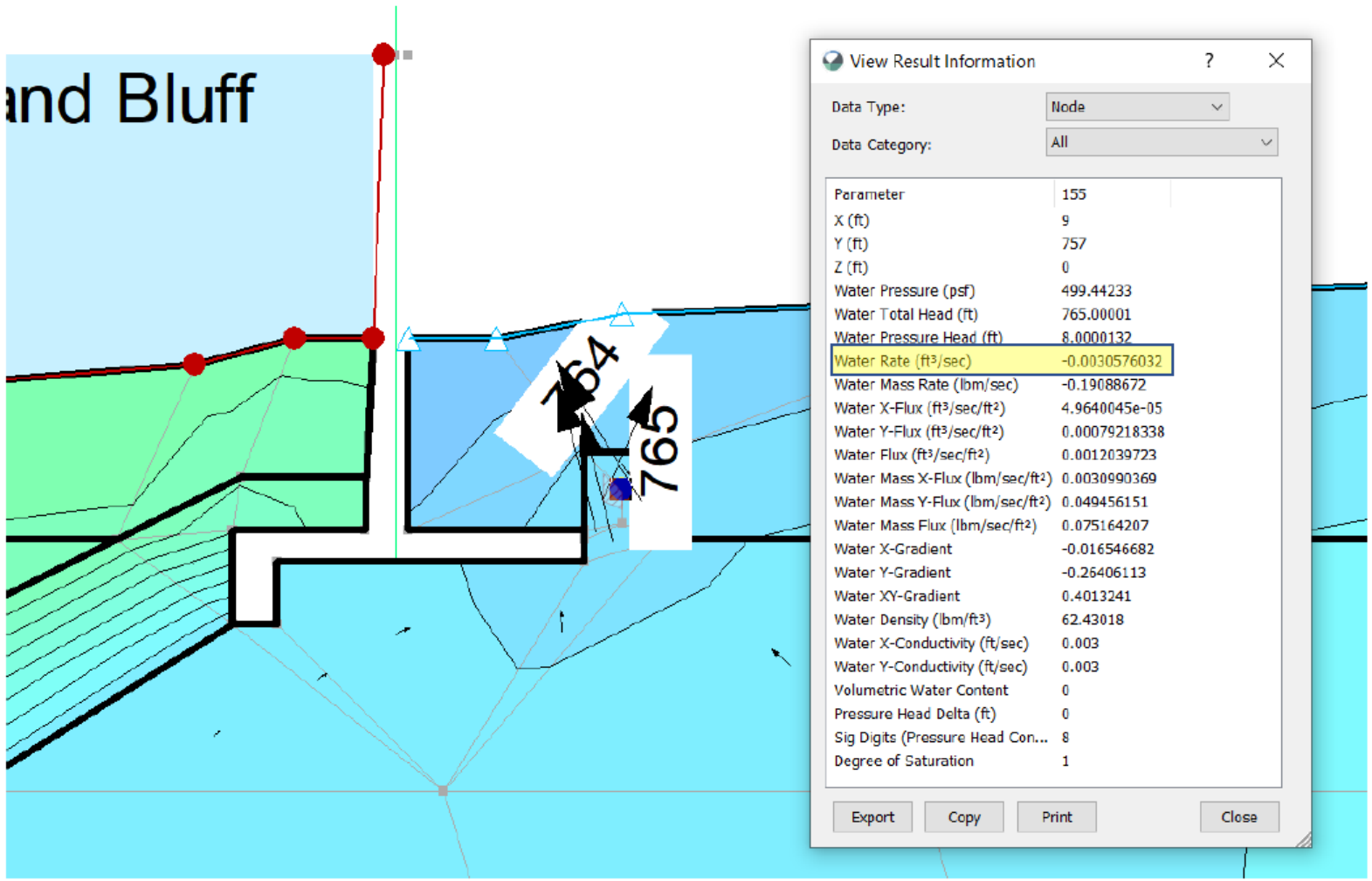
Name	Category	Hydraulic Boundary Type	Color
Bedrock - No Flow	Hydraulic	Water Rate	
Full Head	Hydraulic	Water Total Head	
Landside Slope - Potential Seepage Face	Hydraulic	Water Rate	
Tailwater Head	Hydraulic	Water Total Head	
Toe Drain	Hydraulic	Water Total Head	


-10.5469, 923.37212 ft
View Result Information
Data Type:
Node
Data Category:
All
Parameter 71
Node 71
X (ft) 6.125
Y (ft) 907.15
Z (ft) 0
Water Pressure (psf) 140.46794
Water Total Head (ft) 909.4
Water Pressure Head (ft) 2.2500002
Water Rate (ft³/sec) -3.4280664e-05
Water Mass Rate (lbm/sec) -0.002140148
Water X Flux (ft³/sec/ft²) 1.3861277e-06
Export
Copy
Print
Close

Argentine Comparison

Buried Collector Flow

Water Rate -0.0030576 cfs/ft of drain length
(negative flow out of system)



View Result Information

Data Type:	Node
Data Category:	All

Parameter	155
X (ft)	9
Y (ft)	757
Z (ft)	0
Water Pressure (psf)	499.44233
Water Total Head (ft)	765.00001
Water Pressure Head (ft)	8.0000132
Water Rate (ft³/sec)	-0.0030576032
Water Mass Rate (lbm/sec)	-0.19088672
Water X-Flux (ft³/sec/ft²)	4.9640045e-05
Water Y-Flux (ft³/sec/ft²)	0.00079218338
Water Flux (ft³/sec/ft²)	0.0012039723
Water Mass X-Flux (lbm/sec/ft²)	0.0030990369
Water Mass Y-Flux (lbm/sec/ft²)	0.049456151
Water Mass Flux (lbm/sec/ft²)	0.075164207
Water X-Gradient	-0.016546682
Water Y-Gradient	-0.26406113
Water XY-Gradient	0.4013241
Water Density (lbm/ft³)	62.43018
Water X-Conductivity (ft/sec)	0.003
Water Y-Conductivity (ft/sec)	0.003
Volumetric Water Content	0
Pressure Head Delta (ft)	0
Sig Digits (Pressure Head Con...	8
Degree of Saturation	1

Export Copy Print Close

Expected flow through 15” diameter toe drain is laminar per civil calcs (from Appendix C.3)

US Army Corps of Engineers
Kansas City District

PROJECT Upper Turkey Creek

TASK Toe Drain Pipe Design
Toe Drain A

DATE: 25-Mar-26
INITIALS: FNP
REVIEW DATE:
REVIEWER:

	Q	Q, cumulative	n	Length	Dia	Ground elevation	Discharge elevation	IE in	IE out	Slope	Flow Depth	Theta	P_w	A	Q_{calc}	V	*Flow Depth as percent of Diameter*	*Depth of Manhole Required*
	cfs	*cfs*		*ft*	*in*	*ft*	*ft*	*ft*	*ft*	*ft/ft*	*in*		*in*	ft^2	*cfs*	*ft/s*	*%*	*ft*
Start	0.0053026	0.00530264	0.011	155.96	15	931.67				0.80%	0.33	-1.2723	0.37	0.01	0.006	0.8	*2%*	
Manhole A-1	0.0064287	0.01173136	0.011	189.08	15	930.59				0.80%	0.47	-1.2139	0.45	0.01	0.012	1.1	*3%*	
Manhole A-2	0.0008905	0.01262182	0.011	26.19	15	928.63				0.80%	0.48	-1.2092	0.45	0.01	0.013	1.1	*3%*	
Standpipe A-3	0.0052013	0.01959828	0.011	152.98	15	928.39	928.89			0.60%	0.64	-1.1551	0.52	0.02	0.020	1.1	*4%*	
Manhole A-4	0.0017751	0.00177514	0.011	52.21	15	929.49				0.60%	0.24	-1.3152	0.32	0.00	0.003	0.6	*2%*	
End						929.74												

Re # Through Pipe Perforations (AASHTO Class II Perforation):

Q	0.000034	cfs/ft	
A	0.0069444	ft2/ft	(Min Inlet Area 1.5 in2/ft)
Vel=	0.005	ft/s	
Hyd. Dia=	0.018	ft	(Max Slot Dia 0.236 in, 6mm)
kin. Visc=	0.000012	ft2/s	
Re=	7.2945455	laminar	

Reynolds Number Check

$$Re = \frac{V \times D_h}{\upsilon}$$

Manning eq -> $Q = 1.49*A*R^{2/3}*S^{1/2}/n$

R = A/P_w
R = Hydraulic Radius
A = Flow Area
y = Flow Depth
P_w = Wetted Perimeter
S = Pipe Slope

NOTES & ASSUMPTIONS

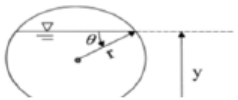
Flow depth cell will turn red if depth exceeds 85% of pipe diameter.
Qcalc cell will turn red if Qcalc is not within 5% of Q.
Spreadsheet does not account for tail water effects.
Header pipe at first manhole is 1.5 ft below discharge elevation

Rational Method -> Q = CiA

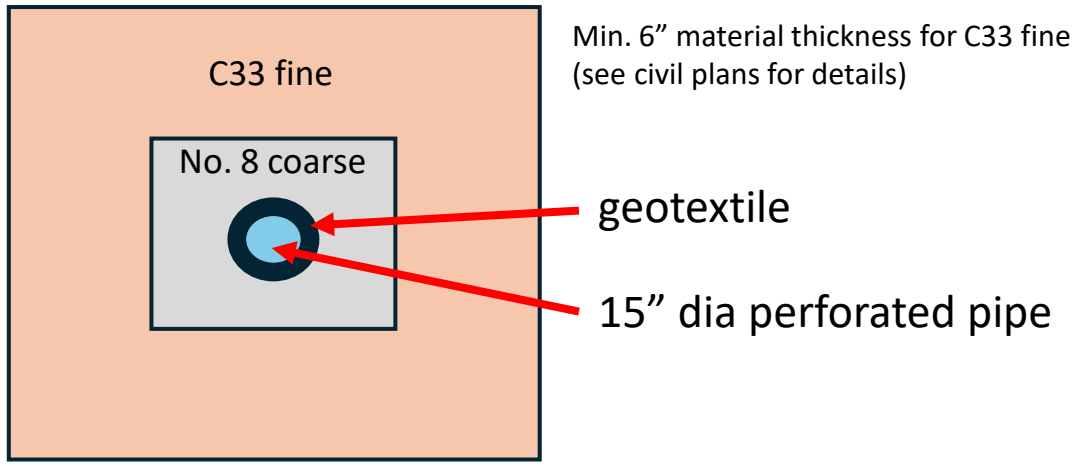
Q = Runoff (cfs)
C = Runoff Coefficient = 0.3+0.6*Impervious Area (APWA Page 18 Sec 5600)
i = Rainfall Intensity (in/hr)
A = Area (ac)

Pipe Geometry

$A = r^2(\pi/2+\theta+\sin\theta\cos\theta)$
$B = 2r\cos\theta$
$\theta = \sin^{-1}(y-r)/r$



Toe Drain Filter Compatibility



Min. 6” material thickness for C33 fine and for No. 8 coarse
(see civil plans for details)

Check ASTM C33 Fine Aggregate FILTER with Clay Subgrade BASE

per EM 1110-2-1913, Appendix D (30 April 2000)

Filter Requirements

D_{15} filter ≤ 9* d_{85} base for Base Soil Category 1

But not <0.2 mm

Say d_{85} for clay is 0.075 mm

D_{15} filter ≤ 0.675 mm

Permeability Requirements

D_{15} filter ≥ 3 to 5 * d_{15} base

Say d_{15} for clay is ~0.075 mm

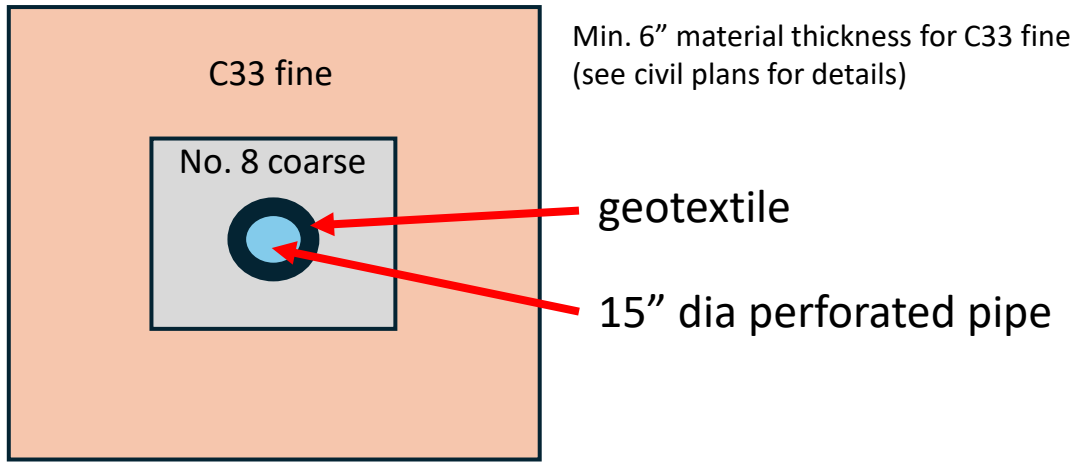
D_{15} filter ≥ 0.225 to 0.375mm

TABLE 1 Grading Requirements for Fine Aggregate

Sieve (Specification E11)	Percent Passing
9.5-mm (⅜-in.)	100
4.75-mm (No. 4)	95 to 100
2.36-mm (No. 8)	80 to 100
1.18-mm (No. 16)	50 to 85
600-µm (No. 30)	25 to 60
300-µm (No. 50)	5 to 30
150-µm (No. 100)	0 to 10
75-µm (No. 200)	0 to 3.0[A,B]

Requirements	C33 Fine	Satisfactory
D_{15} filter ≤ 0.675 mm	D_{15} is ~0.3 mm	Yes
D_{15} filter ≥ 0.225 to 0.375 mm	D_{15} is ~0.3 mm	Yes
Max particle size 75 mm (3”)	100% passing 9.5 mm (3/8”)	Yes
Max 5% passing No. 200	0-3% passing No. 200	Yes
Relatively uniform gradation	Relatively uniform gradation	Yes

Toe Drain Filter Compatibility



Min. 6” material thickness for C33 fine and for No. 8 coarse
(see civil plans for details)

Check ASTM C33 No. 8 Coarse Aggregate FILTER with ASTM C33 Fine Aggregate BASE

per EM 1110-2-1913, Appendix D (30 April 2000)

<u>Filter Requirements</u>

D_{15} filter ≤ 4 to 5 * d_{85} base

Smallest d_{85} for C33 fine is 1.18 mm

D_{15} filter ≤ 4.72 to 5.9 mm

<u>Permeability Requirements</u>

D_{15} filter ≥ 3 to 5 * d_{15} base

Largest d_{15} for C33 fine is ~0.45 mm

D_{15} filter ≥ 1.35 to 2.25 mm

Requirements	C33 No. 8 Coarse	Satisfactory
D_{15} filter ≤ 4.72 to 5.9 mm	D_{15} is 3 to 5 mm	Yes
D_{15} filter ≥ 1.35 to 2.25 mm	D_{15} is 3 to 5 mm	Yes
Max particle size 75 mm (3”)	100% passing 12.5 mm (½”)	Yes
Max 5% passing No. 200	Only 0-5% passing No. 16	Yes
Relatively uniform gradation	Relatively uniform gradation	Yes

(See gradations on next page)

Toe Drain Filter Compatibility

TABLE 1 Grading Requirements for Fine Aggregate

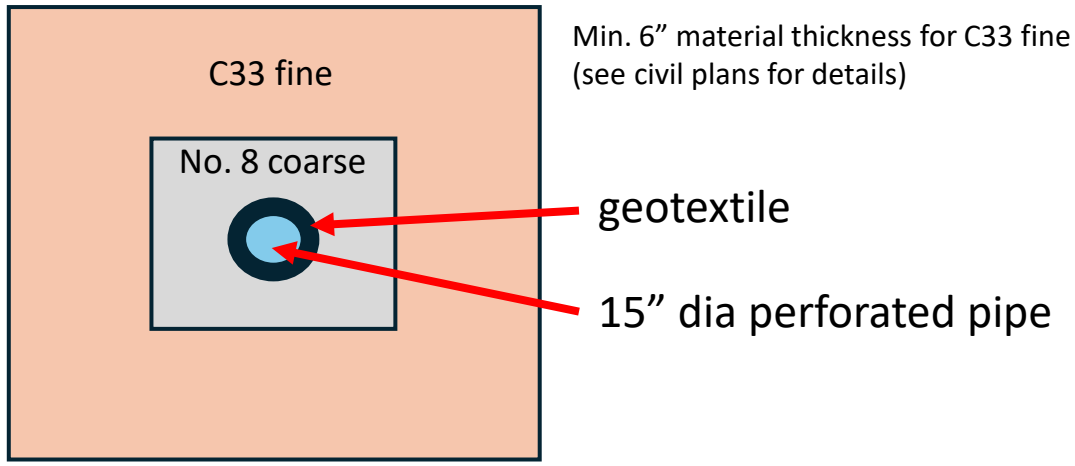
Sieve (Specification E11)	Percent Passing
9.5-mm (⅜-in.)	100
4.75-mm (No. 4)	95 to 100
2.36-mm (No. 8)	80 to 100
1.18-mm (No. 16)	50 to 85
600-µm (No. 30)	25 to 60
300-µm (No. 50)	5 to 30
150-µm (No. 100)	0 to 10
75-µm (No. 200)	0 to 3.0[A,B]

TABLE 3 Grading Requirements for Coarse Aggregates

Size Number	Nominal Size (Sieves with Square Openings)	Amounts Finer than Each Laboratory Sieve (Square-Openings), Mass Percent													
		100 mm (4 in.)	90 mm (3½ in.)	75 mm (3 in.)	63 mm (2½ in.)	50 mm (2 in.)	37.5 mm (1½ in.)	25.0 mm (1 in.)	19.0 mm (¾ in.)	12.5 mm (½ in.)	9.5 mm (⅜ in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	300 µm (No.50)
1	90 to 37.5 mm (3½ to 1½ in.)	100	90 to 100	...	25 to 60	...	0 to 15	...	0 to 5	...	...	...	...	...	...
2	63 to 37.5 mm (2½ to 1½ in.)	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...
3	50 to 25.0 mm (2 to 1 in.)	...	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...
357	50 to 4.75 mm (2 in. to No. 4)	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	...	0 to 5	...	...	...
4	37.5 to 19.0 mm (1½ to ¾ in.)	...	...	...	...	100	90 to 100	20 to 55	0 to 15	...	0 to 5	...	...	...	...
467	37.5 to 4.75 mm (1½ in. to No. 4)	...	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	0 to 5	...	...	...
5	25.0 to 12.5 mm (1 to ½ in.)	...	...	...	...	...	100	90 to 100	20 to 55	0 to 10	0 to 5	...	...	...	...
56	25.0 to 9.5 mm (1 to ⅜ in.)	...	...	...	...	...	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	...	...	...
57	25.0 to 4.75 mm (1 in. to No. 4)	...	...	...	...	...	100	95 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...
6	19.0 to 9.5 mm (¾ to ⅜ in.)	...	...	...	...	...	...	100	90 to 100	20 to 55	0 to 15	0 to 5	...	...	...
67	19.0 to 4.75 mm (¾ in. to No. 4)	...	...	...	...	...	...	100	90 to 100	...	20 to 55	0 to 10	0 to 5	...	...
7	12.5 to 4.75 mm (½ in. to No. 4)	...	...	...	...	...	...	...	100	90 to 100	40 to 70	0 to 15	0 to 5	...	...
8	9.5 to 2.36 mm (⅜ in. to No. 8)	...	...	...	...	...	...	...	...	100	85 to 100	10 to 30	0 to 10	0 to 5	...
89	9.5 to 1.18 mm (⅜ in. to No. 16)	...	...	...	...	...	...	...	...	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5
9[A]	4.75 to 1.18 mm (No. 4 to No. 16)	...	...	...	...	...	...	...	...	...	100	85 to 100	10 to 40	0 to 10	0 to 5

[A] Size number 9 aggregate is defined in Terminology C125 as a fine aggregate. It is included as a coarse aggregate when it is combined with a size number 8 material to create a size number 89, which is a coarse aggregate as defined by Terminology C125.

Toe Drain Filter Compatibility



Min. 6” material thickness for C33 fine and for No. 8 coarse
(see civil plans for details)

Check Perforated Pipe Opening Size limit for ASTM C33 No. 8 Coarse Aggregate

per EM 1110-2-1913, Appendix D (30 April 2000)

Min D_{50} filter material/(hole diameter or slot width) ≥ 1.0

D_{50} of ASTM C33 No. 8 Coarse Filter ranges from 6 to 7 mm

Hole diameter ≤ 6 mm

AASHTO M294 Class 2 HDPE Type SP Pipe normally allows 5 mm to 10 mm perforations. Based on previous projects, it was difficult to limit hole diameter to 6 mm.

Use geotextile to provide filter compatibility between perforated pipe and coarse aggregate

APPENDIX C – CIVIL

C.1 Storm Drainage Matrix
C.2 Non-Storm Utility Matrix
C.3 Toe Drain Example Calculation

This page was intentionally left blank for duplex printing.

APPENDIX C.1 – STORM DRAINAGE MATRIX

APPENDIX C.1 STORM UTILITY MATRIX

UTILITY TABLE																							
EXISTING														PROPOSED									
ID	PLAN SHEET	UTILITY	OWNER	GIS OR SURVEY	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	LEFT/RIGHT BANK	SIZE (IN)	MATERIAL	OUTFALL INVERT (FT)	DESCRIPTION	SUE QUALITY LEVEL	CONFLICT	PROPOSED ACTION	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	SIZE (IN)	MATERIAL	OUTFALL INVERT (FT)	LINEAR LENGTH	STRUCTURES	NEW PIPE SYSTEM
ST-01	CU101	STORM	CITY OF MERRIAM	BOTH	1+74.34	1+77.11	N/A	LEFT	120 X 36	RCB	923.46	BOX CULVERT FROM RUSSEL STOVER	D	TRIBUTARY REALIGNMENT	GRADE DITCH FROM OUTLET	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-02	CU101	STORM	CITY OF MERRIAM	GIS	2+81.45	2+87.84	N/A	LEFT	24	CMP	929.14	DRAINAGE FROM DOWN MASTIN ST TO CHANNEL	D	TRIBUTARY REALIGNMENT & WALL (LB)	RELOCATE	2+68.14	2+91.82	N/A	24	RCP	923.5	-169.26 +251.54	(2) CURB INLET (1) OUTFALL W/ FLAPGATE	ST-02A
ST-03	CU101	STORM	CITY OF MERRIAM	SURVEY	5+76.28	5+76.67	5+76.48	LEFT	12	CMP	924.1	MIDDLE OF 62ND TERRACE	D	TRIBUTARY REALIGNMENT & LEVEE (LB)	RELOCATE	5+75.60	5+76.47	5+76.12	24	RCP	922.01	-26.29 +119.31	(1) CURB INLET (1) OUTFALL W/ FLAPGATE	ST-03A
ST-04	CU101	STORM	CITY OF MERRIAM	BOTH	7+96.04	8+81.10	N/A	RIGHT	18	RCP	UNKNOWN	DRAINAGE FROM FRONTAGE RD	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-05	CU102	STORM	CITY OF MERRIAM	BOTH	10+02.51	10+19.41	10+17.49	LEFT	24	CMP	UNKNOWN	PARKING LOT OF SKATE CITY	D	WALL (LB)	RELOCATE	10+16.43	10+19.41	10+17.49	36	RCP	918.98	-33.45 +24.96	(1) FLUSH INLET (1) OUTFALL W/ FLAPGATE	ST-05A
ST-06	CU102	STORM	CITY OF MERRIAM	SURVEY	10+20.02	10+51.88	N/A	RIGHT	15	RCP	927.72	DRAINAGE FROM SHAWNEE MISSION PARKWAY	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-07	CU102	STORM	CITY OF MERRIAM	SURVEY	11+83.92	12+22.96	N/A	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	CULVERT UNDER SHAWNEE MISSION PARKWAY ENTRY RAMP	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-08	CU102	STORM	CITY OF MERRIAM	SURVEY	15+52.01	15+74.31	15+51.86	RIGHT	18	CMP	918.21	DRAINAGE BEHIND SAELA PEST CONTROL	D	WALL (RB)	RELOCATE W/ ST-10A	N/A	N/A	N/A	N/A	N/A	N/A	-29.2	SEE ST-10A	SEE ST-10A
ST-09	CU102	STORM	CITY OF MERRIAM	SURVEY	15+66.13	17+05.16	16+80.11	LEFT	36	CMP	914.76	DRAINAGE FROM 62ND ST	D	WALL (LB)	RELOCATE W/ GRADED DITCH FROM SKATE CITY	16+75.81	17+00.37	16+84.09	36	RCP	913.63	-56.92 +56.15	(1) FLUSH INLET (1) OUTFALL W/ FLAPGATE	ST-09A
ST-10	CU102-CU103	STORM	CITY OF MERRIAM	SURVEY	19+99.10	20+21.79	20+02.05	RIGHT	18	CMP	913.82	DRAINAGE BEHIND KRAV MAGA & FITNESS CENTER	D	WALL & LEVEE (RB)	RELOCATE	15+39.77	20+36.23	20+08.49	24	RCP	912.03	-32.06 +468.19	(2) FLUSH INLET (1) OUTFALL W/ FLAPGATE	ST-10A
ST-11	CU102-CU103	STORM	CITY OF MERRIAM	SURVEY	19+81.00	20+62.26	N/A	LEFT	36	CMP	914.39	DRAINAGE FROM 62ND ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-12	CU103	STORM	CITY OF MERRIAM	SURVEY	22+59.02	22+59.02	22+59.02	RIGHT	6	PVC	919.59	ROOF DRAIN	D	LEVEE (RB)	RELOCATE	21+88.54	21+93.29	21+90.40	DUAL 48	RCP	911.33	-21.05 +57.6	(1) LARGE AREA INLET (1) OUTFALL W/ FLAPGATES	ST-12A
ST-13	CU103	STORM	CITY OF MERRIAM	SURVEY	22+93.81	22+93.81	22+93.81	RIGHT	6	PVC	919.36	ROOF DRAIN	D	LEVEE (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-22.6	NONE	
ST-14	CU103	STORM	CITY OF MERRIAM	SURVEY	23+25.73	23+25.73	23+25.73	RIGHT	6	PVC	919.27	ROOF DRAIN	D	LEVEE (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-27.5	NONE	
ST-15	CU103	STORM	CITY OF MERRIAM	SURVEY	25+05.46	26+06.21	N/A	LEFT	42	CMP	908.68	DRAINAGE FROM 61ST ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-16	CU103	STORM	CITY OF MERRIAM	BOTH	23+60.24	27+48.93	26+96.74	RIGHT	24	RCP	908.43	DRAINAGE FROM 61ST ST	D	LEVEE (RB)	RELOCATE	27+35.02	27+56.91	27+45.09	24	RCP	905.93	/125.62 +109.05	(1) FLUSH INLET (1) OUTFALL W/ FLAPGATE	ST-16A
ST-17	CU104	STORM	CITY OF MERRIAM	SURVEY	32+36.44	32+42.12	N/A	LEFT	24	CMP	906.6	DRAINAGE AT END OF CAMPBELL ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-18	CU104	STORM	CITY OF MERRIAM	SURVEY	34+52.88	36+23.81	PARALLEL	LEFT	15	RCP	N/A	DRAINAGE FROM 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-19	CU104	STORM	CITY OF MERRIAM	SURVEY	35+26.86	36+39.38	36+27.39	LEFT	38	RCPA	906.46	DRAINAGE BEHIND HOUSES AND FROM 59TH TERRACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-20	CU104	STORM	CITY OF MERRIAM	ASSUMED	34+04.12	38+46.74	38+31.83	RIGHT	48	CMP	UNKNOWN	DRAINAGE ALONG MERRIAM DR SOUTH OF QUAKER CREEK	D	WALL (RB)	RELOCATE	37+90.96	37+98.02	37+92.91	48	RCP	898.52	-62.46 +56.35	(1) MANHOLE (1) OUTFALL W/ FLAPGATE	ST-20A
ST-21	CU104	STORM	CITY OF MERRIAM	SURVEY	38+70.28	38+93.56	38+70.89	RIGHT	84 X 84	RCB	900.71	QUAKER CREEK DRAINAGE BOX	D	WALL (RB)	ADD FLAPGATE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	(1) OUFALL W/ FLAPGATE	NEW STRUCTURE, SEE STRUCTURAL
ST-22	CU104	STORM	CITY OF MERRIAM	SURVEY	39+17.83	39+19.89	39+23.36	RIGHT	18	CPP	904.24	DRAINAGE FROM MERRIAM DRIVE SOUTH OF JOHNSON DR BRIDGE	D	WALL (RB)	RELOCATE	39+18.50	39+20.60	39+20.12	24	RCP	897.64	-48.54 +42.91	(1) CURB INLET (1) OUTFALL W/ FLAPGATE	ST-22A
ST-23	CU104	STORM	CITY OF MERRIAM	SURVEY	39+85.01	39+90.49	39+87.51	LEFT	18	CMP	907.13	DRAINAGE FROM CAMPBELL LANE	D	WALL (LB)	RELOCATE	39+85.01	39+88.61	39+87.51	36	RCP	900.68	-45.87 +56.83	(1) CURB INLET (1) OUTFALL W/ FLAPGATE	ST-23A
ST-24	CU105	STORM	CITY OF MERRIAM	SURVEY	40+70.08	40+72.00	N/A	RIGHT	2	PVC	900.53	UNKNOWN DRAIN	D	WALL (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-25	CU105	STORM	CITY OF MERRIAM	SURVEY	40+82.45	40+82.45	N/A	RIGHT	3	PVC	909.37	UNKNOWN DRAIN	D	WALL (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-26	CU105	STORM	CITY OF MERRIAM	SURVEY/ ASSUMED	41+13.13	41+72.47	41+62.09	RIGHT	18	CIP	902.75	DRAINAGE FROM JOHNSON DRIVE	D	WALL (RB)	RELOCATE	41+55.48	41+73.41	41+62.39	24	RCP	897.6	-34.76 +45.54	(1) CURB INLET (1) OUTFALL W/ FLAPGATE	ST-26A
ST-27	CU105	STORM	CITY OF MERRIAM	SURVEY	42+03.39	42+39.50	N/A	LEFT	18	CMP	892.05	DRAINAGE FROM JOHNSON DRIVE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-28	CU105	STORM	CITY OF MERRIAM	GIS	42+03.39	43+61.62	N/A	LEFT	15	CMP	N/A	DRAINAGE TO ST-27	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-29	CU105	STORM	CITY OF MERRIAM	SURVEY	42+01.84	42+01.84	N/A	RIGHT	6	PVC	908.88	UNKNOWN DRAIN	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	0.0	NONE	
ST-30	CU105	STORM	CITY OF MERRIAM	SURVEY	43+62.84	43+62.84	N/A	LEFT	12	CMP	899.53	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-31	CU105	STORM	CITY OF MERRIAM	SURVEY	44+80.16	44+89.33	N/A	LEFT	12	CMP	901.92	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-32	CU105	STORM	CITY OF MERRIAM	SURVEY	46+19.41	46+22.35	N/A	LEFT	12	CMP	902.24	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-33	CU105	STORM	CITY OF MERRIAM	SURVEY	48+27.64	48+27.64	N/A	LEFT	48	RCPA	899.71	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-34	CU105	STORM	CITY OF MERRIAM	SURVEY/ ASSUMED	47+89.14	49+71.24	49+74.58	RIGHT	24	RCP	894.78	SOUTH SIDE OF MERRIAM MARKETPLACE	D	LEVEE (RB)	RELOCATE INTO DETENTION BASIN	49+76.57	53+72.93	N/A	24 48 60	RCP	895.5	/119.55 +374.63 (24") +123.46 (48") +46.72 (60")	(3) CURB INLETS (1) MANHOLE (1) OUTFALL	ST-34A
ST-35	CU105	STORM	CITY OF MERRIAM	SURVEY	49+73.37	49+73.37	49+73.37	LEFT	12	CMP	906.41	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-36	CU106	STORM	CITY OF MERRIAM	SURVEY	52+23.12	52+32.97	N/A	RIGHT	18	RCP	N/A	NORTH SIDE OF MERRIAM MARKETPLACE	D	LEVEE (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-128.89 /67.81	SEE ST-34A	SEE ST-34A
ST-37	CU106	STORM	CITY OF MERRIAM	SURVEY	52+95.22	53+75.40	52+97.95	RIGHT	48	CMP	892.08	DRAINAGE FROM MERRIAM DRIVE	D	LEVEE (RB)	ABANDON	52+95.22	53+75.40	N/A	N/A	N/A	N/A	/385.11	SEE ST-34A	SEE ST-34A
ST-38	CU106	STORM	CITY OF MERRIAM	GIS	50+72.47	50+72.47	N/A	LEFT	QUAD 24"	CMP	904.4	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-39	CU106	STORM	CITY OF MERRIAM	SURVEY	56+99.46	57+48.88	57+17.58	RIGHT	12	CMP	897.41	DRAINAGE BEHIND BAKERS MARBLE	D	LEVEE (RB)	RELOCATE FROM DETENTION BASIN	57+60.59	57+76.32	57+62.89	DUAL 48	RCP	893.28	/134.36 +176.75	(1) LARGE AREA INLET (1) OUTFALL W/ FLAPGATE	ST-39A
ST-40	CU106	STORM	CITY OF MERRIAM	SURVEY	58+26.41	58+26.41	N/A	LEFT	12	CMP	904.25	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-41	CU106	STORM	CITY OF MERRIAM	SURVEY	58+14.35	58+74.80	N/A	LEFT	24	RCP	905.34	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	
ST-42	CU107	STORM	CITY OF MERRIAM	SURVEY	61+88.21	61+94.26	N/A	LEFT	24	CMP	904.82	DRAINAGE UNDER TRAIL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0.0	NONE	

APPENDIX C.1 STORM UTILITY MATRIX

UTILITY TABLE																						
EXISTING														PROPOSED								
ID	PLAN SHEET	UTILITY	OWNER	GIS OR SURVEY	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	LEFT/RIGHT BANK	SIZE (IN)	MATERIAL	OUTFALL INVERT (FT)	DESCRIPTION	SUE QUALITY LEVEL	CONFLICT	PROPOSED ACTION	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	SIZE (IN)	MATERIAL	OUTFALL INVERT (FT)	LINEAR LENGTH	STRUCTURES	NEW PIPE SYSTEM
ST-43	CU107	STORM	CITY OF MERRIAM	SURVEY	61+87.70	62+54.11	62+12.88	RIGHT	24	CMP	889.96	WEST SIDE OF MERRIAM BRIDGE	D	WALL (RB)	RELOCATE	61+87.70	62+54.11	62+12.88	24	RCP	889.65	-113.51 +115.13	(1) CURB INLET (1) OUTFALL W/ FLAPGATE	ST-43A
ST-44	CU107	STORM	CITY OF MERRIAM	SURVEY	63+86.49	63+91.25	63+90.19	RIGHT	18	RCP	902.35	EAST SIDE OF MERRIAM BRIDGE	D	WALL (RB)	RELOCATE	63+86.49	64+63.80	N/A	24	RCP	899.74	-33.07 /67.96 +133.28	(1) FLUSH INLET SEE ST-45A	ST-44A
ST-45	CU107	STORM	CITY OF MERRIAM	SURVEY	64+63.80	64+66.06	64+64.42	RIGHT	18	RCP	902.64	DRAINAGE FROM BUS DEPOT	D	WALL (RB)	RELOCATE	63+91.25	64+66.06	64+64.42	24	RCP	896.67	-34.58 +52.33	(1) FLUSH INLET (1) OUTFALL W/ FLAPGATE	ST-45A
ST-46	CU107	STORM	CITY OF MERRIAM	SURVEY	65+24.39	65+35.61	65+25.00	RIGHT	15	CMP	895.03	DRAINAGE FROM BUS DEPOT	D	WALL (RB)	RELOCATE	64+63.80	65+20.48	N/A	24	RCP	901.83	-13.88 /34.99 +81.73	(1) FLUSH INLET SEE ST-45A	ST-46A

APPENDIX C.2 – NON-STORM UTILITY MATRIX

APPENDIX C.2 NON-STORM UTILITY MATRIX

UTILITY TABLE

EXISTING																PROPOSED										
ID	HNTB ID	PLAN SHEET	UTILITY	OWNER	GIS OR SURVEY	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	LEFT/RIGHT BANK	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	OVERHEAD / UNDERGROUND	DESCRIPTION	SUE QUALITY LEVEL	CONFLICT	PROPOSED ACTION	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	LINEAR LENGTH	STRUCTURES	NEW SYSTEM / NOTES
ATT-01	ATT-01	CU101	COMMUNICATION	AT&T	ASBUILT	0+00.00	2+76.00	NONE	LEFT	UNKNOWN	COPPER	UNKNOWN	UNDERGROUND	PRIMARY ALONG MASTIN ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-01	GAS-01	CU101	GAS	KANSAS GAS SERVICE	BOTH	0+00.00	2+86.85	NONE	LEFT	2	APCS	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG MASTIN ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-02	GAS-03	CU101	GAS	KANSAS GAS SERVICE	ASBUILT	0+00.00	6+38.66	NONE	BOTH	2	APCS	UNKNOWN	UNDERGROUND	PRIMARY TO ENTERPRISE/LONG JOHN SILVERS	D	TRIBUTARY REALIGNMENT	ABANDON	ALL	ALL	N/A	N/A	N/A	N/A	/670.00	NONE	
UGE-01	UGE-01	CU101	ELECTRIC	EVERGY	SURVEY	0+00.00	2+55.80	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	PRIMARY ALONG EAST SIDE OF MASTIN ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
WL-01	WL-01	CU101	WATER	WATERONE	BOTH	0+00.00	2+72.60	NONE	LEFT	6	CAS/DIP	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG MASTIN ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-02	UGE-04	CU101	ELECTRIC	CITY OF MERRIAM	SURVEY	0+06.26	9+20.84	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG SHAWNEE MISSION PKWY	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-01	SS-01	CU101 - CU102	SANITARY	JOCO WASTEWATER	SURVEY/ ASSUMED	0+32.41	12+13.38	2+99.65	LEFT	24	VCP/PVC	9'-12' / 919-927	UNDERGROUND	PRIMARY ALONG MASTIN ST & 62ND TERRACE	D	WALL (LB)	RECONNECT	11+96.86	12+13.38	N/A	N/A	N/A	N/A	/20.82	NONE	
WL-02	WL-02	CU101 - CU102	WATER	WATERONE	BOTH	0+47.06	14+79.79	NONE	BOTH	8	CAS/DIP	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG NORTH SHAWNEE MISSION PKWY	D	TRIBUTARY REALIGNMENT	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-01	CHAR-01	CU101	COMMUNICATION	CHARTER	ASBUILT	0+48.95	2+72.94	1+59.49	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ROUTE ALONG MASTIN ST	D	TRIBUTARY REALIGNMENT	RELOCATE	0+48.95	2+83.22	1+32.81	N/A	FIBER / CABLE	+20	-470.26 +474.03	SEE OHE-01	CHAR-01A
OHE-01	OHE-01	CU101	ELECTRIC	EVERGY	SURVEY	0+50.64	2+72.63	NONE	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER ALONG MASTIN ST	D	TRIBUTARY REALIGNMENT	RELOCATE	0+48.95	2+83.22	1+32.81	N/A	CABLE	+30	-470.26 +474.03	(3) POWER POLE	OHE-01A
GF-01	GF-01	CU101	COMMUNICATION	GOOGLE FIBER	ASBUILT	0+52.32	2+72.32	1+59.49	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ROUTE ALONG MASTIN ST	D	TRIBUTARY REALIGNMENT	RELOCATE	0+48.95	2+83.22	1+32.81	N/A	FIBER / CABLE	+20	-470.26 +474.03	SEE OHE-01	GF-01A
VER-01		CU101 - CU102	COMMUNICATION	VERIZON	ASBUILT	0+53.71	13+14.82	NONE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	PRIMARY ALONG NORTH SIDE OF SHAWNEE MISSION PKWY	D	TRIBUTARY REALIGNMENT	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-01.1		CU101	COMMUNICATION	CHARTER	ASBUILT	1+03.22	2+52.42	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	CROSSES MASTIN ST AT 62ND TERRACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-01.1		CU101	GAS	KANSAS GAS SERVICE	SURVEY	1+10.58	1+13.88	NONE	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO RUSSEL STOVER	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-03	UGE-02	CU101	ELECTRIC	CITY OF MERRIAM	ASBUILT	1+42.76	2+82.01	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG WEST SIDE OF MASTIN ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-01.1	GF-03	CU101	COMMUNICATION	GOOGLE FIBER	ASBUILT	1+48.55	2+53.23	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	CROSSES MASTIN ST AT 62ND TERRACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-01.2		CU101	COMMUNICATION	CHARTER	ASBUILT	1+58.91	3+76.04	1+58.91	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO ENTERPRISE	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-144.34	NONE	
GF-01.2	GF-01	CU101	COMMUNICATION	GOOGLE FIBER	ASBUILT	1+58.91	3+84.95	1+58.91	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO ENTERPRISE	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-169.65	NONE	
ATT-02	ATT-02	CU101	COMMUNICATION	AT&T	ASBUILT	1+59.37	4+45.67	1+59.37	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO ENTERPRISE	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-324.25	NONE	
ATT-01.1	ATT-01	CU101	COMMUNICATION	AT&T	ASBUILT	1+69.02	1+88.28	NONE	LEFT	UNKNOWN	COPPER	UNKNOWN	UNDERGROUND	CROSSES MASTIN ST AT ST-01 FOR GAS METER MONITOR	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-04.1	UGE-03	CU101	ELECTRIC	PRIVATE	SURVEY	1+70.33	5+04.99	1+70.33	CHANNEL	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	ENTERPRISE STREETLIGHTS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-372.23	NONE	
ATT-05		CU101	COMMUNICATION	AT&T	ASBUILT	2+20.27	2+95.67	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE ALONG EAST SIDE OF MASTIN ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
OHE-02.1	OHE-02	CU101	ELECTRIC	EVERGY	SURVEY	2+23.38	2+55.80	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER ALONG 62ND TERRACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	0	
ATT-03		CU101	COMMUNICATION	AT&T	ASBUILT	2+36.26	9+91.52	2+99.27	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE ALONG SOUTH SIDE OF 62ND TERRACE	D	WALL (LB)	RELOCATE	2+80.74	3+19.09	N/A	N/A	N/A	N/A	-43.35 +43.35	NONE	ATT-03A UP AND OVER
GF-02	GF-02	CU101	COMMUNICATION	GOOGLE FIBER	ASBUILT	2+51.88	9+94.85	2+99.27	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ROUTE ALONG 62ND TERRACE	D	LEVEE (LB)	RELOCATE	2+52.20	6+60.35	2+99.27	N/A	FIBER / CABLE	+15	-434.54 +434.54	SEE OHE-02	GF-02A
OHE-02.2		CU101	ELECTRIC	EVERGY	SURVEY	2+52.31	7+49.03	6+56.96	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO LONG JOHN SILVERS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-79.42	NONE	
CHAR-02	CHAR-02	CU101	COMMUNICATION	CHARTER	ASBUILT	2+52.42	9+94.85	2+99.27	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	BOTH	PRIMARY ROUTE ALONG 62ND TERRACE	D	LEVEE (LB)	RELOCATE	2+52.20	6+60.35	2+99.27	N/A	FIBER / CABLE	+15	-434.54 +434.54	SEE OHE-02	CHAR-02A
ATT-04	ATT-02	CU101	COMMUNICATION	AT&T	ASBUILT	2+52.43	9+97.86	3+05.03	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE ALONG NORTH SIDE OF 62ND TERRACE	D	WALL (LB)	RELOCATE	2+84.50	3+25.40	3+05.03	N/A	N/A	N/A	-43.12 +43.12	NONE	ATT-04A UP AND OVER
GAS-03	GAS-02	CU101 - CU102	GAS	KANSAS GAS SERVICE	BOTH	2+52.68	10+67.71	3+03.68	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO SKATE CITY	D	LEVEE (LB)	RELOCATE	2+74.08	3+67.23	3+03.68	N/A	N/A	N/A	-71.69 +71.69	NONE	GAS-03A UP AND OVER
OHE-02	OHE-03	CU101	ELECTRIC	EVERGY	SURVEY	2+55.80	6+60.41	3+02.84	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER ALONG 62ND TERRACE	D	LEVEE (LB)	RELOCATE	2+55.80	6+60.41	3+02.84	N/A	CABLE	+30	-434.70 +434.70	(3) POWER POLE	OHE-02A
ATT-06		CU101 - CU102	COMMUNICATION	AT&T	ASBUILT	2+73.69	11+95.78	3+00.00	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO SKATE CITY	D	WALL (LB)	RELOCATE	2+80.92	3+19.95	3+00.00	N/A	FIBER / CABLE	-3	-43.29 +43.29	NONE	ATT-06A UP AND OVER
GF-02.1	GF-02	CU101	COMMUNICATION	GOOGLE FIBER	ASBUILT	2+96.32	4+64.37	2+96.32	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICES BARON COLLISON CENTER	D	LEVEE (LB)	RELOCATE	2+96.32	3+81.32	N/A	N/A	FIBER / CABLE	-3	-43.93 +31.65	(1) 45 BEND	GF-02.1A
SS-03.1		CU101	SANITARY	JOCO WASTEWATER	SURVEY	3+82.93	6+41.53	NONE	LEFT	8	VCP	7'-10' / 925-928	UNDERGROUND	SERVICE FROM BARON COLLISON CENTER	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-04	UGE-03	CU101	ELECTRIC	PRIVATE	SURVEY	4+24.75	5+04.99	CHANNEL	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	ENTERPRISE STREETLIGHTS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-74.51	NONE	
GAS-02.1		CU101	GAS	KANSAS GAS SERVICE	ASBUILT	4+65.06	4+72.52	4+65.06	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO ENTERPRISE	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-61.64	NONE	
UGE-04.2	UGE-03	CU101	ELECTRIC	PRIVATE	SURVEY	5+04.99	5+60.43	CHANNEL	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	ENTERPRISE STREETLIGHTS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-176.72	NONE	
ATT-03.1		CU101	COMMUNICATION	AT&T	ASBUILT	5+31.60	9+86.51	6+67.10	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO BARON COLLISION CENTER	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-04.3	UGE-03	CU101	ELECTRIC	PRIVATE	SURVEY	5+83.69	6+56.09	CHANNEL	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	LONG JOHN SILVERS STREETLIGHTS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-72.83	NONE	
GAS-02.2		CU101	GAS	KANSAS GAS SERVICE	ASBUILT	6+20.15	6+34.53	6+20.15	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO LONG JOHN SILVERS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-60.13	NONE	
CHAR-02.1		CU101	COMMUNICATION	CHARTER	ASBUILT	6+36.67	6+61.02	6+61.02	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	BOTH	SERVICE TO LONG JOHN SILVERS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-145.45	NONE	
SS-03		CU101	SANITARY	JOCO WASTEWATER	SURVEY	6+41.53	6+42.75	NONE	LEFT	8	VCP	10'-11' / 920-924	UNDERGROUND	PRIMARY FROM NORTH 62ND TERRACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-03.2		CU101	SANITARY	JOCO WASTEWATER	SURVEY	6+41.53	7+80.68	NONE	LEFT	8	VCP	5'-10' / 924-926	UNDERGROUND	SERVICE FROM FACTORY MOTOR PARTS	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-02.2		CU101	COMMUNICATION	GOOGLE FIBER	ASBUILT	6+42.33	6+60.32	6+60.32	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO LONG JOHN SILVERS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-79.42	NONE	
UGE-04.4	UGE-03	CU101	ELECTRIC	PRIVATE	SURVEY	7+11.26	7+49.49	CHANNEL	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	LONG JOHN SILVERS STREETLIGHTS	D	TRIBUTARY REALIGNMENT	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-158.53	NONE	
SS-02	SS-02	CU101	SANITARY	JOCO WASTEWATER	SURVEY/ ASSUMED	7+77.27	9+84.84	9+79.93	LEFT	8	VCP/DIP	11'-14' / 917-920	UNDERGROUND	PRIMARY FROM ENTERPRISE AND LONG JOHN SILVERS	D	WALL (LB)	RELOCATE	8+98.88	9+80.98	9+52.45	8	PVC	11'-14' / 917-920	-72.71 +61.46	(2) MANHOLES	SS-02A
WL-03	WL-03	CU101	WATER	WATERONE	BOTH	8+09.48	8+45.82	8+09.48	LEFT	8	CAS	UNKNOWN	UNDERGROUND	CROSSING SHAWNEE MISSION PKWY	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-03.1		CU101	GAS	KANSAS GAS SERVICE	SURVEY	8+63.68	9+72.28	NONE	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO FACTORY MOTOR PARTS	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
WL-04	WL-04 & WL-09?	CU101 - CU102	WATER	WATERONE	BOTH	9+82.47	12+74.70	9+93.23	LEFT	6	CAS/DIP	UNKNOWN	UNDERGROUND	CROSSES UTC FROM SHAWNEE MISSION PKWY AND ENDS IN SKATE CITY PARKING LOT	D	WALL (LB)	RELOCATE	9+91.27	12+74.70	9+91.27	6	CAS	-8	-37.44 !89.27 +63.77	(1) HYDRANT (1) VALVE (1) TEE (1) 12.25 BEND	WL-04A
WL-05	WL-05	CU102	WATER	WATERONE	GIS	10+98.91	11+04.89	NONE	RIGHT	8	CAS	UNKNOWN	UNDERGROUND	CROSSES SHAWNEE MISSION PKWY FROM SOUTH TO WL-02	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-04	SS-03	CU102 - CU106	SANITARY	JOCO WASTEWATER	SURVEY	11+14.51	58+47.62	MULTIPLE	BOTH	42	PVC	12' / 887-912	UNDERGROUND	PRIMARY ALONG UTC	D	LEVEE (LB & RB) & WALL (LB & RB)	RELOCATE	11+14.51	58+47.62	MULTIPLE	42	PVC	12' / 887-912	-606.98 /2246.07 +2676.59	(15) MANHOLES	SS-04A
UGE-05	UGE-05	CU102	ELECTRIC	CITY OF MERRIAM	SURVEY	11+78.18	15+47.06	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG WEST SIDE OF RAMP TO SHAWNEE MISSION PKWY	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-06	UGE-06	CU102	ELECTRIC	CITY OF MERRIAM	SURVEY	12+73.57	15+48.72	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG EAST SIDE OF RAMP FROM SHAWNEE MISSION PKWY	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-04.1	SS-04	CU102	SANITARY	JOCO WASTEWATER	SURVEY	13+89.59	14+23.54	14+23.54	LEFT	8	VCP	16' / 912	UNDERGROUND	SERVICE FROM SKATE CITY	D	LEVEE (LB)	RELOCATE	13+89.59	14+23.54	N/A	8	PVC	16' / 912	/227.12 +99.57	(1) MANHOLE	SS-04.1A
SS-04.2	SS-05	CU102	SANITARY	JOCO WASTEWATER	SURVEY	14+23.54	16+13.38	14+23.54	LEFT	8	VCP	13' / 915	UNDERGROUND	SERVICE FROM MERRIAM GARDENS HEALTHCARE	D	LEVEE (LB)	RECONNECT	14+23.54	16+13.38	N/A	8	PVC	13' / 915	/45.44	SEE SS-04A	

APPENDIX C.2 NON-STORM UTILITY MATRIX

UTILITY TABLE

EXISTING																PROPOSED									
ID	HNTB ID	PLAN SHEET	UTILITY	OWNER	GIS OR SURVEY	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	LEFT/RIGHT BANK	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	OVERHEAD / UNDERGROUND	DESCRIPTION	SUE QUALITY LEVEL	CONFLICT	PROPOSED ACTION	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	LINEAR LENGTH	STRUCTURES	NEW SYSTEM / NOTES
ATT-07	ATT-03	CU102	COMMUNICATION	AT&T	ASBUILT	14+74.02	18+60.05	PARALLEL	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	ALONG UTC BEHIND 6230 MERRIAM DR BUILDING? NO IDEA WHAT IT SERVICES	D	WALL (RB)	REMOVE	14+75.24	18+60.05	N/A	N/A	N/A	N/A	-443.94	NONE	
GAS-04.1		CU102	GAS	KANSAS GAS SERVICE	ASBUILT	15+20.88	15+50.12	NONE	RIGHT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO PAELA PEST CONTROL	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-04	GAS-04 & GAS-10	CU102 - CU105	GAS	KANSAS GAS SERVICE	BOTH	15+22.56	40+92.59	27+57.29	RIGHT	2	PE	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG MERRIAM DR SOUTH OF JOHNSON DR	D	LEVEE (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
WL-06		CU102	WATER	WATERONE	BOTH	16+26.04	19+46.41	NONE	LEFT	8	DIP	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG 62ND ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-08	ATT-04	CU102 - CU104	COMMUNICATION	AT&T	ASBUILT	17+27.96	33+42.93	28+01.52	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	PRIMARY SERVICE ALONG MERRIAM DR/LN	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-03		CU102 - CU103	COMMUNICATION	CHARTER	ASBUILT	17+99.31	22+06.27	22+06.27	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY TO EVANS AUTO SERVICE, CONNECTS TO UG CHAR-06 TO OH CHAR-05	D	LEVEE (RB)	RELOCATE	21+13.56	22+21.21	N/A	N/A	FIBER / CABLE	+15	-126.59 +136.49	(1) POWER POLE	CHAR-03A
GF-03	GF-04	CU102 - CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	17+99.31	22+06.27	22+06.27	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY TO EVANS AUTO SERVICE	D	LEVEE (RB)	REMOVE	21+13.56	22+06.27	N/A	N/A	N/A	N/A	-126.59	NONE	
OHE-04	OHE-04	CU102 - CU103	ELECTRIC	CITY OF MERRIAM	SURVEY	18+13.41	20+34.40	PARALLEL	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	STREETLIGHT POWER ALONG W 62ND ST IN FRONT OF REHAB FACILITY	D	WALL (LB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-04.1	CHAR-03	CU102 - CU103	COMMUNICATION	CHARTER	ASBUILT	18+44.48	22+24.16	19+17.19	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MERRIAM GARDENS HEALTHCARE	D	WALL (LB)	RELOCATE	19+46.67	19+98.41	N/A	N/A	FIBER / CABLE	-3	62.92 +62.92	NONE	CHAR-04.1A UP AND OVER
GAS-05.1		CU102	GAS	KANSAS GAS SERVICE	BOTH	19+42.57	19+70.85	19+42.57	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO REHAB FACILITY	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-05		CU102 - CU103	GAS	KANSAS GAS SERVICE	ASBUILT	19+58.03	23+82.46	NONE	LEFT	2	PE	UNKNOWN	UNDERGROUND	SERVICE TO MERRIAM GARDENS HEALTHCARE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-03.1		CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	21+13.13	21+85.48	21+13.13	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICES SMITH AND SONS AUTO SALES	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
OHE-05	OHE-05	CU103	ELECTRIC	EVERGY	SURVEY	21+13.35	22+04.19	22+04.19	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER TO EVANS AUTO SERVICE	D	LEVEE (RB)	REMOVE	21+13.56	22+06.27	N/A	N/A	N/A	N/A	-126.59	NONE	
GAS-06	GAS-05	CU103	GAS	KANSAS GAS SERVICE	ASBUILT	21+90.82	23+01.28	21+90.82	RIGHT	2	PE	UNKNOWN	UNDERGROUND	PRIMARY TO EVANS AUTO SERVICE	D	LEVEE (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-06.1		CU103	GAS	KANSAS GAS SERVICE	ASBUILT	21+94.22	22+66.60	22+66.60	RIGHT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO EVANS AUTO SERVICE	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-90.82	NONE	
UGE-07	UGE-07	CU103	ELECTRIC	CITY OF MERRIAM	ASBUILT	21+96.69	27+85.92	21+96.69	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG W 62ND ST AND MERRIAM LN	D	LEVEE (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-03.1		CU103	COMMUNICATION	CHARTER	ASBUILT	22+06.27	23+32.33	22+06.27	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	CONNECTS OH CHAR-04 TO OH CHAR-05 ALONG 62ND ST	D	LEVEE (RB)	RECONNECT	22+06.27	22+06.27	N/A	N/A	N/A	N/A	-36.27	SEE CHAR-03A	CONNECT TO NEW POLE ON CHAR-03A
GF-03.2		CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	22+06.27	23+15.85	22+06.27	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO EVANS AUTO SERVICE	D	LEVEE (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-115.95	NONE	
SS-05	SS-07	CU103	SANITARY	JOCO WASTEWATER	GIS	22+09.22	23+23.53	NONE	LEFT	8	VCP	10" / 910	UNDERGROUND	PRIMARY FROM KNOX ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-07	SS-06	CU103	SANITARY	JOCO WASTEWATER	SURVEY	22+34.45	23+20.11	NONE	RIGHT	8	PVC	10" / 911	UNDERGROUND	PRIMARY ALONG EAST W 62ND ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
EF-01	EF-01	CU103	COMMUNICATION	EVERFAST	ASBUILT	24+31.01	26+51.01	26+51.01	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	BOTH	PRIMARY ROUTE CROSSING 61ST ST	D	LEVEE (RB)	RECONNECT	N/A	N/A	N/A	N/A	FIBER / CABLE	+20	-445.84 +446.12	SEE OHE-06A	EF-01A
OHE-06	OHE-06	CU103	ELECTRIC	EVERGY	SURVEY	24+64.81	28+39.03	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER ALONG 61ST ST	D	LEVEE (RB)	RELOCATE	24+64.70	27+38.47	25+74.35	N/A	CABLE	+30	-445.84 +446.12	(2) POWER POLE	OHE-06A LONG SPAN (450')
CHAR-04	CHAR-03	CU103	COMMUNICATION	CHARTER	ASBUILT	24+65.56	27+38.77	25+74.35	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ROUTE CROSSING CREEK AT 61ST ST	D	LEVEE (RB)	RELOCATE	24+64.70	27+38.47	25+74.35	N/A	FIBER / CABLE	+20	-445.84 +446.12	SEE OHE-06A	CHAR-04A
GF-04	GF-05	CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	24+65.56	27+38.77	25+74.35	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ROUTE CROSSING 61ST ST	D	LEVEE (RB)	RELOCATE	24+64.70	27+38.47	25+74.35	N/A	FIBER / CABLE	+20	-445.84 +446.12	SEE OHE-06A	GF-04A
WL-07		CU103	WATER	WATERONE	BOTH	26+00.49	28+65.72	NONE	RIGHT	6	PVCC9	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG MERRIAM LN AND 61ST ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-04.1	GF-05	CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	27+38.47	28+58.25	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO SAFEGUARD SEALCOATING	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-08.1		CU103 - CU104	COMMUNICATION	AT&T	ASBUILT	28+01.52	30+39.39	28+24.26	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO SJ ELECTRIC	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-04.2	GF-05	CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	28+75.54	29+56.53	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO SJ ELECTRIC	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-04.3	GF-05	CU103	COMMUNICATION	GOOGLE FIBER	ASBUILT	28+75.54	30+07.35	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MICKEY AUTOMOTIVE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-09	ATT-06	CU103 - CU104	COMMUNICATION	AT&T	ASBUILT	29+08.01	30+07.63	30+07.63	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	PRIMARY ALONG CAMPBELL LN	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-09.1	ATT-06	CU103 - CU104	COMMUNICATION	AT&T	ASBUILT	29+08.01	30+07.63	30+07.63	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO END OF CAMPBELL ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-05.1		CU103 - CU104	COMMUNICATION	GOOGLE FIBER	ASBUILT	29+32.50	30+14.84	30+14.84	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO END OF CAMPBELL ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
OHE-07.1		CU103 - CU104	ELECTRIC	EVERGY	SURVEY	29+53.03	30+11.67	30+11.67	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE AT END OF CAMPBELL ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-08	SS-09	CU103 - CU104	SANITARY	JOCO WASTEWATER	BOTH	29+54.36	31+50.97	NONE	LEFT	8	VCP	4'-6" / 905-930	UNDERGROUND	PRIMARY FROM 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-07.1		CU104	GAS	KANSAS GAS SERVICE	ASBUILT	30+02.29	30+51.29	30+51.29	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO 6028 CAMPBELL ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
WL-08	WL-10 & WL-16	CU104 - CU107	WATER	WATERONE	BOTH	30+09.01	66+85.88	63+84.25	BOTH	6-8	CAS/DIP	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG MERRIAM DR	D	WALL (LB) & WALL (RB)	RELOCATE	62+18.68	65+38.76	64+01.54	8	CAS	-12	-50.15 /214.40 +329.01	(1) VALVE (1) TEE (3) 90 BENDS	WL-08A APPROX. 220' OF ENCASEMENT
WL-09	WL-08	CU104 - CU105	WATER	WATERONE	BOTH	30+09.49	42+08.65	32+34.95	LEFT	4-6-8	DIP	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG CAMPBELL ST	D	WALL (LB)	RELOCATE	38+63.15	39+51.19	39+14.67	6	DIP	-8	-30.18 +30.18	(1) VALVE	WL-09A
OHE-07	OHE-07	CU104	ELECTRIC	EVERGY	SURVEY	30+11.67	34+50.82	32+09.38	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER DOWN CAMPBELL ST FROM 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-07	GAS-08	CU104	GAS	KANSAS GAS SERVICE	BOTH	30+12.78	34+51.97	30+12.78	LEFT	2	PE	UNKNOWN	UNDERGROUND	PRIMARY DOWN CAMPBELL ST FROM 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-05.2		CU104	COMMUNICATION	GOOGLE FIBER	ASBUILT	30+14.84	30+60.11	30+14.84	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO END OF CAMPBELL ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
CHAR-05	CHAR-04	CU104	COMMUNICATION	CHARTER	ASBUILT	30+15.87	35+32.87	30+15.87	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY DOWN CAMPBELL ST FROM 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-05	GF-06	CU104	COMMUNICATION	GOOGLE FIBER	ASBUILT	30+15.87	33+97.16	30+15.87	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY DOWN CAMPBELL ST FROM 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-06	GF-07	CU104 - CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	31+41.60	47+28.65	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE ALONG MERRIAM DR, CROSSES UTC SOUTH OF JOHNSON DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	39+85.10	44+34.83	MULTIPLE	N/A	FIBER / CABLE	+20	-758.23 +744.58	SEE OHE-08A	GF-06A
ATT-08.2		CU104	COMMUNICATION	AT&T	ASBUILT	32+92.39	33+53.78	33+09.29	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO QXO	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-08.3		CU104	COMMUNICATION	AT&T	ASBUILT	33+41.61	37+97.49	37+97.49	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE ALONG MERRIAM DR, ENDS SOUTH OF JOHNSON DR BRIDGE	D	WALL (RB)	REMOVE	37+66.23	37+97.49	N/A	N/A	N/A	N/A	-55.99	NONE	
WL-10	WL-09	CU104	WATER	WATERONE	BOTH	34+09.24	34+15.66	NONE	LEFT	8	CAS	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-09	UGE-10	CU104 - CU105	ELECTRIC	CITY OF MERRIAM	SURVEY	34+13.34	41+90.24	PARALLEL	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG MERRIAM DR SOUTH OF JOHNSON DR	D	WALL (RB)	REMOVE	38+08.91	41+90.24	N/A	N/A	N/A	N/A	-406	NONE	
UGE-08	UGE-09	CU104 - CU105	ELECTRIC	CITY OF MERRIAM	SURVEY	34+15.77	42+03.85	PARALLEL	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG CAMPBELL ST	D	WALL (LB)	REMOVE	39+08.43	42+03.85	N/A	N/A	N/A	N/A	-251.11	NONE	
GAS-08	GAS-09	CU104	GAS	KANSAS GAS SERVICE	ASBUILT	34+21.93	34+51.97	34+51.97	LEFT	2	PE	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG 60TH ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-06.1		CU104	COMMUNICATION	GOOGLE FIBER	ASBUILT	34+50.34	35+59.90	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MERRIAM BODY SHOP	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-09	GAS-08	CU104	GAS	KANSAS GAS SERVICE	SURVEY	34+51.97	39+04.73	NONE	LEFT	2	PE	UNKNOWN	UNDERGROUND	PRIMARY UP CAMPBELL ST FROM 60TH ST	D	WALL (LB)	RELOCATE	38+54.99	38+56.77	N/A	N/A	N/A	N/A	-22.81 +9.68	(1) METER (1) 90 BEND	GAS-09A
SS-09	SS-10	CU104	SANITARY	JOCO WASTEWATER	GIS	35+10.19	35+91.88	NONE	LEFT	8	VCP	11" / 902	UNDERGROUND	PRIMARY FROM 59TH TERRACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-06.2		CU104	COMMUNICATION	GOOGLE FIBER	ASBUILT	35+59.90	35+86.08	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO OSIPIK AUTOMOTIVE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-11	SS-12	CU104	SANITARY	JOCO WASTEWATER	SURVEY	37+97.20	38+77.91	NONE	LEFT	8	VCP	12" / 903-926	UNDERGROUND	SERVICE FROM MERRIAM CHRISTIAN CHURCH	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
WL-11	WL-11	CU104	WATER	WATERONE	BOTH	38+27.67	39+03.47	38+80.92	BOTH	8	DIP	UNKNOWN	UNDERGROUND	CROSSING UTC SOUTH OF JOHNSON DR BRIDGE	D	WALL (RB)	RELOCATE	38+27.67	39+03.47	38+80.92	8	DIP	-8	-45.44 +45.44	(1) VALVE	WL-11A
CHAR-06.1		CU104	COMMUNICATION	CHARTER	ASBUILT	38+29.61	39+28.73	39+28.73	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	BOTH	SERVICE TO MERRIAM CHRISTIAN CHURCH	D	WALL (LB)	RELOCATE	39+28.73	39+96.25	39+79.32	N/A	FIBER / CABLE	+20	-108.73 +93.45	SEE OHE-08A	CHAR-06.1A

APPENDIX C.2 NON-STORM UTILITY MATRIX

UTILITY TABLE

EXISTING															PROPOSED										
ID	HNTB ID	PLAN SHEET	UTILITY	OWNER	GIS OR SURVEY	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	LEFT/RIGHT BANK	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	OVERHEAD / UNDERGROUND	DESCRIPTION	SUE QUALITY LEVEL	CONFLICT	PROPOSED ACTION	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	LINEAR LENGTH	STRUCTURES	NEW SYSTEM / NOTES
SS-06	SS-08 & SS-13	CU104 - CU105	SANITARY	JOCO WASTEWATER	BOTH	38+40.61	42+51.26	41+87.82	RIGHT	8	VCP	12'-18' / 890-899	UNDERGROUND	PRIMARY ALONG MERRIAM DR, CROSSES NORTH OF JOHNSON DR BRIDGE	D	WALL (RB)	RELOCATE	38+40.61	42+51.26	41+87.82	8 & 18	PVC	12'-18' / 890-899	-325.45 /82.33 +326.67 (8") +104.84 (18")	(4) MANHOLES	SS-06A
SS-14	SS-15	CU104 - CU105	SANITARY	JOCO WASTEWATER	BOTH	38+40.61	43+54.72	NONE	LEFT	8	VCP	18' / 905	UNDERGROUND	PRIMARY ALONG WEST JOHNSON DR	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-16	SS-17	CU104 - CU107	SANITARY	JOCO WASTEWATER	BOTH	38+40.61	67+45.00	PARALLEL	LEFT	8	PVC	20' / 890	UNDERGROUND	PRIMARY ALONG GRANT ST AND WEST UTC	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-10	SS-11	CU104	SANITARY	JOCO WASTEWATER	BOTH	38+57.71	38+69.77	NONE	RIGHT	8	VCP	12' / 899	UNDERGROUND	SERVICE FROM EAST OF MERRIAM DR, SOUTH OF JOHNSON DR	D	WALL (RB)	RECONNECT	38+60.83	38+69.77	N/A	8	PVC	12' / 899	-11.43	SEE SS-06A	
SS-11.1	SS-12	CU104	SANITARY	JOCO WASTEWATER	SURVEY	38+65.04	39+02.86	NONE	LEFT	8	VCP	UNKNOWN	UNDERGROUND	COMES INTO SS-11 FROM NORTH?	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
OHE-08.1	OHE-08	CU104 - CU105	ELECTRIC	EVERGY	SURVEY	39+28.73	47+27.79	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO MERRIAM CHRISTIAN CHURCH	D	WALL (LB)	RELOCATE	39+28.73	39+96.25	39+79.32	N/A	CABLE	+30	-109.85 +91.43	SEE OHE-08A	OHE-08.1A
GF-06.3		CU104	COMMUNICATION	GOOGLE FIBER	ASBUILT	39+28.73	39+98.28	39+98.28	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO MERRIAM CHRISTIAN CHURCH	D	WALL (LB)	RELOCATE	39+28.73	39+96.25	39+79.32	N/A	FIBER / CABLE	+20	-109.85 +91.43	SEE OHE-08A	GF-06.3A
OHE-08.2		CU104 - CU105	ELECTRIC	EVERGY	SURVEY	39+85.06	40+52.13	40+52.13	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO TRAFFIC LIGHTS AT JOHNSON DR BRIDGE	D	WALL (RB)	RELOCATE	40+72.06	39+85.83	N/A	N/A	CABLE	+15	-103.72 +113.33	SEE OHE-08A	OHE-08.2A
CHAR-06	CHAR-05	CU104 - CU105	COMMUNICATION	CHARTER	ASBUILT	39+85.10	47+28.65	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE ALONG MERRIAM DR, CROSSES UTC SOUTH OF JOHNSON DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	39+85.10	44+34.83	MULTIPLE	N/A	FIBER / CABLE	+20	-758.23 +744.58	SEE OHE-08A	CHAR-06A
OHE-08	OHE-08	CU104	ELECTRIC	EVERGY	SURVEY	39+85.83	39+98.45	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE ALONG MERRIAM DR, CROSSES UTC SOUTH OF JOHNSON DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	39+85.10	44+34.83	MULTIPLE	N/A	FIBER / CABLE	30	-758.23 +744.58	(5) POWER POLE	OHE-08A
GAS-10	GAS-11?	CU105	GAS	KANSAS GAS SERVICE	SURVEY	40+34.11	42+14.03	40+50.22	BOTH	8	ACPS	3'-5'	UNDERGROUND	CROSSING UNDER UTC SOUTH OF JOHNSON DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	40+48.44	41+09.61	40+70.02	8	APCS OR PE?	-10	-63.66 /150.72 +196.66	(1) 22.5 BEND	GAS-10A HORIZONTAL BORE ALONG JOHNSON DR
SS-13	SS-13	CU105	SANITARY	JOCO WASTEWATER	SURVEY	40+50.93	40+99.21	NONE	RIGHT	24	VCP	18'-19' / 896-899	UNDERGROUND	PRIMARY ALONG EAST JOHNSON DR	D	WALL (RB)	RECONNECT	40+50.93	40+98.35	N/A	24	PVC	18'-19' / 896-899	/17.89	SEE SS-06A	
UGE-10	UGE-13	CU105	ELECTRIC	CITY OF MERRIAM	SURVEY	40+62.16	40+69.74	40+69.74	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG SOUTH SIDE OF JOHNSON DR	D	WALL (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-10	ATT-07	CU105	COMMUNICATION	AT&T	ASBUILT	40+77.12	41+46.53	42+11.37	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	LOCATED ON JOHNSON DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	40+77.12	41+46.53	42+11.37	N/A	FIBER / CABLE	-3	0	NONE	ATT-10A AWAITING FINAL JOHNSON DR BRIDGE ALIGNMENT
WL-12	WL-12	CU105	WATER	WATERONE	GIS	40+78.06	40+95.50	NONE	RIGHT	8	DIP	UNKNOWN	UNDERGROUND	PRIMARY ALONG JOHNSON DR EAST OF BRIDGE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
UGE-11	UGE-14	CU105	ELECTRIC	CITY OF MERRIAM	SURVEY	40+90.93	41+56.69	41+56.69	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG NORTH SIDE OF JOHNSON DR	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
OHE-09	OHE-09	CU105	ELECTRIC	EVERGY	SURVEY	41+00.28	42+83.50	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER ALONG JOHNSON DR BRIDGE, CROSSING ON NORTH SIDE OF BRIDGE	D	JOHNSON DR BRIDGE	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	CONNECT TO NEW POLE ON OHE-08A
UGE-12	UGE-16 & UGE-19	CU105 - CU107	ELECTRIC	CITY OF MERRIAM	SURVEY	41+90.67	63+18.94	62+64.37	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG MERRIAM DR BETWEEN JOHNSON DR AND MERRIAM DR BRIDGE	D	MERRIAM DR BRIDGE	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	UGE-12A AWAITING FINAL MERRIAM DR BRIDGE ALIGNMENT
GAS-11	GAS-13	CU105 - CU107	GAS	KANSAS GAS SERVICE	ASBUILT	42+12.00	60+42.42	NONE	BASIN	2	PE	UNKNOWN	UNDERGROUND	PRIMARY ROUTE ALONG MERRIAM DR NORTH OF JOHNSON DR	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-15	SS-15	CU105	SANITARY	JOCO WASTEWATER	BOTH	42+27.55	42+51.26	NONE	RIGHT	8	VCP	10' / 900	UNDERGROUND	SERVICE FROM PET CARE	D	WALL (RB)	RELOCATE	41+82.45	42+51.26	41+87.70	8	PVC	10' / 900	-255.71 /100.12 +201.01	(2) MANHOLES	SS-15A
WL-13	WL-14	CU105	WATER	WATERONE	BOTH	42+35.28	44+07.93	42+87.92	RIGHT	1	COPPER	UNKNOWN	UNDERGROUND	SERVICE LINE BEHIND KC MUSIC STORES	D	WALL (RB)	RELOCATE	42+74.61	43+52.29	N/A	1	COPPER	-3.5	-108.86 +88.81	(1) 90 BEND (1) 30 BEND	WL-13A
GF-06.4		CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	42+44.63	43+27.36	42+44.63	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO PET CARE	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-124.85	NONE	
OHE-08.1		CU105	ELECTRIC	EVERGY	SURVEY	42+51.01	43+27.95	43+27.95	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO PET CARE	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-57.1	NONE	
ATT-11	ATT-09	CU105	COMMUNICATION	AT&T	ASBUILT	42+51.62	44+33.90	43+26.80	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE BEHIND KC MUSIC STORES	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-277.95	NONE	
GF-06.5	GF-13?	CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	42+93.11	43+27.36	43+27.36	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO KC MUSIC STORES	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-67.68	NONE	
OHE-08.2		CU105	ELECTRIC	EVERGY	SURVEY	43+08.75	43+27.89	43+27.89	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO KC STRINGS	D	WALL (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-57.1	NONE	
OHE-08.3		CU105	ELECTRIC	EVERGY	SURVEY	43+26.12	43+27.39	43+26.12	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO KC MUSIC STORES	D	WALL (RB)	RELOCATE	43+26.13	43+35.80	N/A	N/A	CABLE	+20	-57.10 +41.32	SEE OHE-08A	OHE-08.3A
OHE-08.4		CU105	ELECTRIC	EVERGY	SURVEY	43+65.71	44+28.52	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO KC MUSIC STORES	D	WALL (RB)	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	
GF-06.6		CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	43+67.60	44+34.83	44+34.83	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO KC MUSIC STORES	D	WALL (RB)	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	
GF-06.7		CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	44+04.13	44+34.83	44+34.83	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO KC MUSIC STORES	D	WALL (RB)	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	
GF-06.8		CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	44+34.83	45+27.85	44+34.83	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO NATIONWIDE AUTO	D	WALL (RB)	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	
OHE-08.5		CU105	ELECTRIC	EVERGY	SURVEY	44+34.83	44+34.83	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO KC FRET SHOP	D	WALL (RB)	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	
OHE-08.6		CU105	ELECTRIC	EVERGY	SURVEY	44+35.57	45+30.31	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO NATIONWIDE AUTO	D	WALL (RB)	RECONNECT	N/A	N/A	N/A	N/A	N/A	N/A	0	SEE OHE-08A	
SS-15.1	SS-16	CU105	SANITARY	JOCO WASTEWATER	GIS	45+00.00	45+18.46	45+15.20	RIGHT	8	VCP	13' / 896	UNDERGROUND	SERVICE FROM KC MUSIC SHOPS	D	WALL (RB)	RECONNECT	45+00.00	45+11.36	N/A	8	PVC	13' / 896	-97.55	SEE SS-04A	
OHE-08.7		CU105	ELECTRIC	PRIVATE	SURVEY	45+50.00	46+23.64	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO STREETLIGHT	D	LEVEE (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-81.57	NONE	
GF-06.9		CU105	COMMUNICATION	GOOGLE FIBER	ASBUILT	46+13.09	46+64.12	46+64.12	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO ALL OUT TINT AND WRAP	D	LEVEE (RB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-113.5	NONE	
OHE-08.8		CU105	ELECTRIC	EVERGY	SURVEY	46+13.39	46+24.13	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO ANITA'S CUISINE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
WL-14	WL-15	CU105	WATER	WATERONE	BOTH	46+24.66	47+26.22	47+26.22	RIGHT	2	CAS	UNKNOWN	UNDERGROUND	SERVICE LINE ALONG HOCKER DR	D	LEVEE (RB)	ABANDON	46+24.66	47+26.22	N/A	N/A	N/A	N/A	/110.01	NONE	
GAS-12	GAS-14	CU105	GAS	KANSAS GAS SERVICE	ASBUILT	46+36.32	46+85.65	NONE	RIGHT	2	PE	UNKNOWN	UNDERGROUND	PRIMARY ROUTE TO ALL OUT TINT AND WRAP	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-06.10		CU105 - CU106	COMMUNICATION	GOOGLE FIBER	ASBUILT	47+26.93	50+87.42	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MERRIAM MARKETPLACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-18	SS-19	CU105 - CU106	SANITARY	JOCO WASTEWATER	BOTH	47+32.43	51+02.52	NONE	RIGHT	8	VCP	12'-15' / 894-898	UNDERGROUND	SERVICE FROM MERRIAM MARKETPLACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-17	SS-18	CU105	SANITARY	JOCO WASTEWATER	BOTH	47+32.43	47+60.16	NONE	RIGHT	8	VCP	15' / 894	UNDERGROUND	PRIMARY FROM 58TH ST	D	LEVEE (RB)	RECONNECT	47+32.43	47+53.72	N/A	8	PVC	15' / 894	/45.04	SEE SS-04A	
ATT-12		CU106	COMMUNICATION	AT&T	ASBUILT	50+84.21	56+94.45	53+91.88	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SOUTH OF DETENTION BASIN ALONG MERRIAM DR	D	DETENTION BASIN	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-12.1		CU106	COMMUNICATION	AT&T	ASBUILT	52+53.88	52+56.35	NONE	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MERRIAM MARKETPLACE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
OHE-10	OHE-10 & OHE-11	CU107	ELECTRIC	EVERGY	SURVEY	56+27.28	65+66.97	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY POWER ALONG WEST SIDE OF MERRIAM DR SOUTH OF BRIDGE	D	DETENTION BASIN	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-11.1		CU106	GAS	KANSAS GAS SERVICE	ASBUILT	56+46.67	56+87.68	56+46.67	BASIN	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO RIGDON 360	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-60.69	NONE	
GF-07.1		CU106	COMMUNICATION	GOOGLE FIBER	ASBUILT	56+63.46	57+14.48	56+63.46	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO RIGDON 360	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-52.88	NONE	
GF-07.2		CU106	COMMUNICATION	GOOGLE FIBER	ASBUILT	56+74.10	57+14.48	56+74.10	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO BAKER'S MARBLE	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-117.18	NONE	
SS-19.1	SS-20	CU106	SANITARY	JOCO WASTEWATER	GIS	56+93.55	58+13.31	NONE	RIGHT	8	VCP	10' / 895	UNDERGROUND	SERVICE FROM BAKER'S MARBLE	D	DETENTION BASIN	ABANDON	ALL	ALL	N/A	N/A	N/A	N/A	/127.85	NONE	
GAS-11.2		CU106	GAS	KANSAS GAS SERVICE	ASBUILT	56+96.46	57+41.96	56+96.46	BASIN	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO BAKER'S MARBLE	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-67.2	NONE	
ATT-13	ATT-10 & ATT-11	CU106 - CU107	COMMUNICATION	AT&T	ASBUILT	57+04.36	66+89.08	63+94.49	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	NORTH OF DETENTION BASIN ALONG MEERIAM DR, SERVICES BUS DEPOT	D	DETENTION BASIN & WALL (RB)	RELOCATE	62+33.42	64+58.91	N/A	N/A	FIBER / CABLE	15	-57.49 +325.17	NONE	ATT-13A
CHAR-07		CU106 - CU107	COMMUNICATION	CHARTER	ASBUILT	57+14.48	62+35.06	NONE	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ALONG MERRIAM DR SOUTH OF BRIDGE	D	DETENTION BASIN	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	

APPENDIX C.2 NON-STORM UTILITY MATRIX

UTILITY TABLE

EXISTING															PROPOSED										
ID	HNTB ID	PLAN SHEET	UTILITY	OWNER	GIS OR SURVEY	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	LEFT/RIGHT BANK	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	OVERHEAD / UNDERGROUND	DESCRIPTION	SUE QUALITY LEVEL	CONFLICT	PROPOSED ACTION	UTC BASELINE START STATION	UTC BASELINE END STATION	UTC BASELINE STATION CROSSING	SIZE AT CROSSING (IN)	MATERIAL	DEPTH / ELEVATION	LINEAR LENGTH	STRUCTURES	NEW SYSTEM / NOTES
GF-07	GF-10	CU106 - CU107	COMMUNICATION	GOOGLE FIBER	ASBUILT	57+14.48	62+35.06	NONE	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY ALONG MERRIAM DR SOUTH OF BRIDGE	D	DETENTION BASIN	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-13.1		CU106	COMMUNICATION	AT&T	ASBUILT	57+25.94	57+82.21	57+25.94	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO BAKER'S MARBLE	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-149.5	NONE	
SS-19	SS-20	CU106	SANITARY	JOCO WASTEWATER	BOTH	57+74.59	59+74.05	57+83.84	RIGHT	8	VCP	14'-16' / 894-896	UNDERGROUND	SERVICE FROM 5700 MERRIAM DR	D	DETENTION BASIN	RELOCATE	57+86.62	59+74.05	58+52.24	8	PVC	14'-16' / 894-896	/149.87 +194.31	(1) MANHOLE	SS-19A
GF-07.3		CU106	COMMUNICATION	GOOGLE FIBER	ASBUILT	57+78.36	59+01.77	57+78.36	BASIN	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO 5700 MERRIAM DR	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-129.74	NONE	
GAS-11.3		CU106	GAS	KANSAS GAS SERVICE	ASBUILT	58+17.42	58+62.54	58+17.42	BASIN	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO 5700 MERRIAM DR	D	DETENTION BASIN	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-66.57	NONE	
GF-07.4		CU106	COMMUNICATION	GOOGLE FIBER	ASBUILT	58+28.69	59+01.77	58+28.69	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MIDWESTERN MECHANICAL SPECS	D	WALL (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-07.5		CU106	COMMUNICATION	GOOGLE FIBER	ASBUILT	59+01.77	59+01.77	59+01.77	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MERRIAM HARDWARE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-13	GAS-16	CU106 - CU107	GAS	KANSAS GAS SERVICE	BOTH	59+06.98	66+91.91	63+84.25	BOTH	4	APCS	UNKNOWN	UNDERGROUND	PRIMARY ROUTE CROSSING MERRIAM DR BRIDGE	D	WALL (LB) & WALL (RB)	REPLACE	62+78.74	63+56.21	63+70.45	4	APCS OR PE?	-8	-91.03 +91.03	NONE	GAS-13A HORIZONTAL BORE ALONG MERRIAM DR
CHAR-07.1	CHAR-06	CU106 - CU107	COMMUNICATION	CHARTER	ASBUILT	59+21.86	62+36.55	62+36.55	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	ALONG MERRIAM DR SOUTH OF MERRIAM DR BRIDGE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-13.1		CU106	GAS	KANSAS GAS SERVICE	SURVEY	59+63.43	60+25.31	NONE	BASIN	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO MEDICAL SOLUTIONS	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-07.6		CU106 - CU107	COMMUNICATION	GOOGLE FIBER	ASBUILT	59+83.02	60+52.12	59+83.02	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO MEDICAL SOLUTIONS	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-20	SS-21	CU106	SANITARY	JOCO WASTEWATER	GIS	61+19.69	61+86.08	61+86.08	LEFT	8	VCP	7' / 900-933	UNDERGROUND	PRIMARY FROM HAYES ST	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-15		CU107	GAS	KANSAS GAS SERVICE	ASBUILT	61+89.91	63+68.29	NONE	LEFT	1	PE	UNKNOWN	UNDERGROUND	PRIMARY ROUTE TO SRA ROOFING AND GUTTERS	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GF-08	GF-10	CU107	COMMUNICATION	GOOGLE FIBER	ASBUILT	62+33.11	63+60.21	63+77.29	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY CROSSING MERRIAM DR BRIDGE	D	WALL (LB) & WALL (RB)	RECONNECT	62+37.15	63+60.21	63+77.29	N/A	FIBER / CABLE	+20	-294.53 +294.32	SEE OHE-11A	GF-08A
OHE-11	OHE-10	CU107	ELECTRIC	EVERGY	SURVEY	62+34.49	67+42.54	MULTIPLE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY CROSSING MERRIAM DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	62+37.15	63+60.21	63+77.29	N/A	CABLE	+30	-294.53 +294.32	(2) POWER POLE	OHE-11A
UGE-13	UGE-20	CU107	ELECTRIC	CITY OF MERRIAM	SURVEY	62+35.80	63+56.02	MULTIPLE	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG EAST SIDE OF MERRIAM BRIDGE	D	MERRIAM DR BRIDGE	RELOCATE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	UGE-13A AWAITING FINAL MERRIAM DR BRIDGE ALIGNMENT
CHAR-08	CHAR-06	CU107	COMMUNICATION	CHARTER	ASBUILT	62+37.15	63+60.21	63+77.29	BOTH	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	PRIMARY CROSSING MERRIAM DR BRIDGE	D	WALL (LB) & WALL (RB)	RELOCATE	62+37.15	63+60.21	63+77.29	N/A	FIBER / CABLE	+20	-294.53 +294.32	SEE OHE-11A	CHAR-08A
CHAR-08.1		CU107	COMMUNICATION	CHARTER	ASBUILT	62+59.10	63+74.57	62+59.10	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO SRA ROOFING AND GUTTERS	D	LEVEE (LB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-214.26	NONE	
GF-08.1		CU107	COMMUNICATION	GOOGLE FIBER	ASBUILT	62+59.10	63+74.57	62+59.10	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO SRA ROOFING AND GUTTERS	D	LEVEE (LB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-188.99	NONE	
OHE-11.1	OHE-10	CU107	ELECTRIC	EVERGY	SURVEY	62+59.10	63+74.57	62+59.10	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO SRA ROOFING AND GUTTERS	D	LEVEE (LB)	REMOVE	ALL	ALL	N/A	N/A	N/A	N/A	-188.99	NONE	
GAS-15.1		CU107	GAS	KANSAS GAS SERVICE	BOTH	62+96.17	62+98.91	NONE	LEFT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO SRA ROOFING AND GUTTERS	D	LEVEE (LB)	ABANDON	62+96.17	62+98.91	N/A	N/A	N/A	N/A	/39.02	NONE	
WL-16	WL-16	CU107	WATER	WATERONE	BOTH	63+03.31	63+49.32	NONE	LEFT	1	COPPER	UNKNOWN	UNDERGROUND	SERVICE TO SRA ROOFING AND GUTTERS	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-21	SS-22	CU107	SANITARY	JOCO WASTEWATER	BOTH	63+09.17	63+28.17	63+09.17	LEFT	8	PVC	10' / 897	UNDERGROUND	PRIMARY ALONG MERRIAM DR, NORTH OF BRIDGE	D	WALL (LB)	RECONNECT	63+09.17	63+28.17	N/A	8	PVC	10' / 897	/36.46	SEE SS-04A	
WL-15	WL-16	CU107	WATER	WATERONE	ASBUILT	63+12.23	66+85.62	63+12.23	RIGHT	1	COPPER	UNKNOWN	UNDERGROUND	SERVICE TO BUS DEPOT	D	WALL (RB)	RELOCATE	64+04.34	66+85.62	N/A	1	COPPER	-3.5	-331.47 /178.36 +492.38	(2) 22.5 BEND (1) 45 BEND	WL-15A
UGE-14	UGE-21	CU107	ELECTRIC	CITY OF MERRIAM	SURVEY	63+24.44	63+29.16	NONE	LEFT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	STREETLIGHT POWER ALONG WEST SIDE OF MERRIAM DR NORTH OF BRIDGE	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-14	GAS-17	CU107	GAS	KANSAS GAS SERVICE	ASBUILT	63+73.73	65+90.18	63+73.73	RIGHT	2	APCS	UNKNOWN	UNDERGROUND	PRIMARY ROUTE TO BUS DEPOT	D	WALL (RB)	RELOCATE	63+73.99	64+21.06	N/A	2	APCS OR PE?	-3	-68.33 +87.45	NONE	GAS-14A
ATT-13.2		CU107	COMMUNICATION	AT&T	ASBUILT	63+81.66	66+89.08	63+94.49	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICES BUS DEPOT	D	WALL (RB)	RELOCATE	62+33.42	64+58.91	N/A	N/A	FIBER / CABLE	-3	-142.53 /185.38 +66.54	NONE	ATT-13.2A
SS-22	SS-23	CU107	SANITARY	JOCO WASTEWATER	BOTH	64+36.17	65+83.09	65+37.73	BOTH	8	VCP	20' / 888	UNDERGROUND	PRIMARY FROM BUS DEPOT	D	WALL (LB) & WALL (RB)	RELOCATE	64+36.17	65+83.09	65+37.73	8	PVC	20' / 888	-140.18 +140.12	(1) MANHOLE	SS-22A
GF-08.2		CU107	COMMUNICATION	GOOGLE FIBER	ASBUILT	64+70.88	64+74.59	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	SERVICE TO LINDAN AUTO	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-22.1	SS-23	CU107	SANITARY	JOCO WASTEWATER	GIS	65+18.08	66+00.00	NONE	RIGHT	8	VCP	7'-10' / 900-904	UNDERGROUND	SERVICE 1 FROM BUS DEPOT	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
SS-22.2	SS-23	CU107	SANITARY	JOCO WASTEWATER	BOTH	65+38.61	67+45.00	PARALLEL	RIGHT	8	PVC	9' / 889	UNDERGROUND	SERVICE 2 FROM BUS DEPOT	D	WALL (RB)	RELOCATE	65+32.54	66+60.81	N/A	8	PVC	9' / 889	-149.14 /40.31 +200.51	(2) MANHOLES	SS-22.2A
OHE-10.1	OHE-12	CU107	ELECTRIC	EVERGY	SURVEY	65+66.96	66+06.71	NONE	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	OVERHEAD	SERVICE TO BUS DEPOT	D	WALL (RB)	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
GAS-14.1		CU107	GAS	KANSAS GAS SERVICE	ASBUILT	65+79.83	66+88.93	NONE	RIGHT	1	PE	UNKNOWN	UNDERGROUND	SERVICE TO BUS DEPOT	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	
ATT-14		CU107	COMMUNICATION	AT&T	ASBUILT	65+88.85	66+76.34	66+76.34	RIGHT	UNKNOWN	UNKNOWN	UNKNOWN	UNDERGROUND	NORTH OF BUS DEPOT IN UTC?	D	NONE	LEAVE IN PLACE	N/A	N/A	N/A	N/A	N/A	N/A	0	NONE	

APPENDIX C.3 – TOE DRAIN EXAMPLE CALCULATION

US Army Corps of Engineers
Kansas City District

PROJECT Upper Turkey Creek

TASK Toe Drain Pipe Design

Toe Drain A

DATE: 25-Mar-26
INITIALS: FNP
REVIEW DATE:
REVIEWER:

	Q	Q, cumulative	n	Length	Dia	Ground elevation	Discharge elevation	IE in	IE out	Slope	Flow Depth	Theta	P_w	A	Q_{calc}	V	*Flow Depth as percent of Diameter*	*Depth of Manhole Required*
	cfs	*cfs*		*ft*	*in*	*ft*	*ft*	*ft*	*ft*	*ft/ft*	*in*		*in*	ft^2	*cfs*	*ft/s*	*%*	*ft*
Start	0.0053026	0.00530264	0.011	155.96	15	931.67				0.80%	0.33	-1.2723	0.37	0.01	0.006	0.8	*2%*	
Manhole A-1	0.0064287	0.01173136	0.011	189.08	15	930.59				0.80%	0.47	-1.2139	0.45	0.01	0.012	1.1	*3%*	
Manhole A-2	0.0008905	0.01262182	0.011	26.19	15	928.63				0.80%	0.48	-1.2092	0.45	0.01	0.013	1.1	*3%*	
Standpipe A-3	0.0052013	0.01959828	0.011	152.98	15	928.39	928.89			0.60%	0.64	-1.1551	0.52	0.02	0.020	1.1	*4%*	
Manhole A-4	0.0017751	0.00177514	0.011	52.21	15	929.49				0.60%	0.24	-1.3152	0.32	0.00	0.003	0.6	*2%*	
End						929.74												

Re # Through Pipe Perforations (AASHTO Class II Perforation):

Q	0.000034	cfs/ft	
A	0.0069444	ft2/ft	(Min Inlet Area 1.5 in2/ft)
Vel=	0.005	ft/s	
Hyd. Dia=	0.018	ft	(Max Slot Dia 0.236 in, 6mm)
kin. Visc=	0.000012	ft2/s	
Re=	7.2945455	laminar	

Reynolds Number Check

$$Re = \frac{V \times D_h}{\upsilon}$$

Manning eq -> $Q = 1.49*A*R^{2/3}*S^{1/2}/n$

R = A/P_w
R = Hydraulic Radius
A = Flow Area
y = Flow Depth
P_w = Wetted Perimeter
S = Pipe Slope

Rational Method -> $Q = CiA$

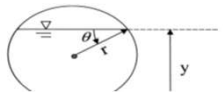
Q = Runoff (cfs)
C = Runoff Coefficient = 0.3+0.6*Impervious Area (APWA Page 18 Sec 5600)
i = Rainfall Intensity (in/hr)
A = Area (ac)

NOTES & ASSUMPTIONS

Flow depth cell will turn red if depth exceeds 85% of pipe diameter.
Qcalc cell will turn red if Qcalc is not within 5% of Q.
Spreadsheet does not account for tail water effects.
Header pipe at first manhole is 1.5 ft below discharge elevation

Pipe Geometry

$A = r^2(\pi/2+\theta+\sin\theta\cos\theta)$
$B = 2r\cos\theta$
$\theta = \sin^{-1}(y-r)/r$



APPENDIX D – STRUCTURAL

D.1 Floodwall Loading Justification Calculations
D.2 14 Ft Floodwall Calculations
D.3 12 Ft Floodwall Calculations
D.4 10 Ft Floodwall Calculations
D.5 10 Ft Floodwall Alternate Surcharge Loading Calculations
D.6 8 Ft Floodwall Calculations
D.7 6 Ft Floodwall Calculations
D.8 4 Ft Floodwall Calculations
D.9 CT Wall Output Files
D.10 I Wall Strength Connection Calculations
D.11 Outfall Structure Calculations
D.12 Quaker Creek Outfall Structure
D.13 Detention Pond Inlet Structure
D.14 Johnson Drive Bridge Report, HNTB Submittal
D.15 Merriam Drive Bridge Calculations
D.16 Johnson Drive Bridge Calculations

Calculations prepared by: John W. Westbrook, P.E., Brett Meyring, P.E.

Date: 20260311

Calculations reviewed by: Paul Muller

Date: 20260320

This page was intentionally left blank for duplex printing.

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T Wall Load Cases, Strength Design and Serviceability - EM 1110-2-2104

Upper Turkey Creek

Merriam, KS

PURPOSE

The Engineering Manual for the design of Reinforced Hydraulic Concrete Structures states that the loading conditions for the floodwalls is outlined in Engineering Manual 1110-2-2104 for Strength Design for Reinforced Concrete Hydraulic Structures. Chapter 3 of EM 1110-2-2104 states that both the strength and serviceability loading requirements of the manual must be met. This package will show that the serviceability loading requirements will govern the design of the floodwalls. The serviceability loading will be used to design the T-wall sections in following calculations in this package.

ASSUMPTIONS AND REFERENCES

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104
- For the 100 year event, the design event for the project, water elevation is assumed to the top of the floodwall and is treated as the Unusual Loading condition. for the usual case; the 10 year event, water is loaded minimally on the foundation of the wall. The hydraulic analysis shows that the 10% event is on the order of several hours in a given year based on H&H analysis and does not reach the stem of the floodwalls. Reference Hydraulic section of the DDR for additional commentary on this. For this analysis the 10 year event is assumed to not load the T-walls.

WALL LAYOUT AND GEOMETRY, SECTION AT CREEK STATION 40+24, 14 FT TALL T-WALL

Concrete Structure

$\gamma_{concrete} := 150\ pcf$ *(unit weight of concrete)*

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(saturated unit weight of soil)*

$Elev_{wall} := 920.12\ ft$ *(elevation at top of stem: ELTS)*

$H_{wall} := 14\ ft$ *(wall height measured from the top of footing: HTS)*

$t_{stem.top} := 1.5\ ft$ *(thickness at top of stem: TTS)*

$t_{stem.bot} := 1.5\ ft$ *(thickness at bottom of stem: TBS)*

$t_{stem.batter} := t_{stem.bot} - t_{stem.top} = 0\ in$ *(distance of batter for stem: TBSR)*

$Elev_{foun.top} := Elev_{wall} - H_{wall} = 906.1\ ft$ *(elevation of the top of the foundation)*

$Elev_{soil} := 908.86\ ft$ *(elevation of the soil on stem)*

$Elev_{key} := 900.62\ ft$ *(elevation of the bottom of the key)*

$Elev_{soilwedge} := 907.72\ ft$

$H_{stem} := Elev_{wall} - Elev_{soil} = 11.3\ ft$ *(exposed height of stem above ground level)*

STRENGTH DESIGN LOAD CASES

Permanent Loads

Effective Lateral Earth Pressure: Driving Side

$\rho_{soil.key} := 0.213\ \frac{kip}{ft}$ *(soil lateral loading at base of key, from CTWALL)*

$Elev_{soil} - Elev_{foun.top} = 2.7\ ft$ *(depth of maximum lateral soil loading)*

$\rho_{soil.stem.d} := \rho_{soil.key} \cdot \frac{\left(Elev_{soil} - Elev_{foun.top}\right)}{\left(Elev_{soil} - Elev_{key}\right)} = 0.071 \ \frac{kip}{ft}$ *(lateral soil loading at the top of foundation, bottom of stem)*

$EH_d := \rho_{soil.stem.d} \cdot 0.5 \cdot \left(Elev_{soil} - Elev_{foun.top}\right) = 0.097 \ kip$ *(lateral soil loading, driving side)*

$(x)_{EH.d} := \frac{1}{3} \cdot \left(Elev_{soil} - Elev_{foun.top}\right) = 0.9 \ ft$ *(resultant height above base of stem)*

Effective Lateral Earth Pressure: Resisting Side

$\rho_{soil.foun.bot} := 0.131 \ \frac{kip}{ft}$ *(soil lateral loading at bottom of foundation, from CTWALL)*

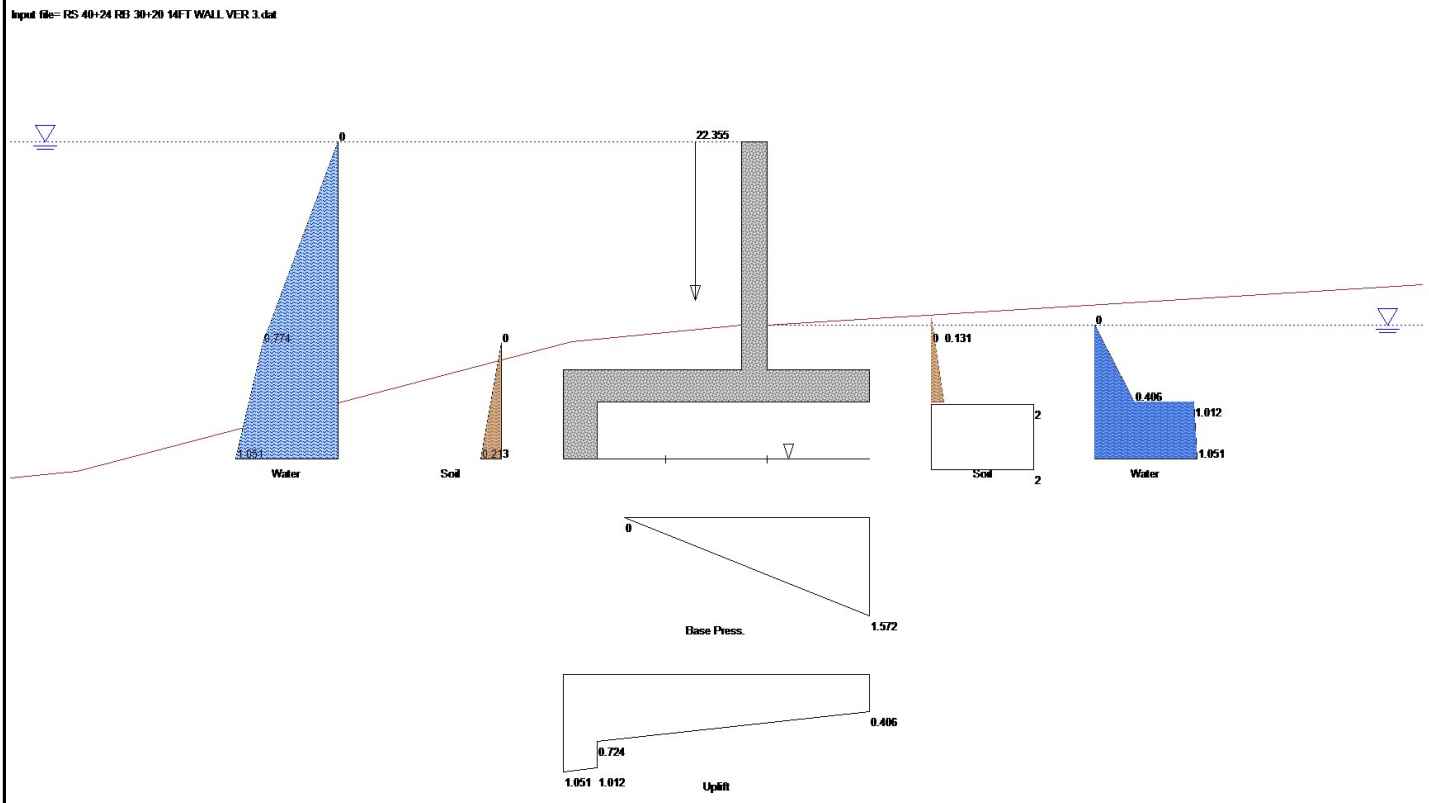
$Elev_{soil} - Elev_{foun.top} = 2.7 \ ft$ *(depth of maximum lateral soil loading)*

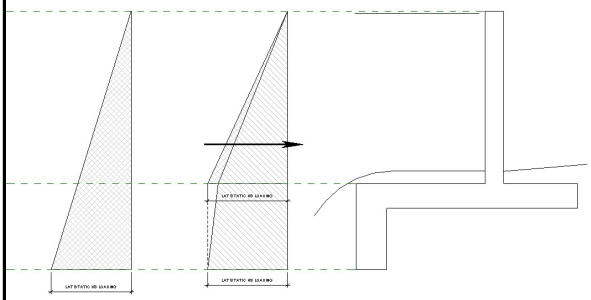
$EH_r := \rho_{soil.foun.bot} \cdot 0.5 \cdot \left(Elev_{soil} - Elev_{foun.top}\right) = 0.179 \ kip$ *(usual lateral soil loading, resisting side)*

$(x)_{EH.r} := \frac{1}{3} \left(Elev_{soil} - Elev_{foun.top}\right) = 0.9 \ ft$ *(resultant height above base of stem)*

Note: reference attached CTWALL printouts for effective lateral earth pressures to the stem of floodwalls.

Reference calculations for the derivation of the lateral static hydraulic pressure. Static hydraulic pressure is calculated assuming that the full fluid pressure at the elevation of the top of the foundation applied to the stem. This provides a conservative estimate of the resultant of the hydraulic loading to the stem without the reduction for the soil's reduction for pore pressure.





Temporary Loading

Hydrostatic Loading - Driving Side

Note: For simplicity water loading assumed to the top of the wall for the unusual case, for the usual case water assumed to not load the stem of the wall.

$\rho_{H.us} := 0.0\ \frac{kip}{ft}$ *(calculated pore pressure to driving side of stem, usual case, assumed 0)*

$\rho_{H.un} := \gamma_{water} \cdot \left(Elev_{wall} - Elev_{foun.top}\right) \cdot 1\ ft = 0.874\ \frac{kip}{ft}$ *(calculated pore pressure to driving side of stem, unusual case)*

$Hs_{us.d} := \rho_{H.us} \cdot 0.5 \cdot \frac{\left(Elev_{wall} - Elev_{foun.top}\right)}{2} = 0\ kip$ *(usual hydrostatic loading or companion load, 10 yr event)*

$(x)_{Hs.us.d} := \frac{1}{3} \cdot \frac{\left(Elev_{wall} - Elev_{foun.top}\right)}{2} = 2.3\ ft$ *(resultant height above base of stem, usual loading driving side)*

$Hs_{un.d} := \rho_{H.un} \cdot 0.5 \cdot \left(Elev_{wall} - Elev_{foun.top}\right) = 6.1\ kip$ *(unusual hydrostatic loading or primary load, 100 yr event)*

$(x)_{Hs.un.d} := \frac{1}{3} \cdot \left(Elev_{wall} - Elev_{foun.top}\right) = 4.7\ ft$ *(resultant height above base of stem, unusual loading driving side)*

Note: the extreme loading even is not considered as the floodwalls are being designed for the 100 year event which is the unusual loading condition

Hydrostatic Loading - Resisting Side

Note: The resisting side hydraulic loading is conservatively assumed to be hydrostatic from the soil surface. Increasing the pressure to include seepage would decrease the demand on the stem.

$\rho_{H.un} := \gamma_{water} \cdot \left(Elev_{wall} - Elev_{foun.top}\right) \cdot 1\ ft = 0.874\ \frac{1}{ft} \cdot kip$ *(calculated pore pressure to driving side of stem, unusual case)*

$Hs_{us.r} := \rho_{H.us} \cdot 0.5 \cdot \frac{\left(Elev_{soil} - Elev_{foun.top}\right)}{2} = 0\ kip$ *(usual hydrostatic loading or companion load, 10 yr event)*

$(x)_{Hs.us.r} := \frac{1}{3} \cdot \frac{\left(Elev_{soil} - Elev_{foun.top}\right)}{2} = 0.5\ ft$ *(resultant height above base of stem, usual loading driving side)*

$Hs_{un.r} := \rho_{H.un} \cdot 0.5 \cdot \left(Elev_{soil} - Elev_{foun.top}\right) = 1.2\ kip$ *(unusual hydrostatic loading or primary load, 100 yr event)*

$(x)_{Hs.un.r} := \frac{1}{3} \cdot \left(Elev_{soil} - Elev_{foun.top}\right) = 0.9\ ft$ *(resultant height above base of stem, unusual loading driving side)*

Note: the extreme loading even is not considered as the floodwalls are being designed for the 100 year event which is the unusual loading condition

Temporary Loads

Peak Hydrostatic Load

$Hs_{peak} := Hs_{un.d}$ *(principle hydrostatic load; maximum differential physically possible on wall)*

Expansion of Ice

Note: Ice loading not included in analysis, reference DDR documentation.

Soil Surcharge

Note: Soil surcharge loading is not included in this calculation. Loading in applicable in certain locations, but does not contribute significantly to the loading condition that governs design. For the strength vs serviceability comparison only load case 1 per Table E-3 of EM 1110-2-2104 will govern design.

Dynamic Loading

Hydrodynamic Loading

Note: Reference DDR section for Hydrodynamic loading as well as the analysis in this document. Hydrodynamic loading considerations excluded as flows are parallel to the face of the floodwall.

Wave Loading

Note: wave loading not considered as fetch lengths are too short and water velocities are perpendicular to the wall in most instances.

Debris Loading

$I_{us} := 0\ kip$ *(impact loading, EM 1110-2-2502 6.12.3)*

$I_{un} := 0.1\ kip$ *(impact loading, EM 1110-2-2502 6.12.3)*

Wind Loading

Note: Wind loading only applicable while water not loaded to the top of floodwall. For comparison of the strength and serviceability loading load case 1 with hydrostatic loading to the top of the floodwall considered for analysis.

LOAD FACTORS

Strength Design

Dead Loads

$\gamma_{p.D} := 1.2$ *(load factor - Usual dead load)*

$\gamma_{p.D\_sub} := 0.9$ *(load factor when loads subtract from predominant load effect - Extreme dead load)*

Effective Lateral Earth Loads

$\gamma_{c.EH\_driving} := 1.5$ *(load factor for at-rest pressures, EM 1110-2-2104 3-3d)*

$\gamma_{c.EH\_resisting} := 0.5$ *(load factor for at-rest pressures, EM 1110-2-2104 3-3d)*

Hydrostatic Loading

$(\gamma_{Hs})_c := 1.0$ *(hydrostatic companion loading, EM 1110-2-2104, Table 3-2)*

$(\gamma_{Hs})_{pr.U} := 1.5$ *(hydrostatic principle Usual loading, EM 1110-2-2104, Table 3-2)*

$(\gamma_{Hs})_{pr.N} := 1.4$ *(hydrostatic principle Unusual loading, EM 1110-2-2104, Table 3-2)*

Debris Loading

$(\gamma_{IM})_c := 1.0$ *(debris companion loading, EM 1110-2-2104, Table 3-2)*

$(\gamma_{IM})_{pr.U} := 0$ *(debris principle Usual loading, EM 1110-2-2104, Table 3-2)*

$(\gamma_{IM})_{pr.N} := 1.4$ *(debris principle Unusual loading, EM 1110-2-2104, 3-3d (2) (b))*

$(\gamma_{IM})_{pr.X} := 1.3$ *(debris principle Extreme loading, EM 1110-2-2104, Table 3-2)*

Serviceability Design

$\Phi_{us} := 2.2$ *(serviceability single load case factor for usual flexural and shear loading, EM 1110-2-2104 Table 3-4*

$\Phi_{un} := 1.6$ *(serviceability single load case factor for unusual flexural and shear loading, EM 1110-2-2104 Table 3-4*

STRENGTH LOADING COMBINATIONS

Permanent Loadings

Driving Side

$(\Sigma\gamma_p L_p)_{driving} := \gamma_{c.EH\_driving} \cdot EH_d = 145.6\ lbf$ *(factored driving side permanent loading)*

Resisting Side

$(\Sigma\gamma_p L_p)_{resisting} := \gamma_{c.EH\_resisting} \cdot EH_r = 89.7\ lbf$ *(factored resisting side permanent loading)*

Temporary Loadings

Note: the controlling load combination contains hydrostatic pressure as the principle loading condition. The hydrostatic water level on the resisting side of the wall is to the ground surface.

Driving Side

$(\Sigma\gamma_{pr} L_{tpr})_{driving} := (\gamma_{Hs})_{pr.U} \cdot Hs_{un.d} = 9172.8\ lbf$ *(factored driving side temporary loading)*

Resisting Side

$(\Sigma\gamma_{pr} L_{tpr})_{resisting} := (\gamma_{Hs})_{pr.U} \cdot Hs_{un.r} = 1795.2\ lbf$ *(factored driving side temporary loading)*

Dynamic Loadings

Driving Side

$(\Sigma\gamma_c L_{dc})_{driving} := (\gamma_{IM})_{pr.N} \cdot I_{un} = 140\ lbf$ *(factored driving side temporary loading)*

Resisting Side

Note: there are no applicable dynamic loads to the resisting side of the floodwall

$(\Sigma\gamma_c L_{rc})_{resisting} := 0\ lbf$ *(factored driving side temporary loading)*

Strength Level Shear Combination

$$(LC_{Str})_{shear} := (\Sigma\gamma_p L_p)_{driving} + (\Sigma\gamma_{pr} L_{tpr})_{driving} + (\Sigma\gamma_c L_{dc})_{driving} - (\Sigma\gamma_p L_p)_{resisting} - (\Sigma\gamma_{pr} L_{tpr})_{resisting} - (\Sigma\gamma_c L_{rc})_{resisting} = 7.6\ kip$$

(factored shear loading for T-wall stem at strength level)

Strength Level Moment Combination

$$(LC_{Str})_{moment} := \left((\Sigma\gamma_p L_p)_{driving} \cdot (x)_{EH.d}\right) + \left((\Sigma\gamma_{pr} L_{tpr})_{driving} \cdot (x)_{Hs.un.d}\right) + \left((\Sigma\gamma_c L_{dc})_{driving} \cdot H_{wall}\right) - \left((\Sigma\gamma_p L_p)_{resisting} \cdot (x)_{EH.r}\right) - \left((\Sigma\gamma_{pr} L_{tpr})_{resisting} \cdot (x)_{Hs.un.r}\right) - \left((\Sigma\gamma_c L_{rc})_{resisting} \cdot H_{wall}\right) = 43.2\ kip \cdot ft$$

(factored moment loading for T-wall stem at strength level)

SERVICEABILITY LIMITS

Permanent Loads

Effective Lateral Earth Pressure: Driving Side

$EH_d = 0.097\ kip$ *(usual lateral soil loading, driving side)*

$EH_d = 0.097\ kip$ *(unusual lateral soil loading, driving side)*

Effective Lateral Earth Pressure: Resisting Side

$EH_r = 0.179\ kip$ *(usual lateral soil loading, resisting side)*

$EH_r = 0.179\ kip$ *(unusual lateral soil loading, resisting side)*

Temporary Loadings

Hydrostatic Loading - Driving Side

$Hs_{us.d} = 0\ kip$ *(usual hydrostatic loading or companion load, 10 yr event)*

$Hs_{un.d} = 6.1\ kip$ *(unusual hydrostatic loading or primary load, 100 yr event)*

Hydrostatic Loading - Resisting Side

$Hs_{us.r} = 0\ kip$ *(usual hydrostatic loading or companion load, 10 yr event)*

$Hs_{un.r} = 1.2\ kip$ *(unusual hydrostatic loading or primary load, 100 yr event)*

Dynamic Loading

Debris Loading

$I_{us} = 0\ kip$ *(impact loading, EM 1110-2-2502 6.12.3)*

$I_{un} = 0.1\ kip$ *(impact loading, EM 1110-2-2502 6.12.3)*

Serviceability Level Shear Combinations

$$(LC_{Ser})_{shear} := \Phi_{us} \cdot \left(EH_d + Hs_{un.d} + I_{un}\right) - \Phi_{us} \cdot \left(EH_r + Hs_{un.r}\right) = 10.9\ kip$$

(factored shear loading for T-wall stem at serviceability level)

Serviceability Level Moment Combinations

$$(LC_{Ser})_{moment} := \Phi_{us} \cdot \left(\left(EH_d \cdot (x)_{EH.d}\right) + \left(Hs_{un.d} \cdot (x)_{Hs.un.d}\right) + \left(I_{un} \cdot H_{wall}\right)\right) - \Phi_{us} \cdot \left(\left(EH_r \cdot (x)_{EH.r}\right) + \left(Hs_{un.r} \cdot (x)_{Hs.un.r}\right)\right) = 63.3\ ft \cdot kip$$

(factored shear loading for T-wall stem at serviceability level)

COMPARISON SHEAR AND MOMENT FORCES

Strength Level Loading	Serviceability Level Loading
$(LC_{Str})_{shear} = 7.6\ kip$	$(LC_{Ser})_{shear} = 10.9\ kip$
$(LC_{Str})_{moment} = 43.2\ kip \cdot ft$	$(LC_{Ser})_{moment} = 63.3\ kip \cdot ft$

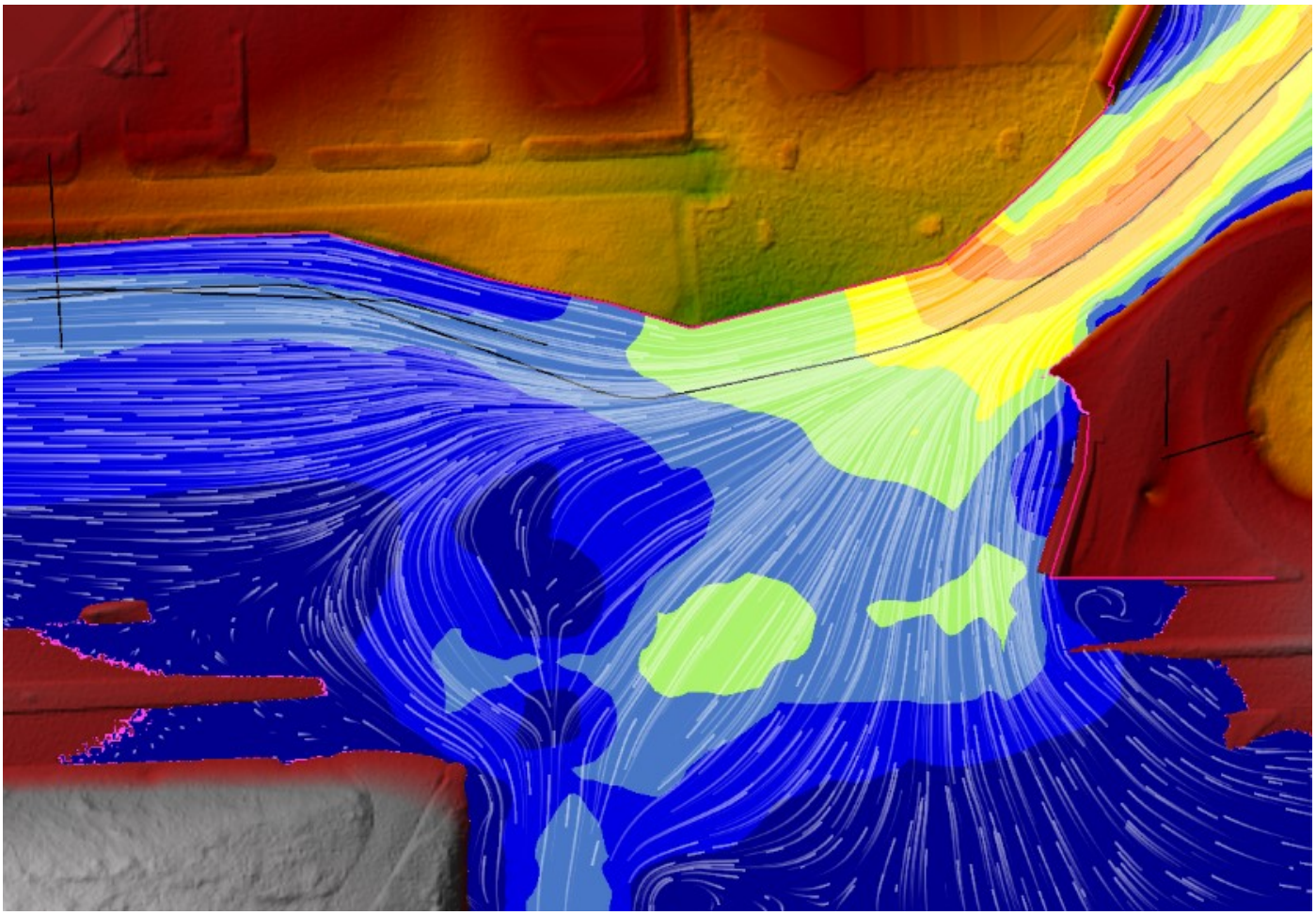
Note: The results show that using the single load factor for serviceability controls over the strength design factors. For the remainder of the design checks, the Serviceability level loadings will be used to size t-wall sections.

IMPACT LOADING

Note: Per EM 1110-2-2502 section 6.12.3 the impacting loading to the tops of T-walls should be considered to be between 100 lbs per foot and 500 lbs per foot depending on the site conditions and wall layout. for the Upper Turkey Creek corridor, channel flows are parallel, or nearly parallel to the face of the wall and thus impact loads would only come from glancing blows from objects being carried through the waterway.

One exception at the confluence between the tributary and Upper Turkey Creek approximately where Shawnee Mission Parkway meets Merriam Drive was reviewed as the T-wall on the left bank appears to be nearly perpendicular to the flow from Turkey Creek. However, after reviewing the velocity vector map provided by the Hydraulics and Hydrology model it can be seen that the flows here are also nearly parallel to the face of the wall.

Per engineering judgement and this analysis, impact loading applied to the top of the wall will be 100 lbs per foot for all wall sections being analyzed.



U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 920.12\ ft$

$HTS := 14\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 5.5\ ft$

$TTOE := 2\ ft$

$TWIDTH := 6\ ft$

$BWIDTH := 18\ ft$

$HK := 3.5\ ft$

$TK := 2\ ft$

$Elev_{soil.d} := 909\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 909\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 906.1\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 904.1\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 2.9\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 2.9\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

********** Echoprint of Input Data

Date: 2025/10/28 Time: 13.40.39

Structural geometry data:

- Elevation of top of stem (ELTS) = 920.12 ft
- Height of stem (HTS) = 14.00 ft
- Thickness top of stem (TTS) = 1.50 ft
- Thickness bottom of stem (TBS) = 1.50 ft
- Dist. of batter at bot. of stem (TBSR) = 0.00 ft
- Depth of heel (THEEL) = 5.50 ft
- Distance of batter for heel (BTRH) = 0.00 ft
- Depth of toe (TTOE) = 2.00 ft
- Width of toe (TWIDTH) = 6.00 ft
- Distance of batter for toe (BTRT) = 0.00 ft
- Width of base (BWIDTH) = 18.00 ft
- Depth of key (HK) = 3.50 ft
- Width of bottom of key (TK) = 2.00 ft
- Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	900.62
0.00	906.12
10.50	906.12
10.50	920.12
12.00	920.12
12.00	906.12
18.00	906.12
18.00	904.12
2.00	904.12
2.00	900.62

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:

Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.120	0.00	909.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-1.20	10.00
2	-3.31	29.00
3	-1.12	500.00

UNDRAINED CASE

********** Echoprint of Input Data **********

Date: 2025/10/28 Time: 13.46.08

Structural geometry data:

- Elevation of top of stem (ELTS) = 920.12 ft
- Height of stem (HTS) = 14.00 ft
- Thickness top of stem (TTS) = 1.50 ft
- Thickness bottom of stem (TBS) = 1.50 ft
- Dist. of batter at bot. of stem (TBSR) = 0.00 ft
- Depth of heel (THEEL) = 5.50 ft
- Distance of batter for heel (BTRH) = 0.00 ft
- Depth of toe (TTOE) = 2.00 ft
- Width of toe (TWIDTH) = 6.00 ft
- Distance of batter for toe (BTRT) = 0.00 ft
- Width of base (BWIDTH) = 18.00 ft
- Depth of key (HK) = 3.50 ft
- Width of bottom of key (TK) = 2.00 ft
- Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	900.62
0.00	906.12
10.50	906.12
10.50	920.12
12.00	920.12
12.00	906.12
18.00	906.12
18.00	904.12
2.00	904.12
2.00	900.62

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:

Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.120	0.00	909.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-1.20	10.00
2	-3.31	29.00
3	-1.12	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1028.50	853.33
2	-528.50	853.33
3	-28.50	900.00
4	0.50	908.00
5	10.50	909.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.120	909.00	0.77

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	12.00	909.00
2	512.00	941.08

Foundation property data:
- phi for soil-structure interface = 28.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 28.00 (deg)
- c for soil-soil interface = 0.000 (ksf)

Water data:
- Driving side elevation = 920.12 ft
- Resisting side elevation = 908.86 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.00	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1028.50	853.33
2	-528.50	853.33
3	-28.50	900.00
4	0.50	908.00
5	10.50	909.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.120	909.00	0.77

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	12.00	909.00
2	512.00	941.08

Foundation property data:
- phi for soil-structure interface = 0.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 0.00 (deg)
- c for soil-soil interface = 1.000 (ksf)

Water data:
- Driving side elevation = 920.12 ft
- Resisting side elevation = 908.86 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.00	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

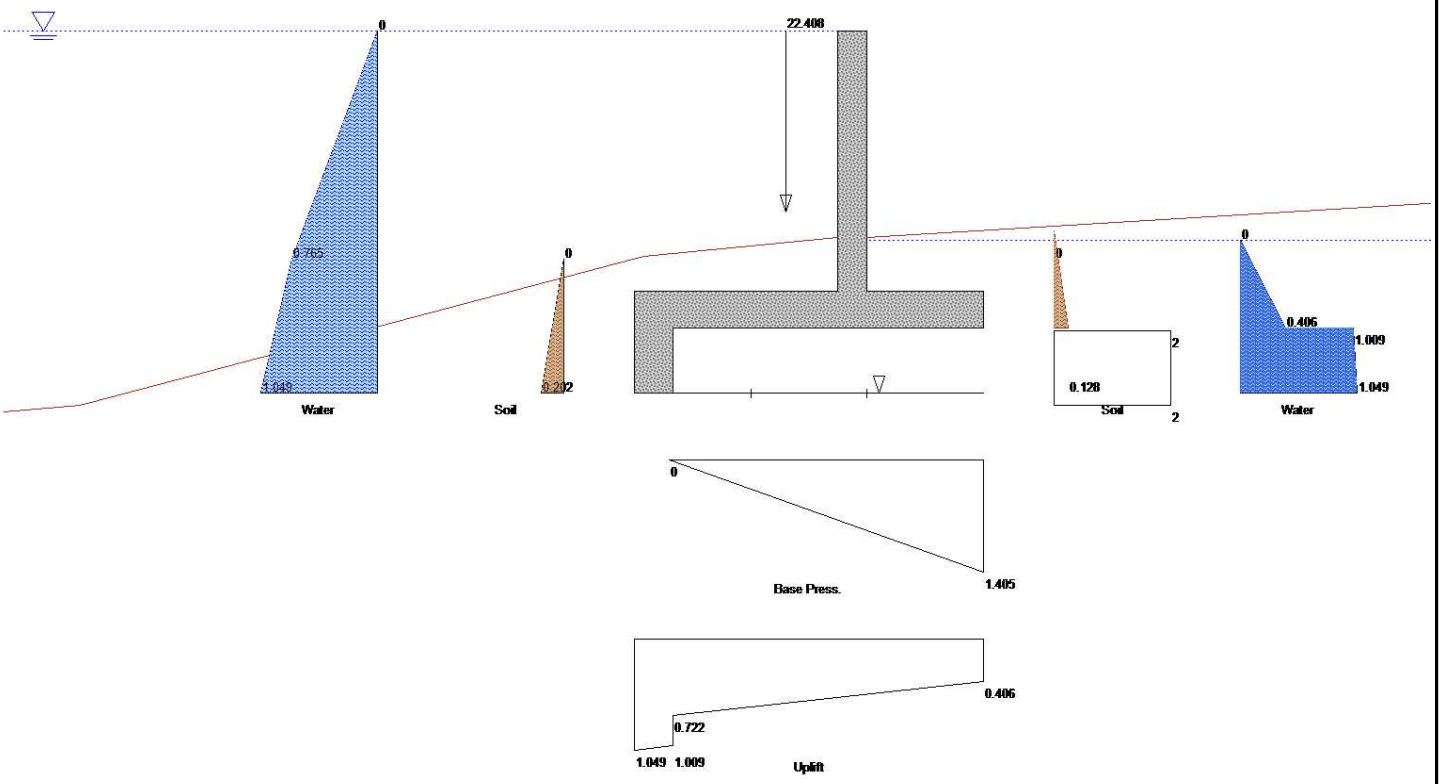
*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

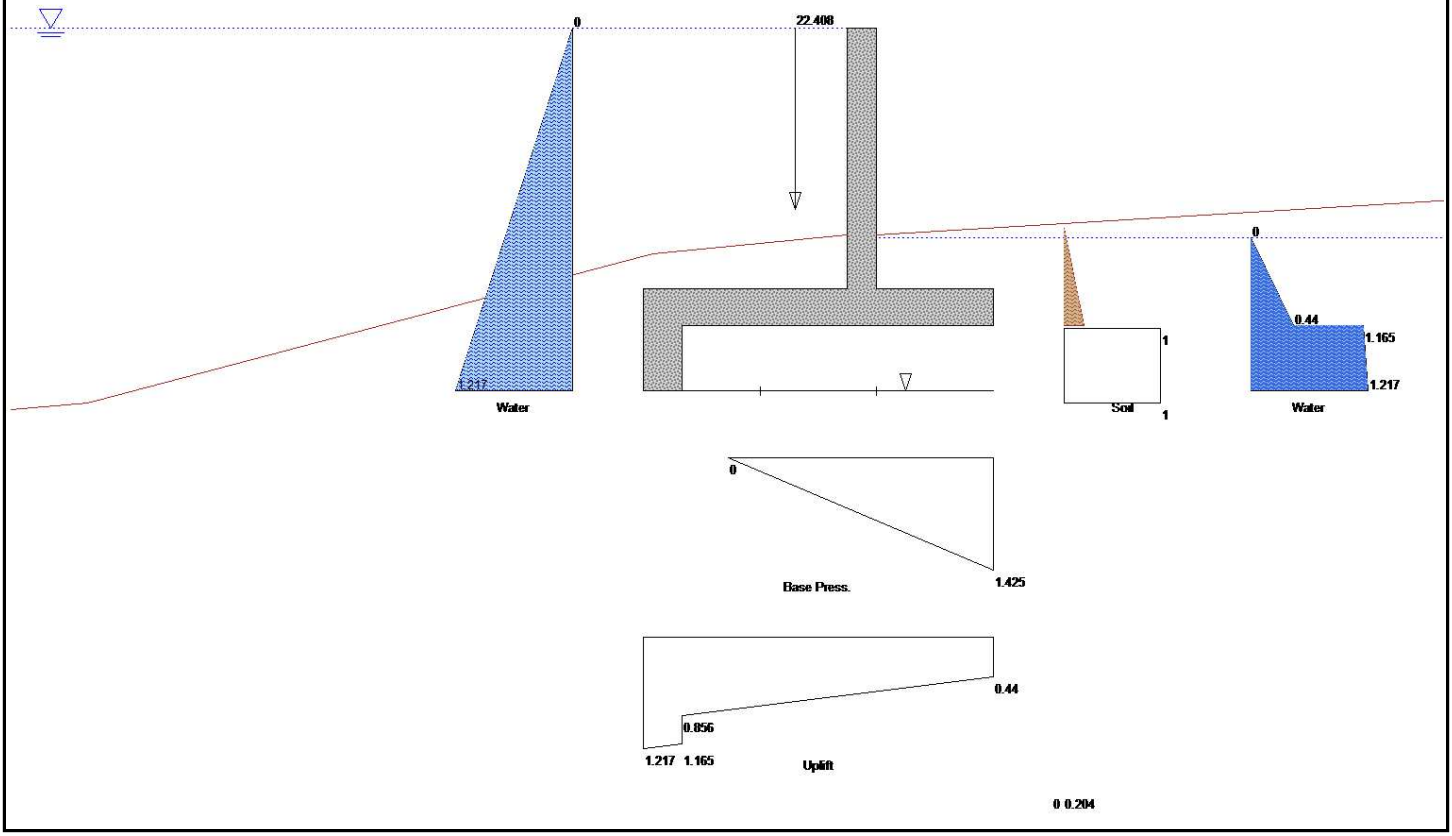
Input file= RS 40+24 RB 30+20 14Ft Wall _ Drained Case.dat

CT Wall - Drained Case Output



Input file= RS 40+24 RB 30+20 14Ft Wall _ Undrained Case.dat

CT Wall - Undrained Case Output



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.202 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.128 \\ 0.204 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 1.41 \\ 1.13 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 1.41 \\ 1.13 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 1.405 \\ 1.425 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

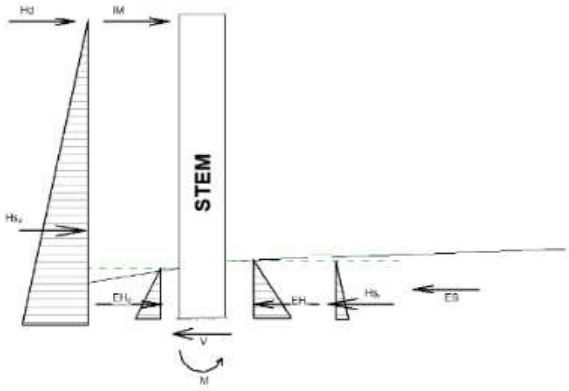
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 873.6 \\ 873.6 \end{bmatrix} psf$ *(hydrostatic pressure loading to driving side)*

$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 6.115 \\ 6.115 \end{bmatrix} klf$ *(hydrostatic loading to driving side)*

$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 4.7\ ft$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 0.18 \\ 0.18 \end{bmatrix} ksf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} klf$ *(hydrostatic loading to resisting side)*

$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.96\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.069 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side of stem)*

$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to driving side of stem)*

$(x_{EH})_d := HSD \cdot \frac{1}{3} = 1\ ft$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.076 \\ 0.12 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side of stem)*

$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0.1 \\ 0.2 \end{bmatrix} klf$ *(lateral earth loading to resisting side of stem)*

$(x_{EH})_r := HSR \cdot \frac{1}{3} = 1\ ft$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

$(x_{IM})_d := HTS = 14\ ft$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$p_{const} := 250\ psf$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$d_{soil} := HSR = 2.9\ ft$ *(depth of soil cover on stem)*

$(x_Q)_d := \frac{HSR}{2} = 1.4\ ft$ *(resultant height above base of stem)*

$x_Q := d_{soil} - (x_Q)_d = 1.4\ ft$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.382\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 11.1\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 11.1\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.194\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.374\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 8.4\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 3.2\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.2\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.2\ klf$

$(x_{EQ}) := HSR = 2.9\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot \langle EH_d + Hs_d + IM \rangle = \begin{bmatrix} 10.1 \\ 9.9 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \langle EH_d + EQ \rangle = \begin{bmatrix} 0.5 \\ 0.3 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \langle EH_d + Q \rangle = \begin{bmatrix} 0.8 \\ 0.6 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot \langle EH_r + Hs_r - W_c \rangle = \begin{bmatrix} 0.3 \\ 0.4 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \langle EH_r \rangle = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \langle EH_r \rangle = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 9.8 \\ 9.6 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.3 \\ 0 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.6 \\ 0.3 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 9.8\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot (\langle EH_d \cdot (x_{EH})_d \rangle + \langle Hs_d \cdot (x_{Hs})_d \rangle + IM \cdot (x_{IM})_d \rangle = \begin{bmatrix} 48.1 \\ 47.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot (\langle EH_d \cdot (x_{EH})_d \rangle + \langle EQ \cdot (x_{EQ}) \rangle) = \begin{bmatrix} 1 \\ 0.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot (\langle EH_d \cdot (x_{EH})_d \rangle + \langle Q \cdot \langle (x_Q)_d \rangle \rangle) = \begin{bmatrix} 1 \\ 0.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot (\langle EH_r \cdot (x_{EH})_r \rangle + \langle Hs_r \cdot (x_{Hs})_r \rangle) = \begin{bmatrix} 0.6 \\ 0.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot (\langle EH_r \cdot (x_{EH})_r \rangle) = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot (\langle EH_r \cdot (x_{EH})_r \rangle) = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 47.5 \\ 47.2 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.9 \\ 0.6 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.9 \\ 0.6 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 47.5\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v(X_U, X_N) := \left| \begin{array}{l} \text{for } i \in \text{ORIGIN} .. \text{last}(X_U) \\ \quad \left| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left| ch_{i,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check(X_U, X_N) := \left| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left| ch_{1,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - (0.5 \cdot (d_b)_{stem.vert}) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)

$$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$$

(nominal flexural strength, EM1110-2-2104, B-49)

$$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{"OK"} \quad 0.95]$$

(nominal flexural check, with Utilization ratio)

Minimum Required Steel for Flexure

Solve Constraints Given Values

$$As_{req} := 1.5\ in^2$$

$$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$$

$$As_{req.stem} := \text{find}\left(As_{req}\right) = 0.772\ in^2$$

$$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.772\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.584\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 1.03\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{stem.req} := \begin{cases} as \leftarrow (A_{s.min})_{stem.ten} & \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \\ as \leftarrow A_{s.min} & \text{else} \end{cases} = 0.584\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{stem.vert} = 0.8\ in^2$$

(area of steel provided in the vertical direction for the stem)

$$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.7]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 14 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 14 ft Floodwall Stem - Driving Side

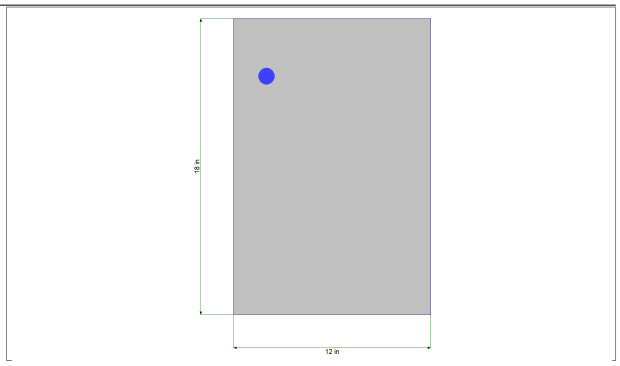
CODE REFERENCES

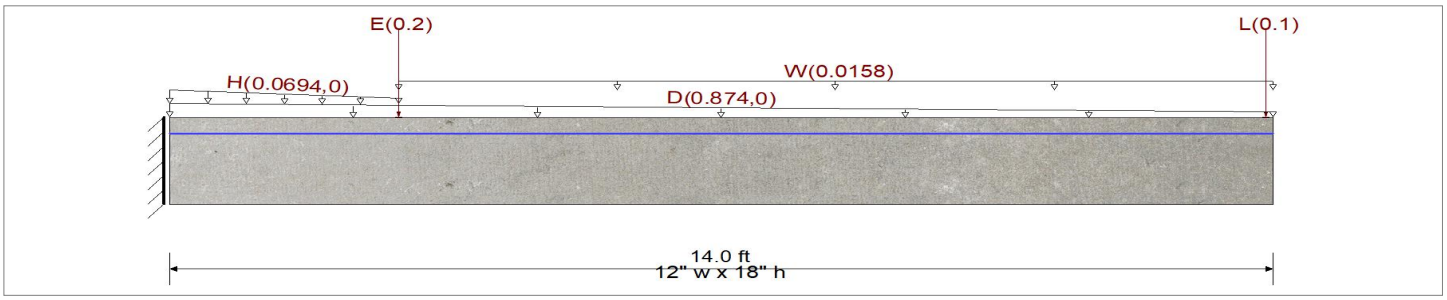
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	=	4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	503.12 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.8250
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,823.68 ksi	Fy - Stirrups		60.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		
			Number of Resisting Legs Per Stirrup =		

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.50 in from Top, from 0.0 to 14.0 ft in this span

Load for Span Number 1

Varying Uniform Load : D= 0.8740->0.0 k/ft, Extent = 0.0 -->> 14.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Driving side)

Varying Uniform Load : H= 0.06940->0.0 k/ft, Extent = 0.0 -->> 2.90 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 13.90 ft, (Impact Loading - Driving Side)

Uniform Load : W = 0.01580 k/ft, Extent = 2.90 -->> 14.0 ft, Tributary Width = 1.0 ft, (Wind Load - Resisting Side [Negative Pressure])

Point Load : E = 0.20 k @ 2.90 ft, (Earthquake Loading - Driving Side)

DESIGN SUMMARY

Check As Min Limits!

Maximum Bending Stress Ratio =	**0.970** : 1				
Section used for this span	**Typical Section**				
Mu : Applied	-48.208 k-ft				
Mn * Phi : Allowable	49.712 k-ft				
Location of maximum on span	0.000 ft				
Span # where maximum occurs	Span # 1				
Maximum Deflection					
Max Downward Transient Deflection	0.000 in	Ratio =	0	<480.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<480.0	
Max Downward Total Deflection	0.215 in	Ratio =	1560	>=480.0	Span: 1 : +D+L+H
Max Upward Total Deflection	0.000 in	Ratio =	0	<480.0	Span: 1 : +D+E+H

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 14 ft Floodwall Stem - Driving Side

Cross Section Strength & Inertia

Top & Bottom references are for tension side of sect

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Bottom	Phi*Mn (k-ft) Top	Moment of Inertia (in^4) I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.5",	0.00	49.71	5,832.00	37.39	895.29

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	10.270	
Max Upward from Load Combinations	10.270	
+1.60D+1.60W+1.60H	10.230	
+1.60D+1.60E+1.60H	10.270	
+1.60H	0.161	
+1.60D+1.60L+1.60H	10.110	

Shear Stirrup Requirements

Entire Beam Span Length : Phi*lambda*sqrt(f'c)*bw*d < Vu <= Phi*Vc, Req'd Vs = Min per 9.6.3.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	14.000	-48.21	49.71	0.97
+1.60D+1.60W+1.60H					
Span # 1	1	14.000	-48.21	49.71	0.97
+1.60D+1.60E+1.60H					
Span # 1	1	14.000	-46.76	49.71	0.94
+1.60H					
Span # 1	1	14.000	-0.16	49.71	0.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+L+H	1	0.2152	14.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+W+H	1	0.2141	14.000	0.0000	0.000
+D+L+H	1	0.2152	14.000	0.0000	0.000
+D+E+H	1	0.1916	14.000	0.0000	0.000
H Only	1	0.0001	14.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.8\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 47.1\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 35.3\ kip$ *(factored shear-friction strength)*

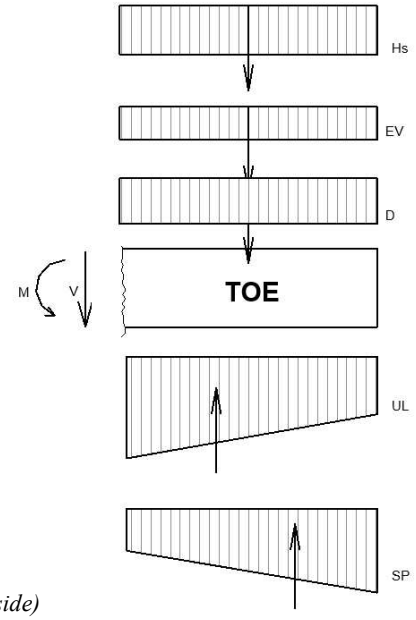
$V_{stem} := (Vu)_{stem} \cdot b = 9.8\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{“OK”}\ \ 0.3]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 300\ psf$

$D := \sigma_D \cdot TWIDTH = 1.8\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 3\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 179.7 \\ 179.7 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 1.1 \\ 1.1 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 3\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 165.9\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.995 \\ 0.995 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 3\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -1.405 \\ -1.425 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 18\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 16.1 \\ 13.6 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 6\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 10.1 \\ 7.6 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -0.88 \\ -0.79 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1 \right)}{2} = \begin{bmatrix} -6.9 \\ -6.7 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 3.2 \\ 3.3 \end{bmatrix} ft$$

(moment arm from the connection of the toe to the resultant of the soil pressure loading)

Uplift Pressures

$$(\sigma_{UL})_{min} = \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$$

(minimum uplift pressure at the end of the toe)

$$(\sigma_{UL})_{max} = \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$$

(maximum uplift pressure behind the key)

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$BWIDTH = 18\ ft$ *(foundation width)*

$TK = 2\ ft$ *(key width)*

$L := BWIDTH - TK = 16\ ft$ *(length of linear uplift pressure)*

$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$ *(linear relationship)*

$$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0198 \\ 0.026 \end{bmatrix} \frac{ksf}{ft}$$

(linear relationship)

$$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.525 \\ 0.596 \end{bmatrix} ksf$$

(uplift pressure at the connection of the toe to the foundation)

$$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -2.8 \\ -3.1 \end{bmatrix} klf$$

(uplift pressure resultant for the toe of the foundation)

$$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 2.9 \\ 2.8 \end{bmatrix} ft$$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -9.2 \\ -9.4 \end{bmatrix} klf$$

(load case 1 for infrequent flooding)

$$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 9.4\ klf$$

(maximum of the drained vs undrained cases)

Factored Shear on Toe at Face of Stem

$$(V_u)_{toe} := \Sigma F_{toe} = 9.4\ klf$$

(maximum factored shear in toe)

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -29.7 \\ -30.6 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$Mu_{toe} := \max(LC1) \cdot 1\ ft = -29.7\ kip \cdot ft$ *(resultant moment at base of floodwall stem per foot of wall)*

$Mu_{toe} := -Mu_{toe}$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \text{Bar Size: \#8} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 19.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$a_{toe.bot} := \frac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \frac{a_{toe.bot}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{"OK"} \quad 0.44]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \text{find}\left(As_{req}\right) = 0.5\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.785\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.785\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.6\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.472\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.6]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.4\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.6]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0034$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.43\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 1]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0019$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.2]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 288\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 31.4\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 23.5\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 9.4\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.4]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

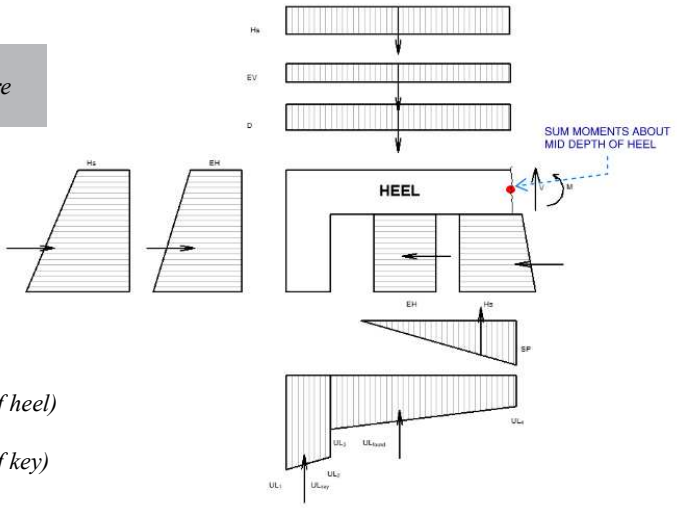
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 10.5\ ft$ *(Length of heel, includes key)*

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 300\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 525\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 3.2\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 1.1\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 4.2\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 5.3\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 9.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 6.3\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.874\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 9.2\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 5.25\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.874\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 1.217\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 5.7\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 3.9\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 3.6 \\ 4.2 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 2.8 \\ 2.8 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.088\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.256\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.9\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 4.198\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.41 \\ 1.13 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.41 \\ 1.13 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 4.9 \\ 4 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 2.8 \\ 2.8 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.166\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 1.74\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 5.25\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 1.87 \\ 4.44 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 8.63 \\ 6.06 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.75 \\ 0.64 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 3.24 \\ 1.93 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 2.9 \\ 2 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

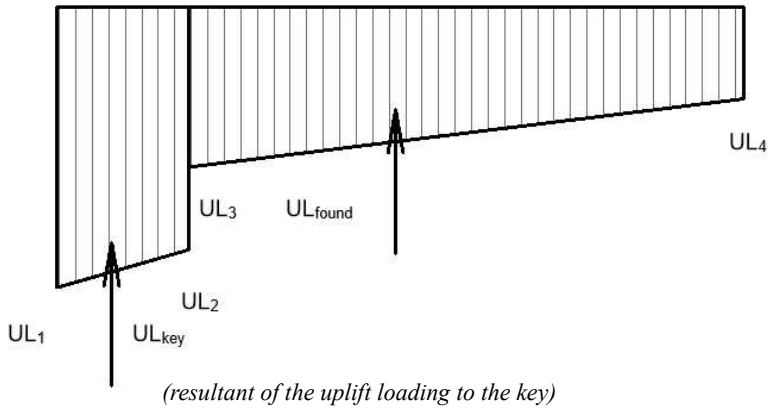
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot (UL_1 + UL_2)}{2} = \begin{bmatrix} 2.1 \\ 2.4 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 8.5\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot (2 \cdot UL_1 + UL_2)}{3 \cdot (UL_1 + UL_2)} \right) = \begin{bmatrix} 9.51 \\ 9.51 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{(UL_4 - UL_3)}{BWIDTH - TK} = \begin{bmatrix} -0.0198 \\ -0.026 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.6 \\ 0.6 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot (UL_3 + UL_{stem})}{2} = \begin{bmatrix} 5.4 \\ 6.3 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot (2 \cdot UL_3 + UL_{stem})}{3 \cdot (UL_3 + UL_{stem})} \right) = \begin{bmatrix} 4.44 \\ 4.46 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left((EH)_{fh} - (EH)_{bh} \right) \\ + \left((Hs)_{fh} - (Hs)_{bh} \right) \end{pmatrix} = \begin{bmatrix} -2.9 \\ -2.3 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -2.3\ klf$

$(\Sigma F_{heel})_{horz} := \max\left(\overrightarrow{|LC1|}\right) = 2.9\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$ $= 0\ kip$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$

Forces on Heel - Vertical Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 41.3 \\ 41.2 \end{bmatrix} klf$ *(summation of the forces in the vertical direction)*

$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 41.3\ klf$ *(maximum of the drained vs undrained cases)*

Resultant Shear at Base

$Vu_{heel} := (\Sigma F_{heel})_{vert} = 41.3\ klf$ *(factored shear at connection of floodwall heel)*

Resultant Axial Load in Base

$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 2.9\ klf$ *(factored axial load at floodwall heel)*

$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$ *(tension in heel; from check above, zero if heel in compression)*

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{D \cdot (x_D)_{heel}}\right) \\ + \left(\overrightarrow{(EV)_{ah} \cdot (x_{EV})_{ah}}\right) + \left(\overrightarrow{(Hs)_{ah} \cdot (x_{Hs})_{ah}}\right) \\ + \left(\overrightarrow{(EH)_{fh} \cdot (x_{EH})_{fh}}\right) + \left(\overrightarrow{(Hs)_{fh} \cdot (x_{Hs})_{fh}}\right) \end{pmatrix} = 176.3\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{(Hs)_{bh} \cdot (x_{Hs})_{bh}}\right) \\ + \left(\overrightarrow{(EH)_{bh} \cdot (x_{EH})_{bh}}\right) \\ + \left(\overrightarrow{SP_{heel} \cdot (x_{SP})_{heel}}\right) \\ + \left(\overrightarrow{UL_{key} \cdot (x_{UL})_{key}}\right) \\ + \left(\overrightarrow{UL_{found} \cdot (x_{UL})_{found}}\right) \end{pmatrix} = \begin{bmatrix} 122.4 \\ 123.5 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = 53.98\ kip$

$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = 54\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$Mu_{heel} = 54\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

$(P_u)_{heel} = 2.9\ \frac{kip}{ft}$ *(maximum factored axial tension at base of floodwall stem on the heel)*

$Vu_{heel} = 41.3\ \frac{kip}{ft}$ *(maximum factored shear at base of floodwall stem on the heel)*

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 8\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$$(A_s)_{heel.top} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 1.178\ in^2$$ *(area of steel for top reinforcement in heel)*

$$(A_s)_{heel.bot} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$$ *(area of steel for top reinforcement in heel)*

$$\rho := \frac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.004$$ *(reinforcement ratio for primary tension steel)*

$$\rho' := \frac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.003$$ *(reinforcement ratio for bottom tension steel)*

$$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 20.5\ in$$ *(depth to top reinforcement in heel)*

$$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$$a_{heel.top} := \frac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.94\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 117.8\ kip$$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$$\phi Pn := \phi_A \cdot Pn_{max} = 106\ kip$$

$$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”}\ \ 0.03]$$ *(tensile)*

Flexural Check

$$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \frac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right) = 62.9\ kip \cdot ft$$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”}\ \ 0.9]$$

Minimum Required Steel for Flexure

Guess Values

$$As_{req} := 1.5\ in^2$$

Constraints

$$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \frac{\dfrac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}}{2}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right)\right)$$

Solver

$$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.613\ in^2$$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0.6\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.825\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.82\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.825\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0.8\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} := \text{if } A_{s\_1.3} < (A_s)_{heel.top} \parallel as \leftarrow (A_{s.min})_{heel.top}$ else $\parallel as \leftarrow A_{s.min} = 0.613\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 1.2\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad 0.52]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.4\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.5]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0036$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 8\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

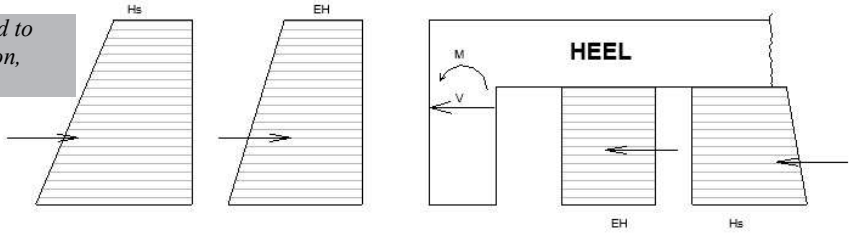
Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



$HK = 3.5\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 5.5\ ft$ *(depth of key, from top of foundation down)*

Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.998\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 1.217\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 3.9\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 1.81\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 3.6 \\ 4.2 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot (2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 1.8 \\ 1.8 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.489\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.596\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 1.9\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 1.808\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.41 \\ 1.13 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.41 \\ 1.13 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 4.9 \\ 4 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 1.8 \\ 1.8 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 16.7\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 24 \\ 22.8 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 7.3 \\ 6.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 7.3\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 19.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.11]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Given Values

$As_{req} := 1.5\ in^2$

Constraints

$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} \right)$

Solve

$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.083\ in^2$

$(A_{s.min})_{key.min} := As_{req.stem} = 0.1\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2} \right) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b \right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.785\ in^2$

$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{key.req} := \text{if } A_{s\_1.3} < (A_s)_{key.main} \ \| \ as \leftarrow (A_{s.min})_{key.min} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.083\ in^2$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{key.main} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main} \right) = [\text{“OK”} \quad 0.1]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Requirements

$(A_{s.min})_{temp} = 0.4\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main} \right) = [\text{“OK”} \quad 0.6]$

(temperature and shrinkage steel of key, check utilization ratio)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}} \right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1} \right)} \right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main} \right)} = 0.0034$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max} \right) = [\text{“OK”} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{key.lat} := \text{Bar Size: \#6} = 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in key)*

$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in key)*

$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 18.6\ in$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.4\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{"OK"} \quad 0.5]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.lat}}{(b \cdot d_{key.main})} = 0.0038$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

14 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS := 920.12\ ft$

$HTS := 14\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 5.5\ ft$

$TTOE := 2\ ft$

$TWIDTH := 6\ ft$

$BWIDTH := 18\ ft$

$HK := 3.5\ ft$

$TK := 2\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 8\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

12 FT T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 921.50\ ft$

$HTS := 12\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 4\ ft$

$TTOE := 2\ ft$

$TWIDTH := 4\ ft$

$BWIDTH := 12.5\ ft$

$HK := 2\ ft$

$TK := 3\ ft$

$Elev_{soil.d} := 912\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 912\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 909.5\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 907.5\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 2.5\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 2.5\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

********** Echoprint of Input Data **********

Date: 2025/11/ 9 Time: 8.07.34

Structural geometry data:

- Elevation of top of stem (ELTS) = 921.50 ft
- Height of stem (HTS) = 12.00 ft
- Thickness top of stem (TTS) = 1.50 ft
- Thickness bottom of stem (TBS) = 1.50 ft
- Dist. of batter at bot. of stem (TBSR) = 0.00 ft
- Depth of heel (THEEL) = 4.00 ft
- Distance of batter for heel (BTRH) = 0.00 ft
- Depth of toe (TTOE) = 2.00 ft
- Width of toe (TWIDTH) = 4.00 ft
- Distance of batter for toe (BTRT) = 0.00 ft
- Width of base (BWIDTH) = 12.50 ft
- Depth of key (HK) = 2.00 ft
- Width of bottom of key (TK) = 3.00 ft
- Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	905.50
0.00	909.50
7.00	909.50
7.00	921.50
8.50	921.50
8.50	909.50
12.50	909.50
12.50	907.50
3.00	907.50
3.00	905.50

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:

Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	0.00	11.00
2	-10.29	17.50
3	-0.89	500.00

UNDRAINED CASE

********** Echoprint of Input Data **********

Date: 2025/11/ 9 Time: 8.10.12

Structural geometry data:

- Elevation of top of stem (ELTS) = 921.50 ft
- Height of stem (HTS) = 12.00 ft
- Thickness top of stem (TTS) = 1.50 ft
- Thickness bottom of stem (TBS) = 1.50 ft
- Dist. of batter at bot. of stem (TBSR) = 0.00 ft
- Depth of heel (THEEL) = 4.00 ft
- Distance of batter for heel (BTRH) = 0.00 ft
- Depth of toe (TTOE) = 2.00 ft
- Width of toe (TWIDTH) = 4.00 ft
- Distance of batter for toe (BTRT) = 0.00 ft
- Width of base (BWIDTH) = 12.50 ft
- Depth of key (HK) = 2.00 ft
- Width of bottom of key (TK) = 3.00 ft
- Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	905.50
0.00	909.50
7.00	909.50
7.00	921.50
8.50	921.50
8.50	909.50
12.50	909.50
12.50	907.50
3.00	907.50
3.00	905.50

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:

Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	0.00	11.00
2	-10.29	17.50
3	-0.89	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1021.50	859.91
2	-521.50	859.91
3	-21.50	896.99
4	-4.00	912.00
5	7.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	915.00	0.65

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	8.50	915.00
2	508.50	942.08

Foundation property data:
- phi for soil-structure interface = 28.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 28.00 (deg)
- c for soil-soil interface = 0.000 (ksf)

Water data:
- Driving side elevation = 921.27 ft
- Resisting side elevation = 915.00 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
921.20	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1021.50	859.91
2	-521.50	859.91
3	-21.50	896.99
4	-4.00	912.00
5	7.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	915.00	0.65

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	8.50	915.00
2	508.50	942.08

Foundation property data:
- phi for soil-structure interface = 0.00 (deg)
- c for soil-structure interface = 1.000 (ksf)
- phi for soil-soil interface = 0.00 (deg)
- c for soil-soil interface = 1.000 (ksf)

Water data:
- Driving side elevation = 921.27 ft
- Resisting side elevation = 915.00 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
921.20	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack *is* down to bottom of heel
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

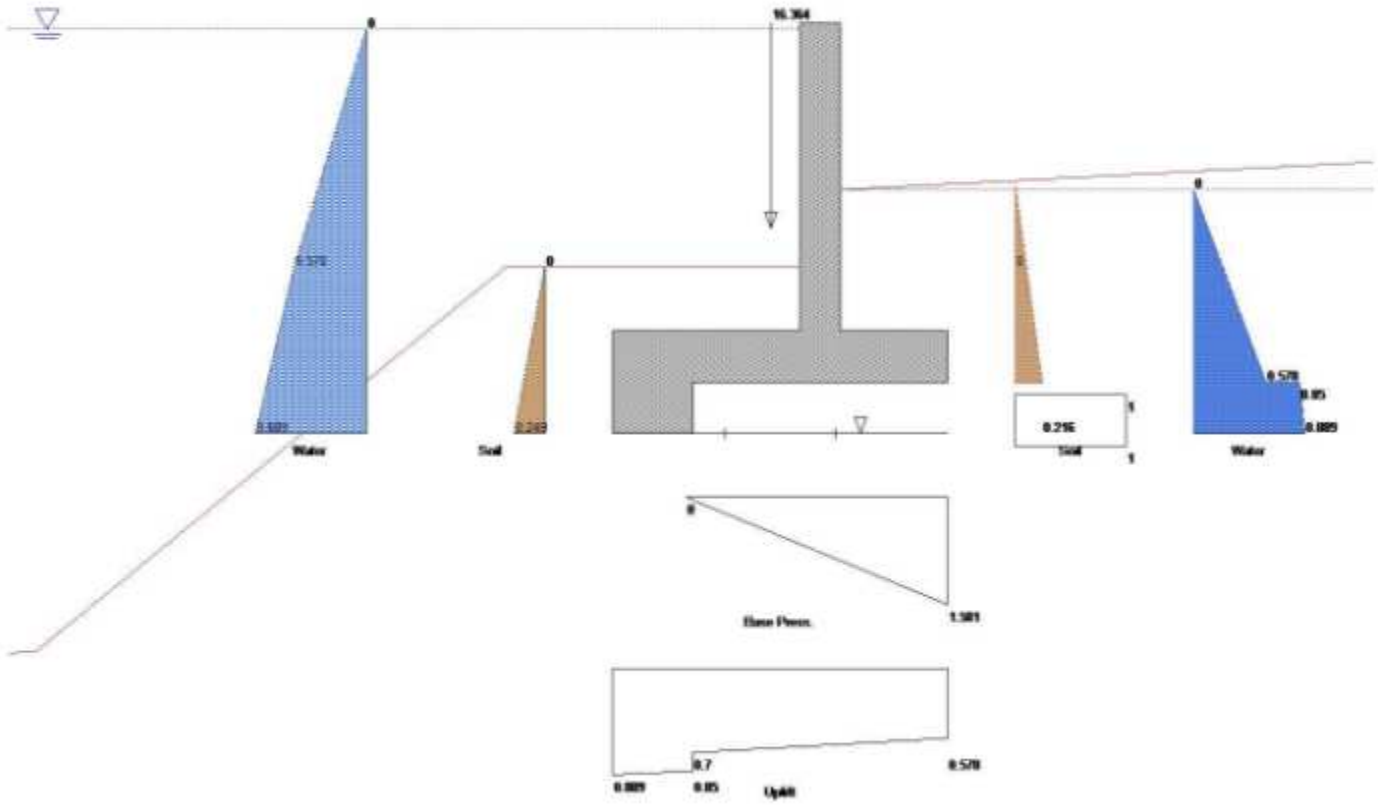
*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

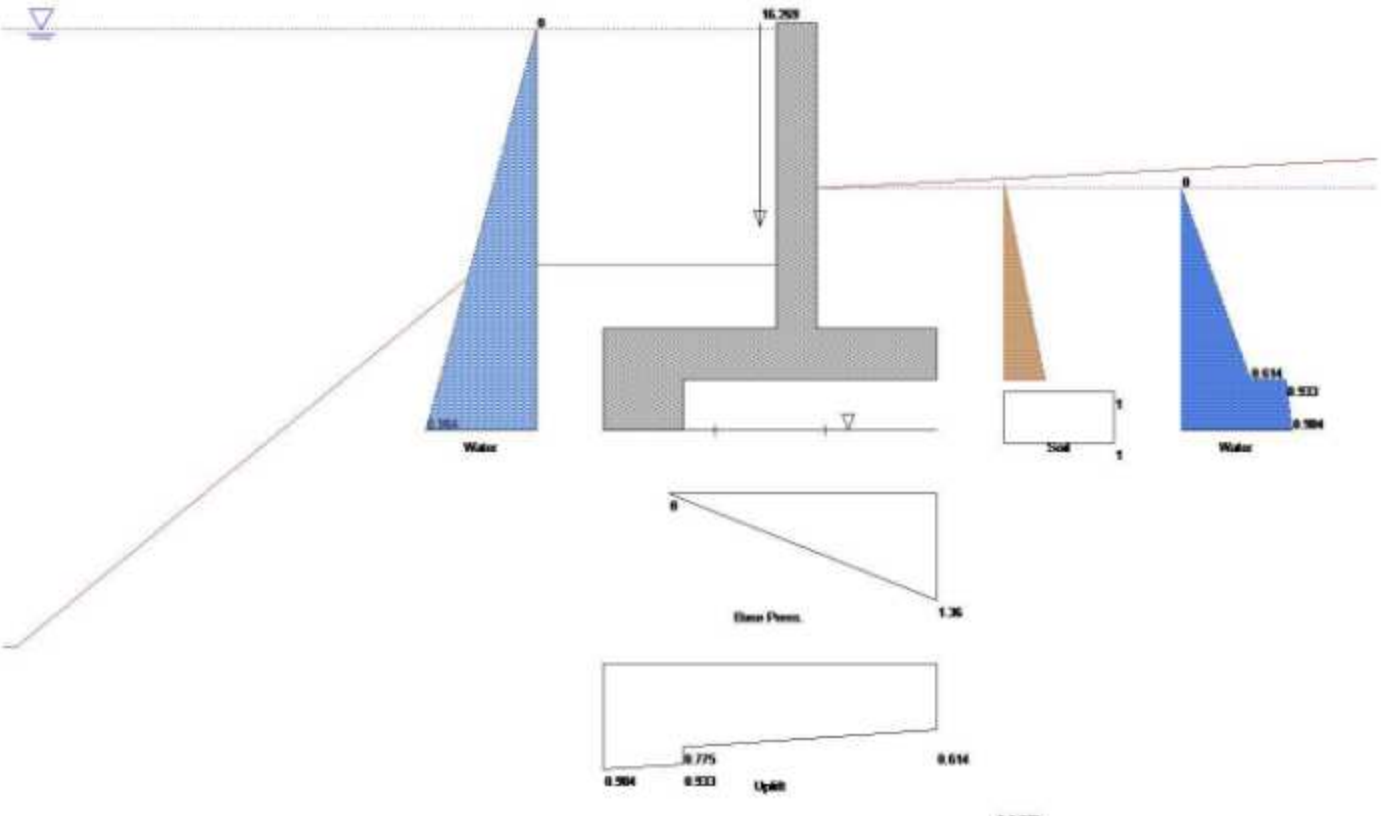
Input file: RS 32+61 RB 24+40 ALT 12FT WALL-Drained.dat

CT Wall - Drained Case Output



Input file: RS 32+61 RB 24+40 ALT 12FT WALL-Undrained.dat

CT Wall - Undrained Case Output



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.249 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.216 \\ 0.372 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 1.81 \\ 1.10 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 1.81 \\ 1.1 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 1.581 \\ 1.36 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 0.889 \\ 0.984 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 0.85 \\ 0.933 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.700 \\ 0.775 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.578 \\ 0.614 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.578 \\ 0.614 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.7 \\ 0.775 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 0.85 \\ 0.933 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

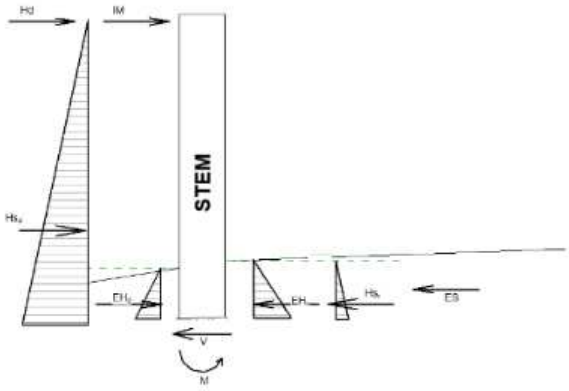
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 0.889 \\ 0.984 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 748.8 \\ 748.8 \end{bmatrix} psf$ *(hydrostatic pressure loading to driving side)*

$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 4.493 \\ 4.493 \end{bmatrix} klf$ *(hydrostatic loading to driving side)*

$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 4\ ft$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 156 \\ 156 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} klf$ *(hydrostatic loading to resisting side)*

$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.83\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.096 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side of stem)*

$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to driving side of stem)*

$(x_{EH})_d := HSD \cdot \frac{1}{3} = 0.8\ ft$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.12 \\ 0.207 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side of stem)*

$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} klf$ *(lateral earth loading to resisting side of stem)*

$(x_{EH})_r := HSR \cdot \frac{1}{3} = 0.8\ ft$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

$(x_{IM})_d := HTS = 12\ ft$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$p_{const} := 250\ psf$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$d_{soil} := HSR = 2.5\ ft$ *(depth of soil cover on stem)*

$(x_Q)_d := \frac{HSR}{2} = 1.3\ ft$ *(resultant height above base of stem)*

$x_Q := d_{soil} - (x_Q)_d = 1.3\ ft$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.332\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 9.5\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 9.5\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.166\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.319\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 7.3\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 2.7\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.2\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.2\ klf$

$(x_{EQ}) := HSR = 2.5\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Hs_d + IM) = \begin{bmatrix} 7.5 \\ 7.3 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot (EH_d + EQ) = \begin{bmatrix} 0.5 \\ 0.3 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Q) = \begin{bmatrix} 0.7 \\ 0.5 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot (EH_r + Hs_r - W_c) = \begin{bmatrix} 0.3 \\ 0.5 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.2 \\ 0.4 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.2 \\ 0.4 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 7.3 \\ 6.9 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.2 \\ -0.2 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.5 \\ 0.1 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 7.3\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Hs_d \cdot (x_{Hs})_d \right) + IM \cdot (x_{IM})_d \right) = \begin{bmatrix} 30.8 \\ 30.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(EQ \cdot (x_{EQ}) \right) \right) = \begin{bmatrix} 0.8 \\ 0.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Q \cdot \left((x_Q)_d \right) \right) \right) = \begin{bmatrix} 0.8 \\ 0.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) + \left(Hs_r \cdot (x_{Hs})_r \right) \right) = \begin{bmatrix} 0.5 \\ 0.6 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0.2 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 30.4 \\ 30.1 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.6 \\ 0.3 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.6 \\ 0.3 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 30.4\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$Check_v(X_U, X_N) :=$
- for $i \in$ **ORIGIN** .. last(X_U)
  - if $X_{U_i} > X_{N_i}$
    - $ch_{i,1} \leftarrow$ "NO GOOD"
    - $ch_{i,2} \leftarrow \frac{X_{U_i}}{X_{N_i}}$
  - else if $X_{N_i} = 0$ *(if the capacity is zero, used for error handling)*
    - $ch_{i,1} \leftarrow$ "N/A"
  - else *(if the required is less than the capacity)*
    - $ch_{i,1} \leftarrow$ "OK"
    - $ch_{i,2} \leftarrow \frac{X_{U_i}}{X_{N_i}}$
- ch

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$Check(X_U, X_N) :=$
- if $X_U > X_N$
  - $ch_{1,1} \leftarrow$ "NO GOOD"
  - $ch_{1,2} \leftarrow \frac{X_U}{X_N}$
- else if $X_U = 0$ *(if the capacity is zero, used for error handling)*
  - $ch_{1,1} \leftarrow$ "N/A"
- else *(if the required is less than the capacity)*
  - $ch_{1,1} \leftarrow$ "OK"
  - $ch_{1,2} \leftarrow \frac{X_U}{X_N}$
- ch

(if the required is greater than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)

$$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$$

(nominal flexural strength, EM1110-2-2104, B-49)

$$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{"OK"} \quad 0.61]$$

(nominal flexural check, with Utilization ratio)

Minimum Required Steel for Flexure

Solve Constraints Given Values

$$As_{req} := 1.5\ in^2$$

$$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.483\ in^2$$

$$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.483\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.584\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.644\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{stem.req} := \begin{cases} as \leftarrow (A_{s.min})_{stem.ten} & \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \\ as \leftarrow A_{s.min} & \text{else} \end{cases} = 0.483\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{stem.vert} = 0.8\ in^2$$

(area of steel provided in the vertical direction for the stem)

$$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.6]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 12 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 12 ft Floodwall Stem - Driving Side

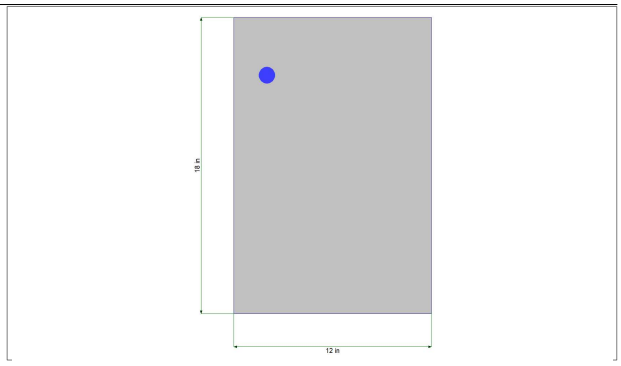
CODE REFERENCES

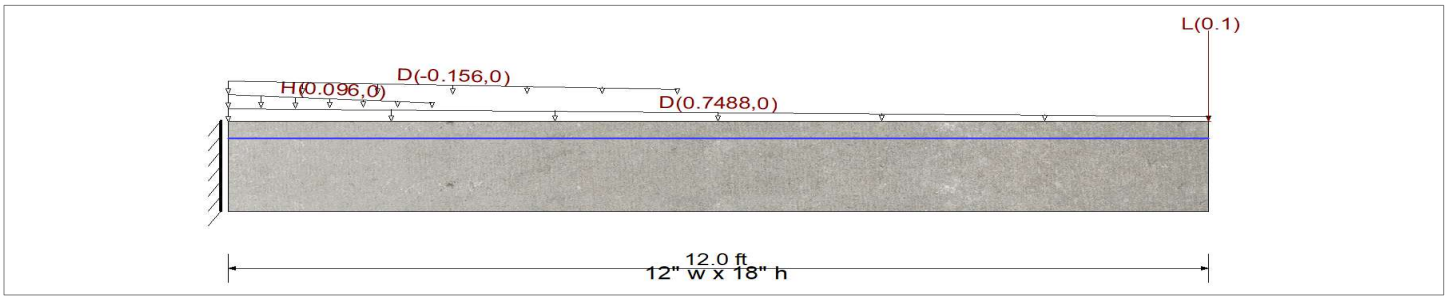
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	=	4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ *7.50	=	503.12 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.8250
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,823.68 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		
			Number of Resisting Legs Per Stirrup	=	

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.50 in from Top, from 0.0 to 12.0 ft in this span

Load for Span Number 1

Varying Uniform Load : D= 0.7488->0.0 k/ft, Extent = 0.0 -->> 12.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Driving side)

Varying Uniform Load : H= 0.0960->0.0 k/ft, Extent = 0.0 -->> 2.50 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 12.0 ft, (Impact Loading - Driving Side)

Varying Uniform Load : D= -0.1560->0.0 k/ft, Extent = 0.0 -->> 5.50 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Resisting Side)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.595**	: 1
Section used for this span	**Typical Section**	
Mu : Applied	-29.575	k-ft
Mn * Phi : Allowable	49.712	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Downward Total Deflection	0.044 in	Ratio =	6588	>=180.0	Span: 1 : +D+L+H
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+L+H

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 12 ft Floodwall Stem - Driving Side

Cross Section Strength & Inertia

Top & Bottom references are for tension side of secti

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Bottom	Phi*Mn (k-ft) Top	Moment of Inertia (in^4) I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.5",	0.00	49.71	5,832.00	37.39	895.29

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.854	
Max Upward from Load Combinations	6.854	
Max Upward from Load Cases	4.064	
D Only	4.064	
+1.60D+1.60H	6.694	
+1.60H	0.192	
+1.60D+1.60L+1.60H	6.854	

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	12.000	-29.58	49.71	0.59
+1.60D+1.60H					
Span # 1	1	12.000	-27.66	49.71	0.56
+1.60D+1.60L+1.60H					
Span # 1	1	12.000	-29.58	49.71	0.59
+1.60H					
Span # 1	1	12.000	-0.16	49.71	0.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+L+H	1	0.0437	12.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+H	1	0.0392	12.000	0.0000	0.000
+D+L+H	1	0.0437	12.000	0.0000	0.000
H Only	1	0.0001	12.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #6 $= 0.75\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.4\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 26.5\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 19.9\ kip$ *(factored shear-friction strength)*

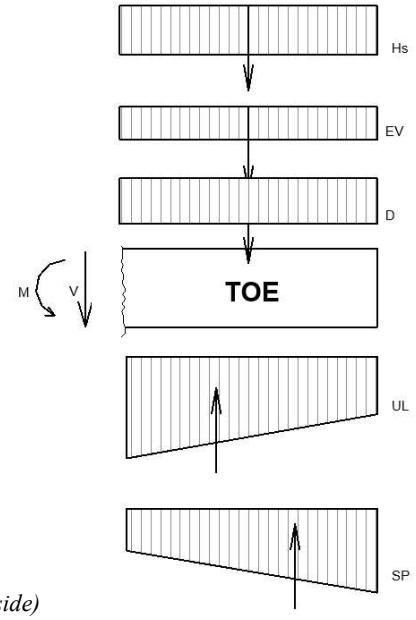
$V_{stem} := (Vu)_{stem} \cdot b = 7.3\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{“OK”} \quad 0.4]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 300\ psf$

$D := \sigma_D \cdot TWIDTH = 1.2\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 2\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 156 \\ 156 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 0.6 \\ 0.6 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 2\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 144\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.576 \\ 0.576 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 2\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -1.581 \\ -1.36 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 12.5\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 11.2 \\ 9.4 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 4\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 7.2 \\ 5.4 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -1.02 \\ -0.78 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)}{2} = \begin{bmatrix} -5.2 \\ -4.3 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 2.1 \\ 2.2 \end{bmatrix} ft$ *(moment arm from the connection of the toe to the resultant of the soil pressure loading)*

Uplift Pressures

$(\sigma_{UL})_{min} = \begin{bmatrix} 0.578 \\ 0.614 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} = \begin{bmatrix} 0.7 \\ 0.775 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$BWIDTH = 12.5\ ft$ *(foundation width)*

$TK = 3\ ft$ *(key width)*

$L := BWIDTH - TK = 9.5\ ft$ *(length of linear uplift pressure)*

$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$ *(linear relationship)*

$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0128 \\ 0.0169 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship)*

$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.629 \\ 0.682 \end{bmatrix} ksf$ *(uplift pressure at the connection of the toe to the foundation)*

$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -2.4 \\ -2.6 \end{bmatrix} klf$ *(uplift pressure resultant for the toe of the foundation)*

$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 2 \\ 2 \end{bmatrix} ft$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -8.3 \\ -7.2 \end{bmatrix} klf$ *(load case 1 for infrequent flooding)*

$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 8.3\ klf$ *(maximum of the drained vs undrained cases)*

Factored Shear on Toe at Face of Stem

$(V_u)_{toe} := \Sigma F_{toe} = 8.3\ klf$ *(maximum factored shear in toe)*

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -17.8 \\ -15.4 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$Mu_{toe} := \max(LC1) \cdot 1\ ft = -15.4\ kip \cdot ft$ *(resultant moment at base of floodwall stem per foot of wall)*

$Mu_{toe} := -Mu_{toe}$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \text{Bar Size: \#8} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 19.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$a_{toe.bot} := \dfrac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \dfrac{a_{toe.bot}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{“OK”} \quad 0.23]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \dfrac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \text{find}\left(As_{req}\right) = 0.2\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.785\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.785\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.3\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.241\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.3]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.4\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.6]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0034$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.43\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 0.98]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0019$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.2]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 288\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 31.4\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 23.5\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 8.3\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.4]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

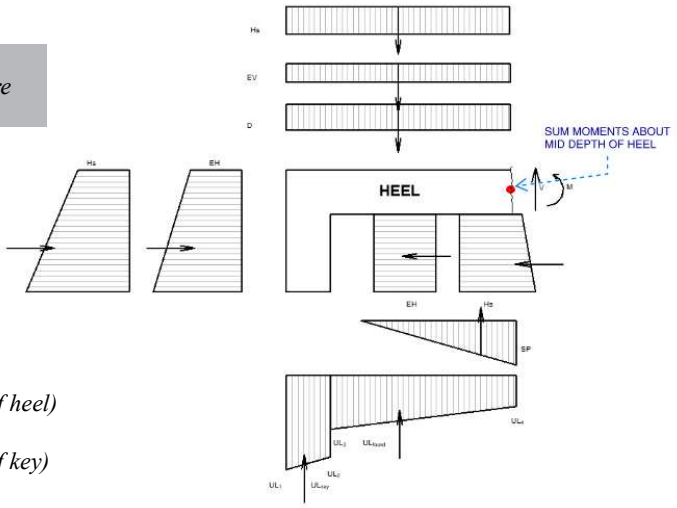
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 7\ ft$

(Length of heel, includes key)

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 300\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 300\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 2.1\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 0.9\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 3\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 3.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 5.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 4.1\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.749\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 5.2\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 3.5\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.749\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 0.998\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 4\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 3.5\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 3.1\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.85 \\ 0.933 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.889 \\ 0.984 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 2\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 1.7 \\ 1.9 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 2 \\ 2 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.076\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.199\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 4\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.6\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 3.296\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.81 \\ 1.1 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.81 \\ 1.1 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 2\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 3.6 \\ 2.2 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 2 \\ 2 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.144\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 1.01\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 3.5\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 1.3 \\ 3.08 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 5.7 \\ 3.92 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.8 \\ 0.57 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 2.29 \\ 1.11 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 1.9 \\ 1.3 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

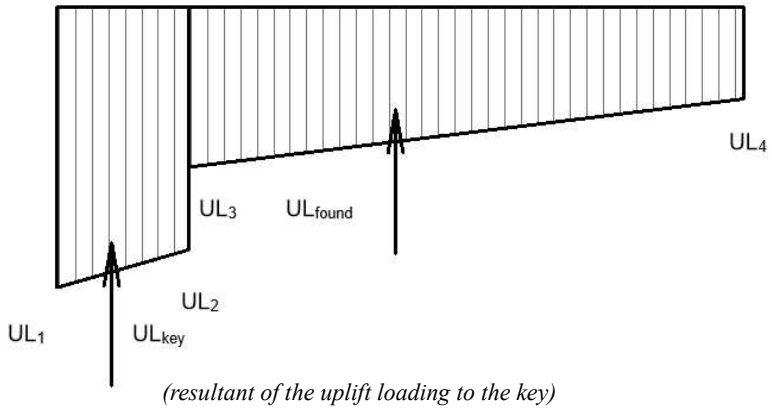
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot (UL_1 + UL_2)}{2} = \begin{bmatrix} 3.1 \\ 3.6 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 4\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot (2 \cdot UL_1 + UL_2)}{3 \cdot (UL_1 + UL_2)} \right) = \begin{bmatrix} 5.51 \\ 5.51 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{(UL_4 - UL_3)}{BWIDTH - TK} = \begin{bmatrix} -0.0333 \\ -0.0438 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.6 \\ 0.7 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot (UL_3 + UL_{stem})}{2} = \begin{bmatrix} 2.6 \\ 3.1 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot (2 \cdot UL_3 + UL_{stem})}{3 \cdot (UL_3 + UL_{stem})} \right) = \begin{bmatrix} 2.07 \\ 2.08 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left((EH)_{fh} - (EH)_{bh} \right) \\ + \left((Hs)_{fh} - (Hs)_{bh} \right) \end{pmatrix} = \begin{bmatrix} -2.1 \\ -0.1 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -0.1\ klf$

$(\Sigma F_{heel})_{horz} := \max\left(\overrightarrow{|LC1|}\right) = 2.1\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$
$= 0\ kip$

Forces on Heel - Vertical Forces

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 27.6 \\ 27.2 \end{bmatrix} klf$$

(summation of the forces in the vertical direction)

$$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 27.6\ klf$$

(maximum of the drained vs undrained cases)

Resultant Shear at Base

$$Vu_{heel} := (\Sigma F_{heel})_{vert} = 27.6\ klf$$

(factored shear at connection of floodwall heel)

Resultant Axial Load in Base

$$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 2.1\ klf$$

(factored axial load at floodwall heel)

$$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$$

(tension in heel; from check above, zero if heel in compression)

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \overrightarrow{\left(D \cdot (x_D)_{heel}\right)} \\ + \overrightarrow{\left((EV)_{ah} \cdot (x_{EV})_{ah}\right)} + \overrightarrow{\left((Hs)_{ah} \cdot (x_{Hs})_{ah}\right)} \\ + \overrightarrow{\left((EH)_{fh} \cdot (x_{EH})_{fh}\right)} + \overrightarrow{\left((Hs)_{fh} \cdot (x_{Hs})_{fh}\right)} \end{pmatrix} = 74.9\ \frac{kip \cdot ft}{ft}$$

(load case for positive bending moments applied to heel)

$$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \overrightarrow{\left((Hs)_{bh} \cdot (x_{Hs})_{bh}\right)} \\ + \overrightarrow{\left((EH)_{bh} \cdot (x_{EH})_{bh}\right)} \\ + \overrightarrow{\left(SP_{heel} \cdot (x_{SP})_{heel}\right)} \\ + \overrightarrow{\left(UL_{key} \cdot (x_{UL})_{key}\right)} \\ + \overrightarrow{\left(UL_{found} \cdot (x_{UL})_{found}\right)} \end{pmatrix} = \begin{bmatrix} 60 \\ 57.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case for negative bending moments applied to heel)

Resultant Moment at Base

$$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = 17.65\ kip$$

$$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = 17.7\ kip \cdot ft$$

(maximum factored moment at base of floodwall stem on the heel)

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$$Mu_{heel} = 17.7\ kip \cdot ft$$

(maximum factored moment at base of floodwall stem on the heel)

$$(P_u)_{heel} = 2.1\ \frac{kip}{ft}$$

(maximum factored axial tension at base of floodwall stem on the heel)

$$Vu_{heel} = 27.6\ \frac{kip}{ft}$$

(maximum factored shear at base of floodwall stem on the heel)

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 12\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$(A_s)_{heel.top} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$(A_s)_{heel.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$\rho := \dfrac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for primary tension steel)*

$\rho' := \dfrac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for bottom tension steel)*

$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 20.5\ in$ *(depth to top reinforcement in heel)*

$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{heel.top} := \dfrac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.63\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 94.2\ kip$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$\phi Pn := \phi_A \cdot Pn_{max} = 84.8\ kip$

$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”} \quad 0.02]$ *(tensile)*

Flexural Check

$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \dfrac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \dfrac{TTOE}{2}\right) \cdot b\right) = 42.3\ kip \cdot ft$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”} \quad 0.4]$

Minimum Required Steel for Flexure

Given

Constraints

Values

$As_{req} := 1.5\ in^2$

$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \dfrac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \dfrac{TTOE}{2}\right) \cdot b\right)\right)$

Solver

$As_{req.stem} := \text{find}\left(\mathbf{As_{req}}\right) = 0.207\ in^2$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0.2\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.825\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.82\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.825\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0.3\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} := \text{if } A_{s\_1.3} < (A_s)_{heel.top} \parallel as \leftarrow (A_{s.min})_{heel.top}$ else $\parallel as \leftarrow A_{s.min} = 0.207\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad 0.26]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0032$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.4\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.5]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0036$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

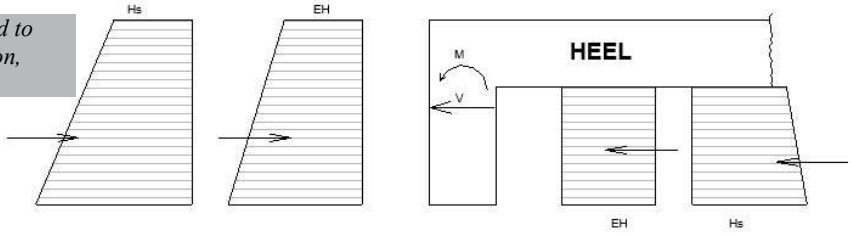
KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last

$HK = 2\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 4\ ft$ *(depth of key, from top of foundation down)*



Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.874\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 0.998\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 1.9\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 1.02\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.85 \\ 0.933 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.889 \\ 0.984 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 2\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 1.7 \\ 1.9 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 1 \\ 1 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.428\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.489\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 4\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 0.9\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 1.022\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.81 \\ 1.1 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.81 \\ 1.1 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 2\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 3.6 \\ 2.2 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 1 \\ 1 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 4.6\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 8.6 \\ 6.6 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 4 \\ 2.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 4\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 19.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.06]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Solve Constraints Values

$$As_{req} := 1.5\ in^2$$

$$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(As_{req}\right) = 0.046\ in^2$$

$$(A_{s.min})_{key.min} := As_{req.stem} = 0\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2}\right) = 0.648\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$ *(minimum area of steel required for flexural capacity)*

$$A_{s.min} = 0.785\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0.1\ in^2$$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$$(A_s)_{key.req} := \begin{array}{l} \text{if } A_{s\_1.3} < (A_s)_{key.main} \\ \quad as \leftarrow (A_{s.min})_{key.min} \\ \text{else} \\ \quad as \leftarrow A_{s.min} \end{array} = 0.046\ in^2$$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{key.main} = 0.8\ in^2$$ *(area of steel provided in the horizontal direction for the heel)*

$$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.1]$$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Requirements

$$(A_{s.min})_{temp} = 0.6\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.8]$$

(temperature and shrinkage steel of key, check utilization ratio)

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$ *(reinforcement ratio from EM 1110-2-2104)*

$$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main}\right)} = 0.0034$$ *(actual reinforcement ratio for concrete section)*

$$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.4]$$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{key.lat} := \text{Bar Size: \#6} = 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in key)*

$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in key)*

$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 30.6\ in$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.648\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.6\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{"OK"} \quad 0.7]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.lat}}{\left(b \cdot d_{key.main}\right)} = 0.0038$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

12 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS := 920.12\ ft$

$HTS := 14\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 5.5\ ft$

$TTOE := 2\ ft$

$TWIDTH := 6\ ft$

$BWIDTH := 18\ ft$

$HK := 3.5\ ft$

$TK := 2\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 920.31\ ft$

$HTS := 10\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 5.5\ ft$

$TTOE := 2\ ft$

$TWIDTH := 2.5\ ft$

$BWIDTH := 13\ ft$

$HK := 3.5\ ft$

$TK := 3\ ft$

$Elev_{soil.d} := 912\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 912\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 910.3\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 908.3\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 1.7\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 1.7\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

RS36+75 RB28+40 Alternate 10ft Wall - Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data
Date: 2025/11/ 8 Time: 7.44.51

Structural geometry data:
Elevation of top of stem (ELTS) = 920.31 ft
Height of stem (HTS) = 10.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 2.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 13.00 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	904.81
0.00	910.31
9.00	910.31
9.00	920.31
10.50	920.31
10.50	910.31
13.00	910.31
13.00	908.31
3.00	908.31
3.00	904.81

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-0.50	10.00
2	-6.00	24.00
3	-1.26	500.00

UNDRAINED CASE

RS36+75 RB28+40 Alternate 10ft Wall - Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/11/ 8 Time: 7.41.33

Structural geometry data:
Elevation of top of stem (ELTS) = 920.31 ft
Height of stem (HTS) = 10.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 2.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 13.00 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	904.81
0.00	910.31
9.00	910.31
9.00	920.31
10.50	920.31
10.50	910.31
13.00	910.31
13.00	908.31
3.00	908.31
3.00	904.81

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-0.50	10.00
2	-6.00	24.00
3	-1.26	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1025.00	847.08
2	-525.00	847.08
3	-25.00	899.58
4	-1.00	911.58
5	9.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	912.00	0.16

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	10.50	912.00
2	510.50	918.67

Foundation property data:
- phi for soil-structure interface = 28.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 28.00 (deg)
- c for soil-soil interface = 0.000 (ksf)

Water data:
- Driving side elevation = 920.17 ft
- Resisting side elevation = 911.82 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.17	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used*
in the overturning analysis.*

*Forces on the resisting side *are used* in the sliding analysis.*

**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1025.00	847.08
2	-525.00	847.08
3	-25.00	899.58
4	-1.00	911.58
5	9.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	912.00	0.16

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	10.50	912.00
2	510.50	918.67

Foundation property data:
- phi for soil-structure interface = 0.00 (deg)
- c for soil-structure interface = 1.000 (ksf)
- phi for soil-soil interface = 0.00 (deg)
- c for soil-soil interface = 1.000 (ksf)

Water data:
- Driving side elevation = 920.17 ft
- Resisting side elevation = 911.82 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.17	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack *is* down to bottom of heel
- o Computed cracks *will* be filled with water

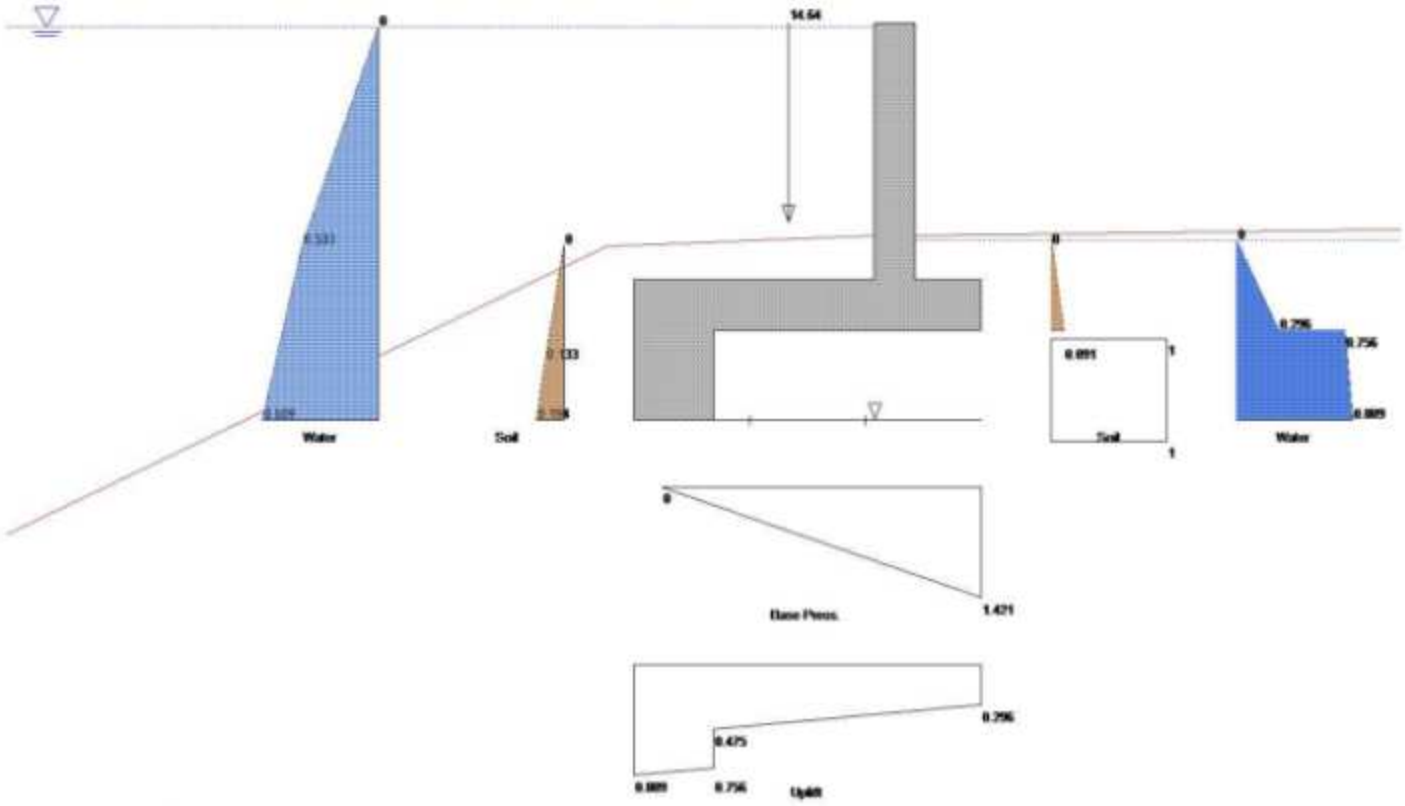
Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used*
in the overturning analysis.*

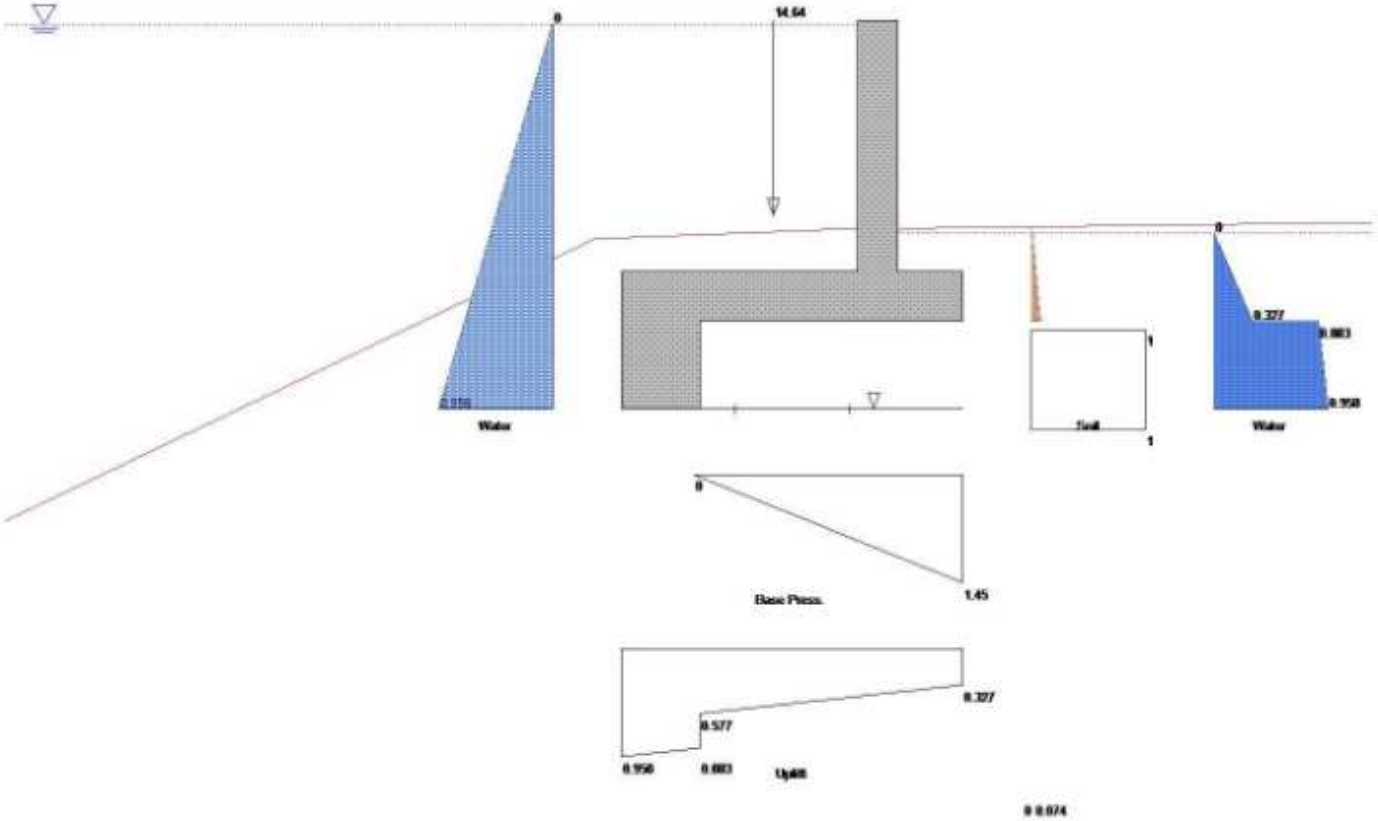
*Forces on the resisting side *are used* in the sliding analysis.*

**Do* iterate in overturning analysis.*

CT Wall - Drained Case Output



CT Wall - Undrained Case Output



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.194 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.0906 \\ 0.074 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 1.421 \\ 1.45 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 0.809 \\ 0.958 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.475 \\ 0.577 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.296 \\ 0.327 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.296 \\ 0.327 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.475 \\ 0.577 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

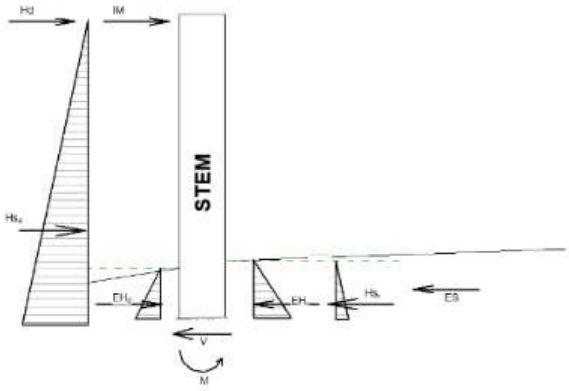
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 0.809 \\ 0.959 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 624 \\ 624 \end{bmatrix} psf$ *(hydrostatic pressure loading to driving side)*

$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 3.12 \\ 3.12 \end{bmatrix} klf$ *(hydrostatic loading to driving side)*

$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 3.3\ ft$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 105.5 \\ 105.5 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(hydrostatic loading to resisting side)*

$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.56\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.046 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side of stem)*

$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to driving side of stem)*

$(x_{EH})_d := HSD \cdot \frac{1}{3} = 0.6\ ft$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.041 \\ 0.034 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side of stem)*

$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to resisting side of stem)*

$(x_{EH})_r := HSR \cdot \frac{1}{3} = 0.6\ ft$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

$(x_{IM})_d := HTS = 10\ ft$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$p_{const} := 250\ psf$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$d_{soil} := HSR = 1.7\ ft$ *(depth of soil cover on stem)*

$(x_Q)_d := \frac{HSR}{2} = 0.8\ ft$ *(resultant height above base of stem)*

$x_Q := d_{soil} - (x_Q)_d = 0.8\ ft$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.224\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 8.3\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 8.3\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.145\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.279\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 5.8\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 2.3\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.1\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.1\ klf$

$(x_{EQ}) := HSR = 1.7\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Hs_d + IM) = \begin{bmatrix} 5.2 \\ 5.2 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot (EH_d + EQ) = \begin{bmatrix} 0.3 \\ 0.2 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Q) = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot (EH_r + Hs_r - W_c) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 5.2 \\ 5.2 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 5.2\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Hs_d \cdot (x_{Hs})_d \right) + IM \cdot (x_{IM})_d \right) = \begin{bmatrix} 18.3 \\ 18.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(EQ \cdot (x_{EQ}) \right) \right) = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Q \cdot \left((x_Q)_d \right) \right) \right) = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) + \left(Hs_r \cdot (x_{Hs})_r \right) \right) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0.03 \\ 0.03 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0.03 \\ 0.03 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 18.2 \\ 18.1 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 18.2\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v\left(X_U, X_N\right) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left(X_U\right) \\ \quad \left\| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left\| ch_{i,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check\left(X_U, X_N\right) := \left\| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left\| ch_{1,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)

$$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$$

(nominal flexural strength, EM1110-2-2104, B-49)

$$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{"OK"} \quad 0.37]$$

(nominal flexural check, with Utilization ratio)

Minimum Required Steel for Flexure

Solve Constraints Given Values

$$As_{req} := 1.5\ in^2$$

$$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.284\ in^2$$

$$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.284\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.584\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.379\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{stem.req} := \begin{cases} as \leftarrow (A_{s.min})_{stem.ten} & \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \\ as \leftarrow A_{s.min} & \text{else} \end{cases} = 0.284\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{stem.vert} = 0.8\ in^2$$

(area of steel provided in the vertical direction for the stem)

$$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 10 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 10 ft Floodwall Stem

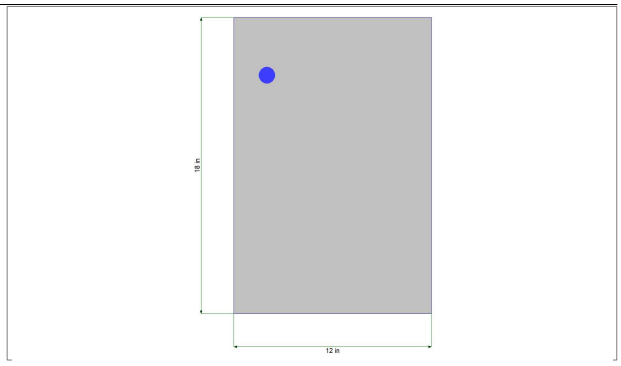
CODE REFERENCES

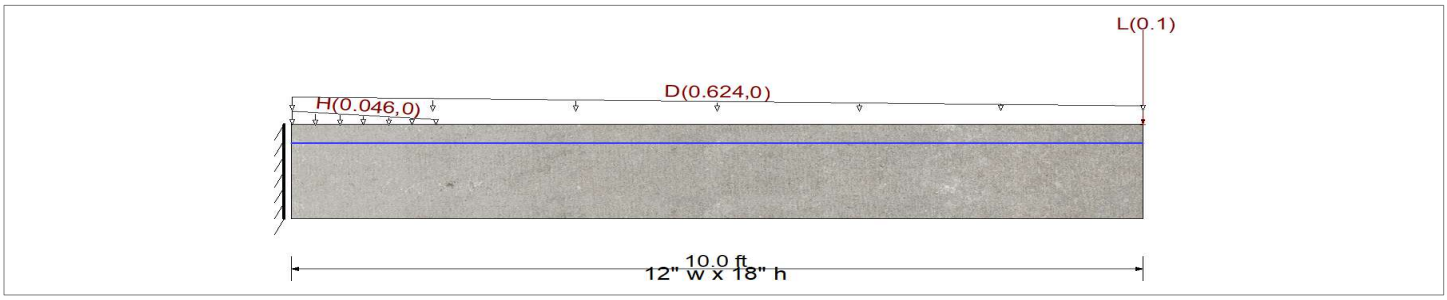
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	= 4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 *7.50	= 503.12 psi		Shear :	0.750
ψ Density	= 150.0 pcf	β 1	=	0.8250
λ LtWt Factor	= 1.0			
Elastic Modulus	= 3,823.68 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		
		Number of Resisting Legs Per Stirrup	=	

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.50 in from Top, from 0.0 to 10.0 ft in this span

Load for Span Number 1
Varying Uniform Load : H= 0.0460->0.0 k/ft, Extent = 0.0 -->> 1.70 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 10.0 ft, (Impact Loading - Driving Side)
Varying Uniform Load : D= 0.6240->0.0 k/ft, Extent = 0.0 -->> 10.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Resisting Side)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.368** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-18.275	k-ft
Mn * Phi : Allowable	49.712	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	
Max Downward Total Deflection	0.019 in	Ratio =	12828 >=180.0	Span: 1 : +D+L+H
Max Upward Total Deflection	0.000 in	Ratio =	0 <180.0	Span: 1 : +D+L+H

Cross Section Strength & Inertia

Top & Bottom references are for tension side of secti

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.5",	0.00	49.71	5,832.00	37.39	895.29

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 10 ft Floodwall Stem

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	5.215	
Max Upward from Load Combinations	5.215	
Max Upward from Load Cases	3.120	
D Only	3.120	
+1.60D+1.60H	5.055	
+1.60H	0.063	
+1.60D+1.60L+1.60H	5.215	

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	10.000	-18.28	49.71	0.37
+1.40D+1.60H					
Span # 1	1	10.000	-14.60	49.71	0.29
+1.60D+1.60H					
Span # 1	1	10.000	-16.68	49.71	0.34
+1.60D+1.60L+1.60H					
Span # 1	1	10.000	-18.28	49.71	0.37
+1.60H					
Span # 1	1	10.000	-0.04	49.71	0.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+L+H	1	0.0187	10.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+H	1	0.0161	10.000	0.0000	0.000
+D+L+H	1	0.0187	10.000	0.0000	0.000
H Only	1	0.0000	10.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #6 $= 0.75\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.4\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 26.5\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 19.9\ kip$ *(factored shear-friction strength)*

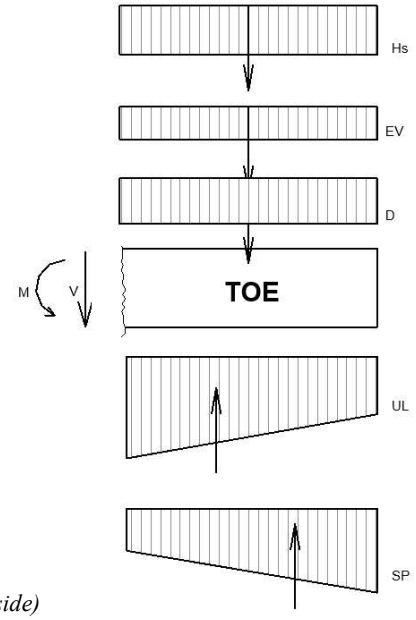
$V_{stem} := (Vu)_{stem} \cdot b = 5.2\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{"OK"} \quad 0.3]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 300\ psf$

$D := \sigma_D \cdot TWIDTH = 0.8\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 1.3\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 105.5 \\ 105.5 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 1.25\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 97.3\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.243 \\ 0.243 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 1.3\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -1.421 \\ -1.45 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 13\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 11.6 \\ 9.8 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 2.5\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 9.1 \\ 7.3 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -1.12 \\ -1.08 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)}{2} = \begin{bmatrix} -3.2 \\ -3.2 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 1.3 \\ 1.3 \end{bmatrix} ft$$

(moment arm from the connection of the toe to the resultant of the soil pressure loading)

Uplift Pressures

$$(\sigma_{UL})_{min} = \begin{bmatrix} 0.296 \\ 0.327 \end{bmatrix} ksf$$

(minimum uplift pressure at the end of the toe)

$$(\sigma_{UL})_{max} = \begin{bmatrix} 0.475 \\ 0.577 \end{bmatrix} ksf$$

(maximum uplift pressure behind the key)

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$BWIDTH = 13\ ft$ *(foundation width)*

$TK = 3\ ft$ *(key width)*

$L := BWIDTH - TK = 10\ ft$ *(length of linear uplift pressure)*

$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$ *(linear relationship)*

$$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0179 \\ 0.025 \end{bmatrix} \frac{ksf}{ft}$$

(linear relationship)

$$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.341 \\ 0.39 \end{bmatrix} ksf$$

(uplift pressure at the connection of the toe to the foundation)

$$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -0.8 \\ -0.9 \end{bmatrix} klf$$

(uplift pressure resultant for the toe of the foundation)

$$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 1.2 \\ 1.2 \end{bmatrix} ft$$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -4.3 \\ -4.5 \end{bmatrix} klf$$

(load case 1 for infrequent flooding)

$$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 4.5\ klf$$

(maximum of the drained vs undrained cases)

Factored Shear on Toe at Face of Stem

$$(V_u)_{toe} := \Sigma F_{toe} = 4.5\ klf$$

(maximum factored shear in toe)

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -5.6 \\ -5.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$$Mu_{toe} := \max(LC1) \cdot 1\ ft = -5.6\ kip \cdot ft$$

(resultant moment at base of floodwall stem per foot of wall)

$$Mu_{toe} := -Mu_{toe}$$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \text{Bar Size: \#8} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 19.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$a_{toe.bot} := \dfrac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \dfrac{a_{toe.bot}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{“OK”} \quad 0.08]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \dfrac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \text{find}\left(As_{req}\right) = 0.1\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.785\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.785\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.087\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.1]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.4\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.6]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0034$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.43\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 0.98]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0019$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.2]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 288\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 31.4\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 23.5\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 4.5\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.2]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

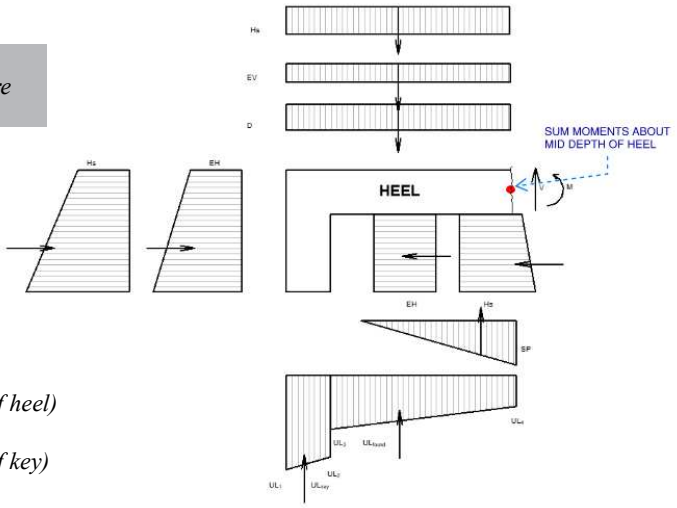
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 9\ ft$

(Length of heel, includes key)

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 300\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 525\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 2.7\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 1.6\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 4.3\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 4.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 7.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 5.6\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.624\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 5.6\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 4.5\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.624\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 0.967\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 4.4\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 3.95\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.809 \\ 0.959 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 2.7 \\ 3.2 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 2.8 \\ 2.8 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.052\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.22\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.7\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 4.318\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.2 \\ 1.01 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.2 \\ 1.01 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 4.2 \\ 3.5 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 2.8 \\ 2.8 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.097\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 0.88\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 4.5\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 1.35 \\ 3.2 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 7.65 \\ 5.8 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.93 \\ 0.86 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 3.57 \\ 2.49 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 2.5 \\ 1.9 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

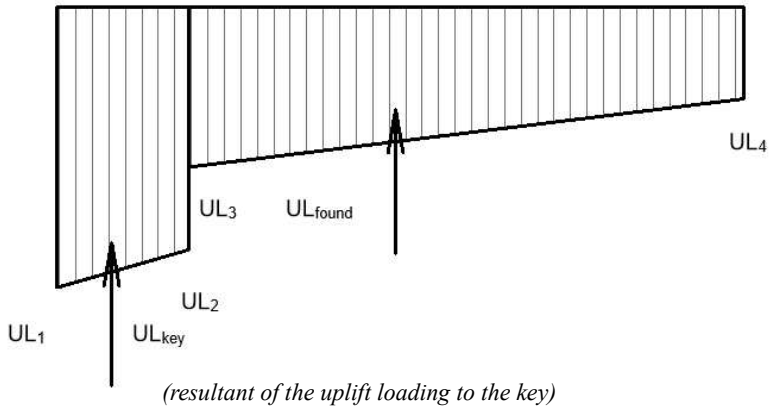
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot (UL_1 + UL_2)}{2} = \begin{bmatrix} 3.1 \\ 3.6 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 6\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot (2 \cdot UL_1 + UL_2)}{3 \cdot (UL_1 + UL_2)} \right) = \begin{bmatrix} 7.51 \\ 7.51 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{(UL_4 - UL_3)}{BWIDTH - TK} = \begin{bmatrix} -0.0316 \\ -0.0416 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.5 \\ 0.6 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot (UL_3 + UL_{stem})}{2} = \begin{bmatrix} 3.8 \\ 4.4 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot (2 \cdot UL_3 + UL_{stem})}{3 \cdot (UL_3 + UL_{stem})} \right) = \begin{bmatrix} 3.15 \\ 3.17 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left((EH)_{fh} - (EH)_{bh} \right) \\ + \left((Hs)_{fh} - (Hs)_{bh} \right) \end{pmatrix} = \begin{bmatrix} -2.9 \\ -2.6 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -2.6\ klf$

$(\Sigma F_{heel})_{horz} := \max\left(\overrightarrow{|LC1|}\right) = 2.9\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$ $= 0\ kip$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$

Forces on Heel - Vertical Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 33.9 \\ 33.9 \end{bmatrix} klf$ *(summation of the forces in the vertical direction)*

$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 33.9\ klf$ *(maximum of the drained vs undrained cases)*

Resultant Shear at Base

$Vu_{heel} := (\Sigma F_{heel})_{vert} = 33.9\ klf$ *(factored shear at connection of floodwall heel)*

Resultant Axial Load in Base

$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 2.9\ klf$ *(factored axial load at floodwall heel)*

$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$ *(tension in heel; from check above, zero if heel in compression)*

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{D \cdot (x_D)_{heel}}\right) \\ + \left(\overrightarrow{(EV)_{ah} \cdot (x_{EV})_{ah}}\right) + \left(\overrightarrow{(Hs)_{ah} \cdot (x_{Hs})_{ah}}\right) \\ + \left(\overrightarrow{(EH)_{fh} \cdot (x_{EH})_{fh}}\right) + \left(\overrightarrow{(Hs)_{fh} \cdot (x_{Hs})_{fh}}\right) \end{pmatrix} = 117.9\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{(Hs)_{bh} \cdot (x_{Hs})_{bh}}\right) \\ + \left(\overrightarrow{(EH)_{bh} \cdot (x_{EH})_{bh}}\right) \\ + \left(\overrightarrow{SP_{heel} \cdot (x_{SP})_{heel}}\right) \\ + \left(\overrightarrow{UL_{key} \cdot (x_{UL})_{key}}\right) \\ + \left(\overrightarrow{UL_{found} \cdot (x_{UL})_{found}}\right) \end{pmatrix} = \begin{bmatrix} 101.3 \\ 102.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = 16.61\ kip$

$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = 16.6\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$Mu_{heel} = 16.6\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

$(P_u)_{heel} = 2.9\ \frac{kip}{ft}$ *(maximum factored axial tension at base of floodwall stem on the heel)*

$Vu_{heel} = 33.9\ \frac{kip}{ft}$ *(maximum factored shear at base of floodwall stem on the heel)*

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 12\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$(A_s)_{heel.top} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$(A_s)_{heel.bot} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$\rho := \frac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for primary tension steel)*

$\rho' := \frac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for bottom tension steel)*

$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 20.5\ in$ *(depth to top reinforcement in heel)*

$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{heel.top} := \frac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.64\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 94.2\ kip$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$\phi Pn := \phi_A \cdot Pn_{max} = 84.8\ kip$

$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”} \quad 0.03]$ *(tensile)*

Flexural Check

$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \frac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right) = 42.5\ kip \cdot ft$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”} \quad 0.4]$

Minimum Required Steel for Flexure

Given

Constraints

Values

$As_{req} := 1.5\ in^2$

$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \frac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right)\right)$

Solver

$As_{req.stem} := \text{find}\left(\mathbf{As_{req}}\right) = 0.201\ in^2$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0.2\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.825\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.82\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.825\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0.3\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} := \text{if } A_{s\_1.3} < (A_s)_{heel.top} \parallel as \leftarrow (A_{s.min})_{heel.top}$ else $\parallel as \leftarrow A_{s.min} = 0.201\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad 0.26]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0032$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.4\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.5]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0036$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

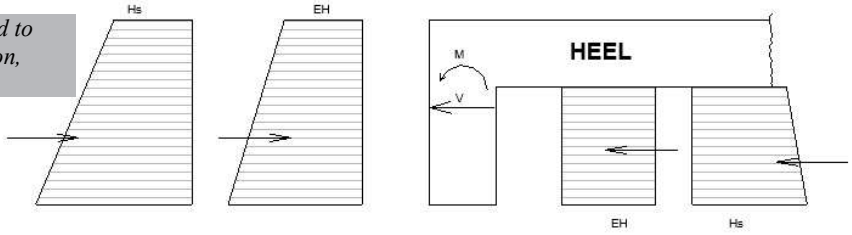
KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last

$HK = 3.5\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 5.5\ ft$ *(depth of key, from top of foundation down)*



Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.749\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 0.967\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 3\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 1.82\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.809 \\ 0.959 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 2.7 \\ 3.2 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot (2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 1.8 \\ 1.8 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.367\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.474\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 1.5\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 1.824\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 4.2 \\ 3.5 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 1.8 \\ 1.8 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 13.1 \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 19.5 \\ 19 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 6.5 \\ 6 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 6.5\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 19.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.1]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Solve Constraints Values

$$As_{req} := 1.5\ in^2$$

$$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}}\right)$$

$$As_{req.stem} := \text{find}\left(As_{req}\right) = 0.074\ in^2$$

$$(A_{s.min})_{key.min} := As_{req.stem} = 0.1\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2}\right) = 0.648\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$ *(minimum area of steel required for flexural capacity)*

$$A_{s.min} = 0.785\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0.1\ in^2$$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$$(A_s)_{key.req} := \begin{array}{l} \text{if } A_{s\_1.3} < (A_s)_{key.main} \\ \quad as \leftarrow (A_{s.min})_{key.min} \\ \text{else} \\ \quad as \leftarrow A_{s.min} \end{array} = 0.074\ in^2$$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{key.main} = 0.8\ in^2$$ *(area of steel provided in the horizontal direction for the heel)*

$$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.1]$$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Requirements

$$(A_{s.min})_{temp} = 0.6\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.8]$$

(temperature and shrinkage steel of key, check utilization ratio)

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$ *(reinforcement ratio from EM 1110-2-2104)*

$$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main}\right)} = 0.0034$$ *(actual reinforcement ratio for concrete section)*

$$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.4]$$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{key.lat} := \text{Bar Size: \#6} = 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in key)*

$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in key)*

$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 30.6\ in$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.648\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.6\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{"OK"} \quad 0.7]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.lat}}{\left(b \cdot d_{key.main}\right)} = 0.0038$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

10 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS = 920.3\ ft$

$HTS = 10\ ft$

$TTS = 1.5\ ft$

$TBS = 1.5\ ft$

$THEEL = 5.5\ ft$

$TTOE = 2\ ft$

$TWIDTH = 2.5\ ft$

$BWIDTH = 13\ ft$

$HK = 3.5\ ft$

$TK = 3\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 920.31\ ft$

$HTS := 10\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 5.5\ ft$

$TTOE := 2\ ft$

$TWIDTH := 2.5\ ft$

$BWIDTH := 13\ ft$

$HK := 3.5\ ft$

$TK := 3\ ft$

$Elev_{soil.d} := 912\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 912\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 910.3\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 908.3\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 1.7\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 1.7\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

RS36+75 RB28+40 Alternate 10ft Wall - Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data
Date: 2025/11/ 8 Time: 7.44.51

Structural geometry data:
Elevation of top of stem (ELTS) = 920.31 ft
Height of stem (HTS) = 10.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 2.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 13.00 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	904.81
0.00	910.31
9.00	910.31
9.00	920.31
10.50	920.31
10.50	910.31
13.00	910.31
13.00	908.31
3.00	908.31
3.00	904.81

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-0.50	10.00
2	-6.00	24.00
3	-1.26	500.00

UNDRAINED CASE

RS36+75 RB28+40 Alternate 10ft Wall - Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/11/ 8 Time: 7.41.33

Structural geometry data:
Elevation of top of stem (ELTS) = 920.31 ft
Height of stem (HTS) = 10.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 2.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 13.00 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	904.81
0.00	910.31
9.00	910.31
9.00	920.31
10.50	920.31
10.50	910.31
13.00	910.31
13.00	908.31
3.00	908.31
3.00	904.81

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-0.50	10.00
2	-6.00	24.00
3	-1.26	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1025.00	847.08
2	-525.00	847.08
3	-25.00	899.58
4	-1.00	911.58
5	9.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	912.00	0.16

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	10.50	912.00
2	510.50	918.67

Foundation property data:
- phi for soil-structure interface = 28.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 28.00 (deg)
- c for soil-soil interface = 0.000 (ksf)

Water data:
- Driving side elevation = 920.17 ft
- Resisting side elevation = 911.82 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.17	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used*
in the overturning analysis.*

*Forces on the resisting side *are used* in the sliding analysis.*

**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1025.00	847.08
2	-525.00	847.08
3	-25.00	899.58
4	-1.00	911.58
5	9.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	912.00	0.16

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	10.50	912.00
2	510.50	918.67

Foundation property data:
- phi for soil-structure interface = 0.00 (deg)
- c for soil-structure interface = 1.000 (ksf)
- phi for soil-soil interface = 0.00 (deg)
- c for soil-soil interface = 1.000 (ksf)

Water data:
- Driving side elevation = 920.17 ft
- Resisting side elevation = 911.82 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.17	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack *is* down to bottom of heel
- o Computed cracks *will* be filled with water

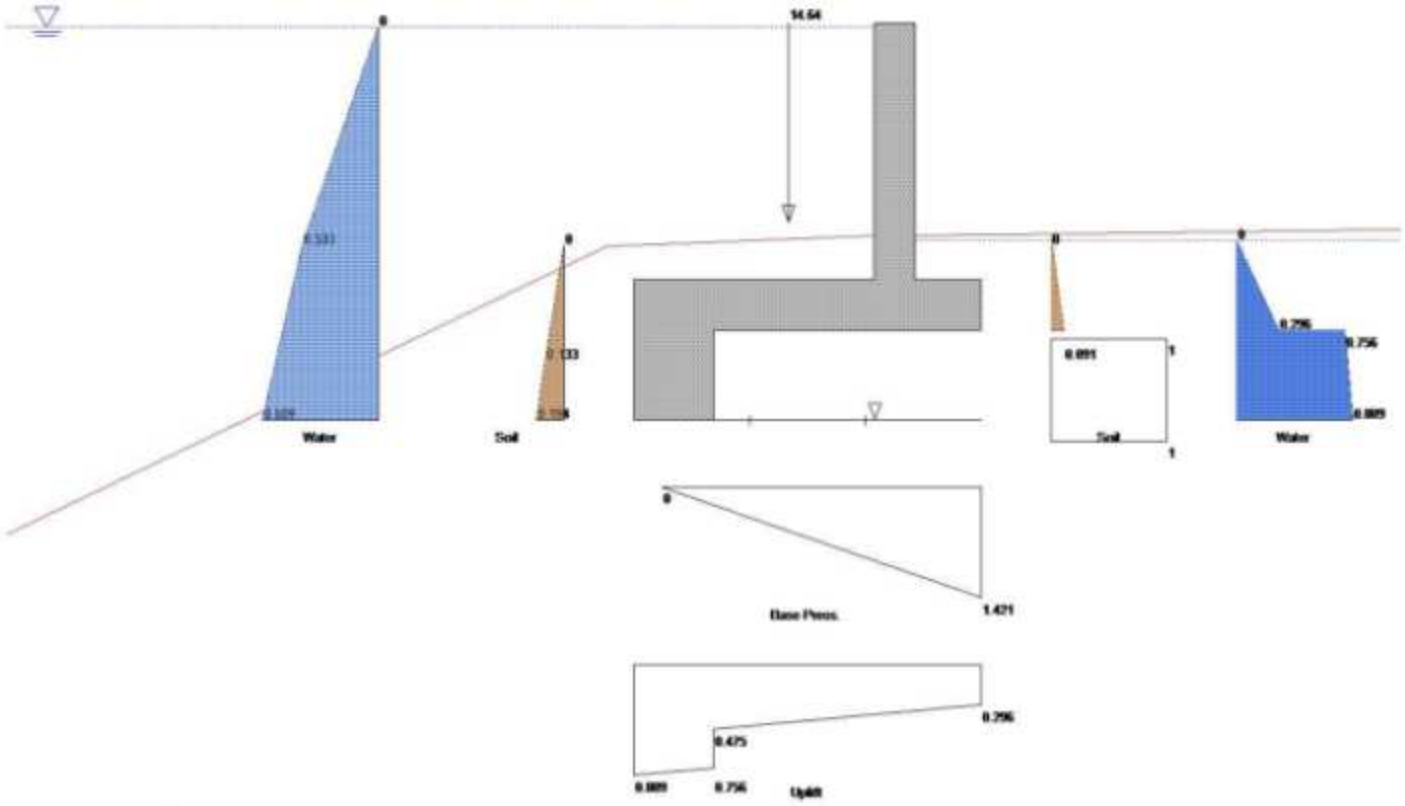
Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used*
in the overturning analysis.*

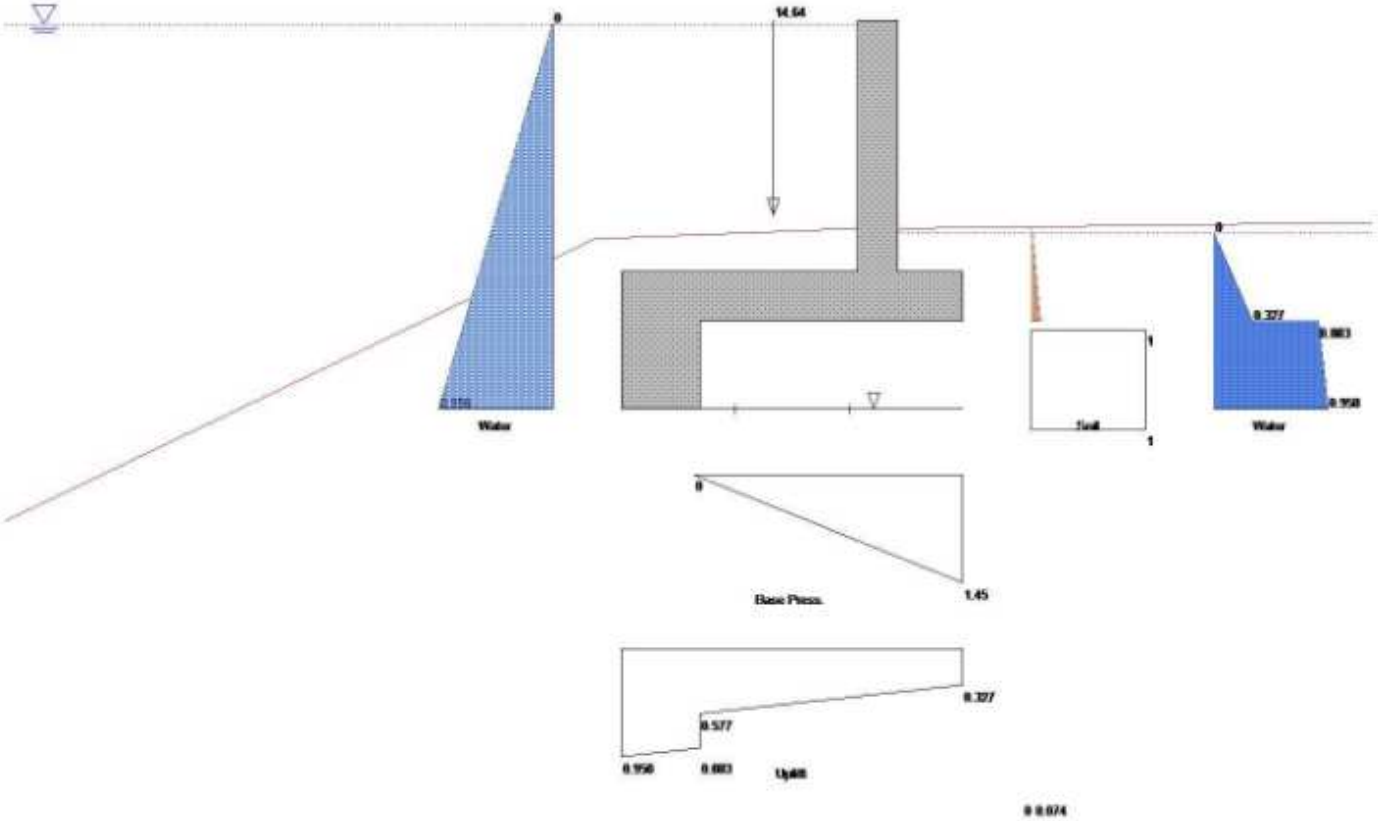
*Forces on the resisting side *are used* in the sliding analysis.*

**Do* iterate in overturning analysis.*

CT Wall - Drained Case Output



CT Wall - Undrained Case Output



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.194 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.0906 \\ 0.074 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 1.421 \\ 1.45 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 0.809 \\ 0.958 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.475 \\ 0.577 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.296 \\ 0.327 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.296 \\ 0.327 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.475 \\ 0.577 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

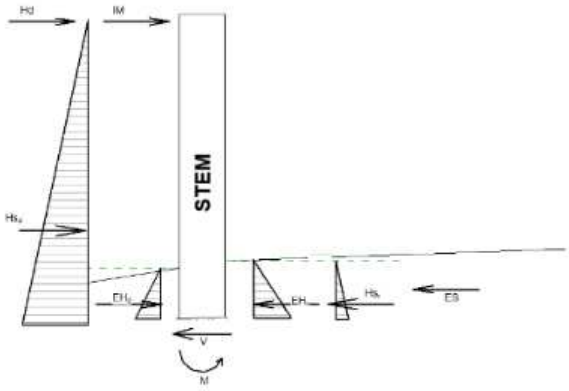
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 0.809 \\ 0.959 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 624 \\ 624 \end{bmatrix} psf$ *(hydrostatic pressure loading to driving side)*

$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 3.12 \\ 3.12 \end{bmatrix} klf$ *(hydrostatic loading to driving side)*

$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 3.3\ ft$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 105.5 \\ 105.5 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(hydrostatic loading to resisting side)*

$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.56\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.046 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side of stem)*

$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to driving side of stem)*

$(x_{EH})_d := HSD \cdot \frac{1}{3} = 0.6\ ft$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.041 \\ 0.034 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side of stem)*

$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to resisting side of stem)*

$(x_{EH})_r := HSR \cdot \frac{1}{3} = 0.6\ ft$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

$(x_{IM})_d := HTS = 10\ ft$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$p_{const} := 250\ psf$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$d_{soil} := HSR = 1.7\ ft$ *(depth of soil cover on stem)*

$(x_Q)_d := \frac{HSR}{2} = 0.8\ ft$ *(resultant height above base of stem)*

$x_Q := d_{soil} - (x_Q)_d = 0.8\ ft$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.224\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 8.3\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 8.3\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.145\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.279\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 5.8\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 2.3\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.1\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.1\ klf$

$(x_{EQ}) := HSR = 1.7\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Hs_d + IM) = \begin{bmatrix} 5.2 \\ 5.2 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot (EH_d + EQ) = \begin{bmatrix} 0.3 \\ 0.2 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Q) = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot (EH_r + Hs_r - W_c) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 5.2 \\ 5.2 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 5.2\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Hs_d \cdot (x_{Hs})_d \right) + IM \cdot (x_{IM})_d \right) = \begin{bmatrix} 18.3 \\ 18.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(EQ \cdot (x_{EQ}) \right) \right) = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Q \cdot \left((x_Q)_d \right) \right) \right) = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) + \left(Hs_r \cdot (x_{Hs})_r \right) \right) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 18.2 \\ 18.1 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 18.2\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v\left(X_U, X_N\right) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left(X_U\right) \\ \quad \left\| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left\| ch_{i,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check\left(X_U, X_N\right) := \left\| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left\| ch_{1,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)

$$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$$

(nominal flexural strength, EM1110-2-2104, B-49)

$$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{"OK"} \quad 0.37]$$

(nominal flexural check, with Utilization ratio)

Minimum Required Steel for Flexure

Solve Constraints Given Values

$$As_{req} := 1.5\ in^2$$

$$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.284\ in^2$$

$$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.284\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.584\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.379\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{stem.req} := \begin{cases} as \leftarrow (A_{s.min})_{stem.ten} & \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \\ as \leftarrow A_{s.min} & \text{else} \end{cases} = 0.284\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{stem.vert} = 0.8\ in^2$$

(area of steel provided in the vertical direction for the stem)

$$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 10 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 10 ft Floodwall Stem

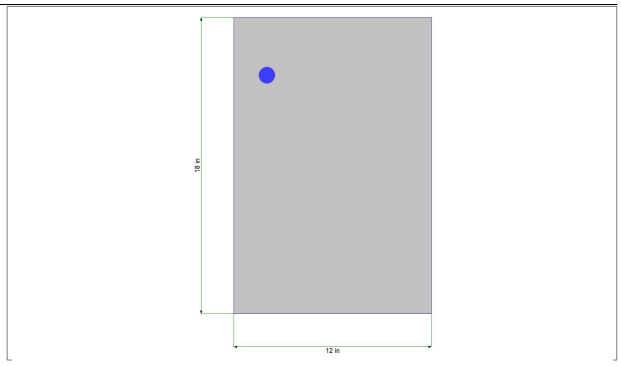
CODE REFERENCES

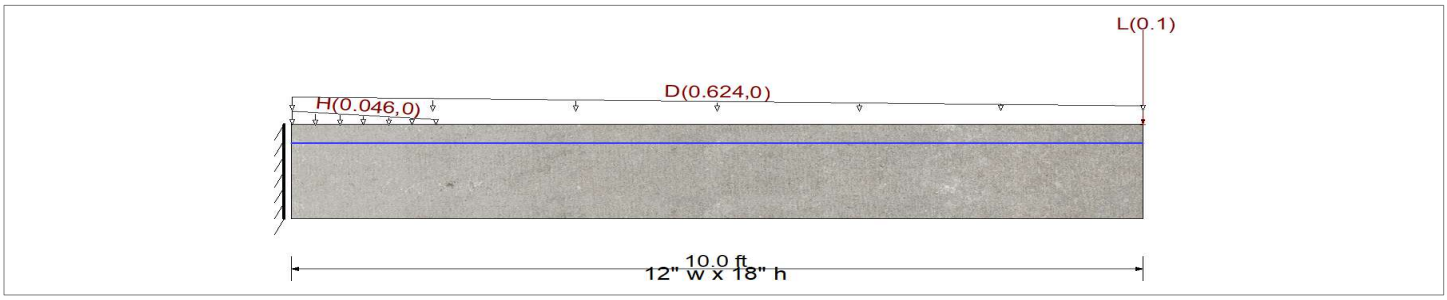
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	= 4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 *7.50	= 503.12 psi		Shear :	0.750
ψ Density	= 150.0 pcf	β 1	=	0.8250
λ LtWt Factor	= 1.0			
Elastic Modulus	= 3,823.68 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		
		Number of Resisting Legs Per Stirrup	=	

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.50 in from Top, from 0.0 to 10.0 ft in this span

Load for Span Number 1
Varying Uniform Load : H= 0.0460->0.0 k/ft, Extent = 0.0 -->> 1.70 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 10.0 ft, (Impact Loading - Driving Side)
Varying Uniform Load : D= 0.6240->0.0 k/ft, Extent = 0.0 -->> 10.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Resisting Side)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.368** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-18.275	k-ft
Mn * Phi : Allowable	49.712	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	
Max Downward Total Deflection	0.019 in	Ratio =	12828 >=180.0	Span: 1 : +D+L+H
Max Upward Total Deflection	0.000 in	Ratio =	0 <180.0	Span: 1 : +D+L+H

Cross Section Strength & Inertia

Top & Bottom references are for tension side of secti

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.5",	0.00	49.71	5,832.00	37.39	895.29

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 10 ft Floodwall Stem

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	5.215	
Max Upward from Load Combinations	5.215	
Max Upward from Load Cases	3.120	
D Only	3.120	
+1.60D+1.60H	5.055	
+1.60H	0.063	
+1.60D+1.60L+1.60H	5.215	

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	10.000	-18.28	49.71	0.37
+1.40D+1.60H					
Span # 1	1	10.000	-14.60	49.71	0.29
+1.60D+1.60H					
Span # 1	1	10.000	-16.68	49.71	0.34
+1.60D+1.60L+1.60H					
Span # 1	1	10.000	-18.28	49.71	0.37
+1.60H					
Span # 1	1	10.000	-0.04	49.71	0.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+L+H	1	0.0187	10.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+H	1	0.0161	10.000	0.0000	0.000
+D+L+H	1	0.0187	10.000	0.0000	0.000
H Only	1	0.0000	10.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #6 $= 0.75\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.4\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 26.5\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 19.9\ kip$ *(factored shear-friction strength)*

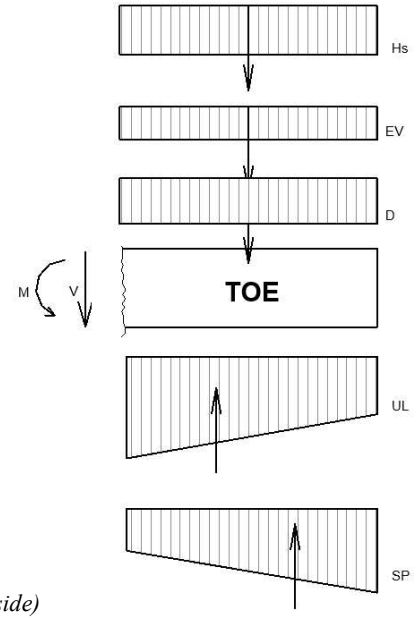
$V_{stem} := (Vu)_{stem} \cdot b = 5.2\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{"OK"} \quad 0.3]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 300\ psf$

$D := \sigma_D \cdot TWIDTH = 0.8\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 1.3\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 105.5 \\ 105.5 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 1.25\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 97.3\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.243 \\ 0.243 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 1.3\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -1.421 \\ -1.45 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 13\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 11.6 \\ 9.8 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 2.5\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 9.1 \\ 7.3 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -1.12 \\ -1.08 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)}{2} = \begin{bmatrix} -3.2 \\ -3.2 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 1.3 \\ 1.3 \end{bmatrix} ft$$

(moment arm from the connection of the toe to the resultant of the soil pressure loading)

Uplift Pressures

$$(\sigma_{UL})_{min} = \begin{bmatrix} 0.296 \\ 0.327 \end{bmatrix} ksf$$

(minimum uplift pressure at the end of the toe)

$$(\sigma_{UL})_{max} = \begin{bmatrix} 0.475 \\ 0.577 \end{bmatrix} ksf$$

(maximum uplift pressure behind the key)

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$BWIDTH = 13\ ft$ *(foundation width)*

$TK = 3\ ft$ *(key width)*

$L := BWIDTH - TK = 10\ ft$ *(length of linear uplift pressure)*

$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$ *(linear relationship)*

$$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0179 \\ 0.025 \end{bmatrix} \frac{ksf}{ft}$$

(linear relationship)

$$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.341 \\ 0.39 \end{bmatrix} ksf$$

(uplift pressure at the connection of the toe to the foundation)

$$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -0.8 \\ -0.9 \end{bmatrix} klf$$

(uplift pressure resultant for the toe of the foundation)

$$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 1.2 \\ 1.2 \end{bmatrix} ft$$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -4.3 \\ -4.5 \end{bmatrix} klf$$

(load case 1 for infrequent flooding)

$$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 4.5\ klf$$

(maximum of the drained vs undrained cases)

Factored Shear on Toe at Face of Stem

$$(V_u)_{toe} := \Sigma F_{toe} = 4.5\ klf$$

(maximum factored shear in toe)

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -5.6 \\ -5.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$$Mu_{toe} := \max(LC1) \cdot 1\ ft = -5.6\ kip \cdot ft$$

(resultant moment at base of floodwall stem per foot of wall)

$$Mu_{toe} := -Mu_{toe}$$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \text{Bar Size: \#8} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 19.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$a_{toe.bot} := \dfrac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \dfrac{a_{toe.bot}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{“OK”} \quad 0.08]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \dfrac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \text{find}\left(As_{req}\right) = 0.1\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.785\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.785\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.087\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.1]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.4\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.6]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0034$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.43\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 0.98]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0019$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.2]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 288\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 31.4\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 23.5\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 4.5\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.2]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

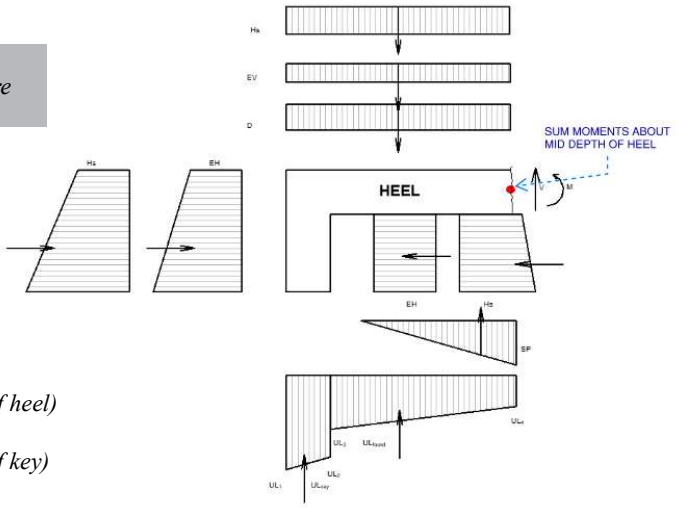
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 9\ ft$

(Length of heel, includes key)

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 300\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 525\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 2.7\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 1.6\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 4.3\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 4.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 7.5\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 5.6\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.624\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 5.6\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 4.5\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.624\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 0.967\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 4.4\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 3.95\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.809 \\ 0.959 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 2.7 \\ 3.2 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 2.8 \\ 2.8 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.052\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.22\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.7\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 4.318\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.2 \\ 1.01 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.2 \\ 1.01 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 4.2 \\ 3.5 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 2.8 \\ 2.8 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.097\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 0.88\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 4.5\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 1.35 \\ 3.2 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 7.65 \\ 5.8 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.93 \\ 0.86 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 3.57 \\ 2.49 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 2.5 \\ 1.9 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

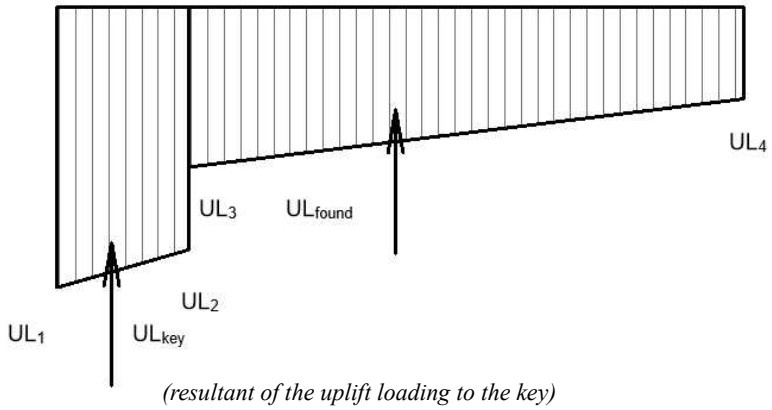
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot (UL_1 + UL_2)}{2} = \begin{bmatrix} 3.1 \\ 3.6 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 6\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot (2 \cdot UL_1 + UL_2)}{3 \cdot (UL_1 + UL_2)} \right) = \begin{bmatrix} 7.51 \\ 7.51 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{(UL_4 - UL_3)}{BWIDTH - TK} = \begin{bmatrix} -0.0316 \\ -0.0416 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.5 \\ 0.6 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot (UL_3 + UL_{stem})}{2} = \begin{bmatrix} 3.8 \\ 4.4 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot (2 \cdot UL_3 + UL_{stem})}{3 \cdot (UL_3 + UL_{stem})} \right) = \begin{bmatrix} 3.15 \\ 3.17 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left((EH)_{fh} - (EH)_{bh} \right) \\ + \left((Hs)_{fh} - (Hs)_{bh} \right) \end{pmatrix} = \begin{bmatrix} -2.9 \\ -2.6 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -2.6\ klf$

$(\Sigma F_{heel})_{horz} := \max\left(\overrightarrow{|LC1|}\right) = 2.9\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$ $= 0\ kip$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$

Forces on Heel - Vertical Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 33.9 \\ 33.9 \end{bmatrix} klf$ *(summation of the forces in the vertical direction)*

$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 33.9\ klf$ *(maximum of the drained vs undrained cases)*

Resultant Shear at Base

$Vu_{heel} := (\Sigma F_{heel})_{vert} = 33.9\ klf$ *(factored shear at connection of floodwall heel)*

Resultant Axial Load in Base

$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 2.9\ klf$ *(factored axial load at floodwall heel)*

$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$ *(tension in heel; from check above, zero if heel in compression)*

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{D \cdot (x_D)_{heel}}\right) \\ + \left(\overrightarrow{(EV)_{ah} \cdot (x_{EV})_{ah}}\right) + \left(\overrightarrow{(Hs)_{ah} \cdot (x_{Hs})_{ah}}\right) \\ + \left(\overrightarrow{(EH)_{fh} \cdot (x_{EH})_{fh}}\right) + \left(\overrightarrow{(Hs)_{fh} \cdot (x_{Hs})_{fh}}\right) \end{pmatrix} = 117.9\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{(Hs)_{bh} \cdot (x_{Hs})_{bh}}\right) \\ + \left(\overrightarrow{(EH)_{bh} \cdot (x_{EH})_{bh}}\right) \\ + \left(\overrightarrow{SP_{heel} \cdot (x_{SP})_{heel}}\right) \\ + \left(\overrightarrow{UL_{key} \cdot (x_{UL})_{key}}\right) \\ + \left(\overrightarrow{UL_{found} \cdot (x_{UL})_{found}}\right) \end{pmatrix} = \begin{bmatrix} 101.3 \\ 102.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = 16.61\ kip$

$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = 16.6\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$Mu_{heel} = 16.6\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

$(P_u)_{heel} = 2.9\ \frac{kip}{ft}$ *(maximum factored axial tension at base of floodwall stem on the heel)*

$Vu_{heel} = 33.9\ \frac{kip}{ft}$ *(maximum factored shear at base of floodwall stem on the heel)*

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 12\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$(A_s)_{heel.top} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$(A_s)_{heel.bot} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$\rho := \frac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for primary tension steel)*

$\rho' := \frac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for bottom tension steel)*

$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 20.5\ in$ *(depth to top reinforcement in heel)*

$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{heel.top} := \frac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.64\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 94.2\ kip$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$\phi Pn := \phi_A \cdot Pn_{max} = 84.8\ kip$

$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”} \quad 0.03]$ *(tensile)*

Flexural Check

$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \frac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right) = 42.5\ kip \cdot ft$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”} \quad 0.4]$

Minimum Required Steel for Flexure

Given

Constraints

Values

$As_{req} := 1.5\ in^2$

$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \frac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right)\right)$

Solver

$As_{req.stem} := \text{find}\left(\mathbf{As_{req}}\right) = 0.201\ in^2$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0.2\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.825\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.82\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.825\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0.3\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} := \text{if } A_{s\_1.3} < (A_s)_{heel.top} \parallel as \leftarrow (A_{s.min})_{heel.top}$ else $\parallel as \leftarrow A_{s.min} = 0.201\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad 0.26]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0032$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.4\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.5]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0036$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

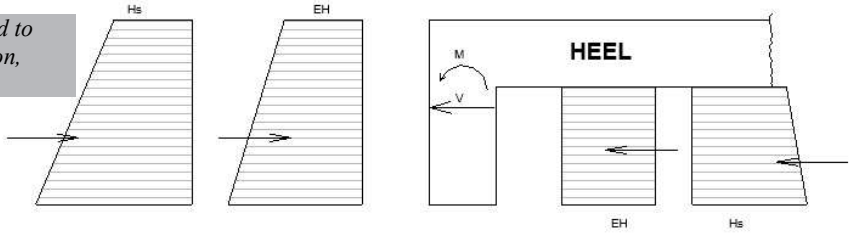
KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last

$HK = 3.5\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 5.5\ ft$ *(depth of key, from top of foundation down)*



Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.749\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 0.967\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 3\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 1.82\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.756 \\ 0.883 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.809 \\ 0.959 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 2.7 \\ 3.2 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot (2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 1.8 \\ 1.8 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.367\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.474\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 5.5\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 1.5\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 1.824\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.202 \\ 1.007 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3.5\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 4.2 \\ 3.5 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 1.8 \\ 1.8 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 13.1 \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 19.5 \\ 19 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 6.5 \\ 6 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 6.5\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 19.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.1]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Solve Constraints Values

$$As_{req} := 1.5\ in^2$$

$$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}}\right)$$

$$As_{req.stem} := \text{find}\left(As_{req}\right) = 0.074\ in^2$$

$$(A_{s.min})_{key.min} := As_{req.stem} = 0.1\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2}\right) = 0.648\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$ *(minimum area of steel required for flexural capacity)*

$$A_{s.min} = 0.785\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0.1\ in^2$$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$$(A_s)_{key.req} := \begin{array}{l} \text{if } A_{s\_1.3} < (A_s)_{key.main} \\ \quad as \leftarrow (A_{s.min})_{key.min} \\ \text{else} \\ \quad as \leftarrow A_{s.min} \end{array} = 0.074\ in^2$$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{key.main} = 0.8\ in^2$$ *(area of steel provided in the horizontal direction for the heel)*

$$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.1]$$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Requirements

$$(A_{s.min})_{temp} = 0.6\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.8]$$

(temperature and shrinkage steel of key, check utilization ratio)

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$ *(reinforcement ratio from EM 1110-2-2104)*

$$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main}\right)} = 0.0034$$ *(actual reinforcement ratio for concrete section)*

$$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.4]$$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{key.lat} := \text{Bar Size: \#6} = 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in key)*

$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in key)*

$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 30.6\ in$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.648\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.6\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{"OK"} \quad 0.7]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.lat}}{\left(b \cdot d_{key.main}\right)} = 0.0038$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

10 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS = 920.3\ ft$

$HTS = 10\ ft$

$TTS = 1.5\ ft$

$TBS = 1.5\ ft$

$THEEL = 5.5\ ft$

$TTOE = 2\ ft$

$TWIDTH = 2.5\ ft$

$BWIDTH = 13\ ft$

$HK = 3.5\ ft$

$TK = 3\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 930.66\ ft$

$HTS := 8\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 5\ ft$

$TTOE := 2\ ft$

$TWIDTH := 4.5\ ft$

$BWIDTH := 10.5\ ft$

$HK := 3\ ft$

$TK := 2.5\ ft$

$Elev_{soil.d} := 924.20\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 924.20\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 922.7\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 920.7\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 1.5\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 1.5\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

RS16+28 RB7+80 8ft Wall Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/ 8/21 Time: 10.07.27

Structural geometry data:
- *Elevation of top of stem (ELTS) = 930.66 ft*
- *Height of stem (HTS) = 8.00 ft*
- *Thickness top of stem (TTS) = 1.50 ft*
- *Thickness bottom of stem (TBS) = 1.50 ft*
- *Dist. of batter at bot. of stem (TBSR) = 0.00 ft*
- *Depth of heel (THEEL) = 5.00 ft*
- *Distance of batter for heel (BTRH) = 0.00 ft*
- *Depth of toe (TTOE) = 2.00 ft*
- *Width of toe (TWIDTH) = 4.50 ft*
- *Distance of batter for toe (BTRT) = 0.00 ft*
- *Width of base (BWIDTH) = 10.50 ft*
- *Depth of key (HK) = 3.00 ft*
- *Width of bottom of key (TK) = 2.50 ft*
- *Dist. of batter at bot. of key (BTRK) = 0.00 ft*

Structure coordinates:

x (ft)	y (ft)
0.00	917.66
0.00	922.66
4.50	922.66
4.50	930.66
6.00	930.66
6.00	922.66
10.50	922.66
10.50	920.66
2.50	920.66
2.50	917.66

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	924.20

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.48	14.50
2	-7.20	10.00
3	-0.53	500.00

UNDRAINED CASE

RS16+28 RB7+80 8ft Wall Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/12/15 Time: 9.22.35

Structural geometry data:
- *Elevation of top of stem (ELTS) = 930.66 ft*
- *Height of stem (HTS) = 8.00 ft*
- *Thickness top of stem (TTS) = 1.50 ft*
- *Thickness bottom of stem (TBS) = 1.50 ft*
- *Dist. of batter at bot. of stem (TBSR) = 0.00 ft*
- *Depth of heel (THEEL) = 5.00 ft*
- *Distance of batter for heel (BTRH) = 0.00 ft*
- *Depth of toe (TTOE) = 2.00 ft*
- *Width of toe (TWIDTH) = 4.50 ft*
- *Distance of batter for toe (BTRT) = 0.00 ft*
- *Width of base (BWIDTH) = 10.50 ft*
- *Depth of key (HK) = 3.00 ft*
- *Width of bottom of key (TK) = 2.00 ft*
- *Dist. of batter at bot. of key (BTRK) = 0.00 ft*

Structure coordinates:

x (ft)	y (ft)
0.00	917.66
0.00	922.66
4.50	922.66
4.50	930.66
6.00	930.66
6.00	922.66
10.50	922.66
10.50	920.66
2.00	920.66
2.00	917.66

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	924.20

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.48	14.50
2	-7.20	10.00
3	-0.53	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1020.00	893.12
2	-520.00	893.12
3	-20.00	915.20
4	-10.00	921.20
5	4.50	924.20

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	924.20	0.23

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	6.00	924.20
2	506.00	933.78

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)
phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 929.86 ft
Resisting side elevation = 924.16 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Strip load data:

Dist. to left end (ft)	Width (ft)	Magnitude (k/ft)
-20.00	-10.00	0.00

Horizontal line load data:

Elevation (ft)	Force (kips)
930.30	0.50

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
*o Computed cracks *will* be filled with water*

Strength mobilization factor = 0.666
*At-rest pressures on the resisting side *are used**
in the overturning analysis.
*Forces on the resisting side *are used** in the sliding analysis.*
**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1020.00	893.12
2	-520.00	893.12
3	-20.00	915.20
4	-10.00	921.20
5	4.50	924.20

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	924.20	0.97

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	6.00	924.20
2	506.00	964.62

Foundation property data:
phi for soil-structure interface = 0.00 (deg)
c for soil-structure interface = 1.000 (ksf)
phi for soil-soil interface = 0.00 (deg)
c for soil-soil interface = 1.000 (ksf)

Water data:
Driving side elevation = 930.66 ft
Resisting side elevation = 924.20 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
930.66	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
*o Crack *is* down to bottom of heel*
*o Computed cracks *will* be filled with water*

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** in the sliding analysis.*

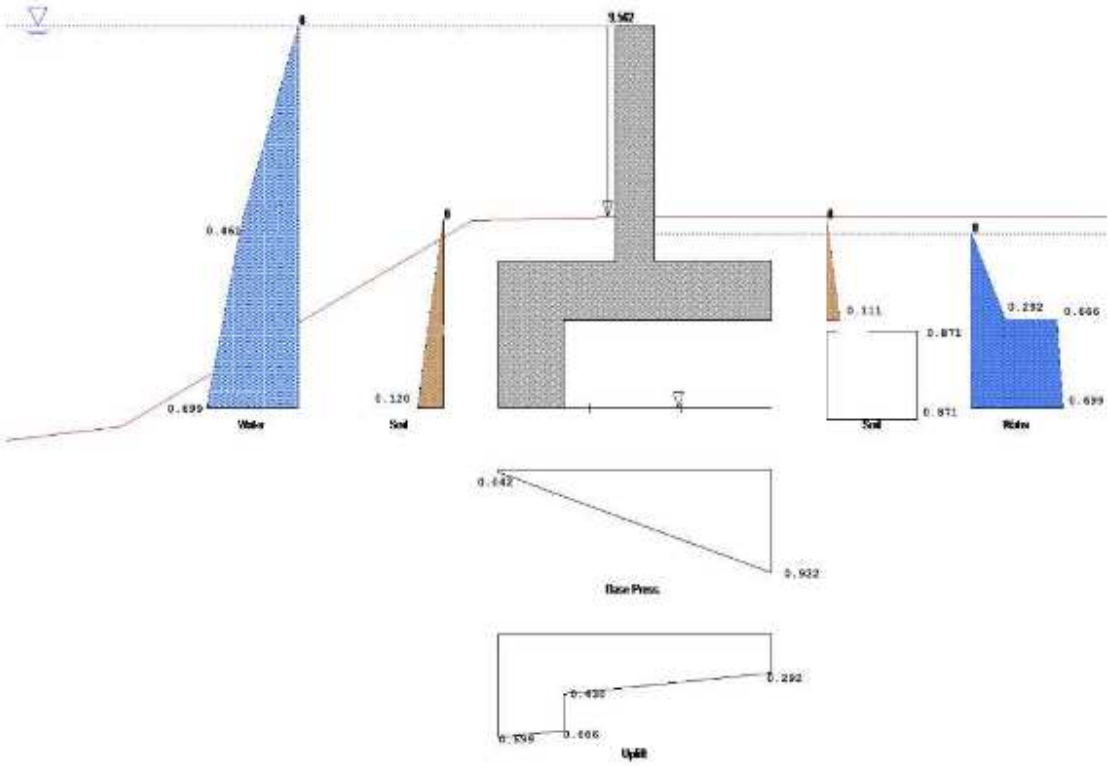
**Do* iterate in overturning analysis.*

g

Input file: RS 16+20 RB 7+80 8FT WALL _ Drained

CT Wall - Drained Case Output

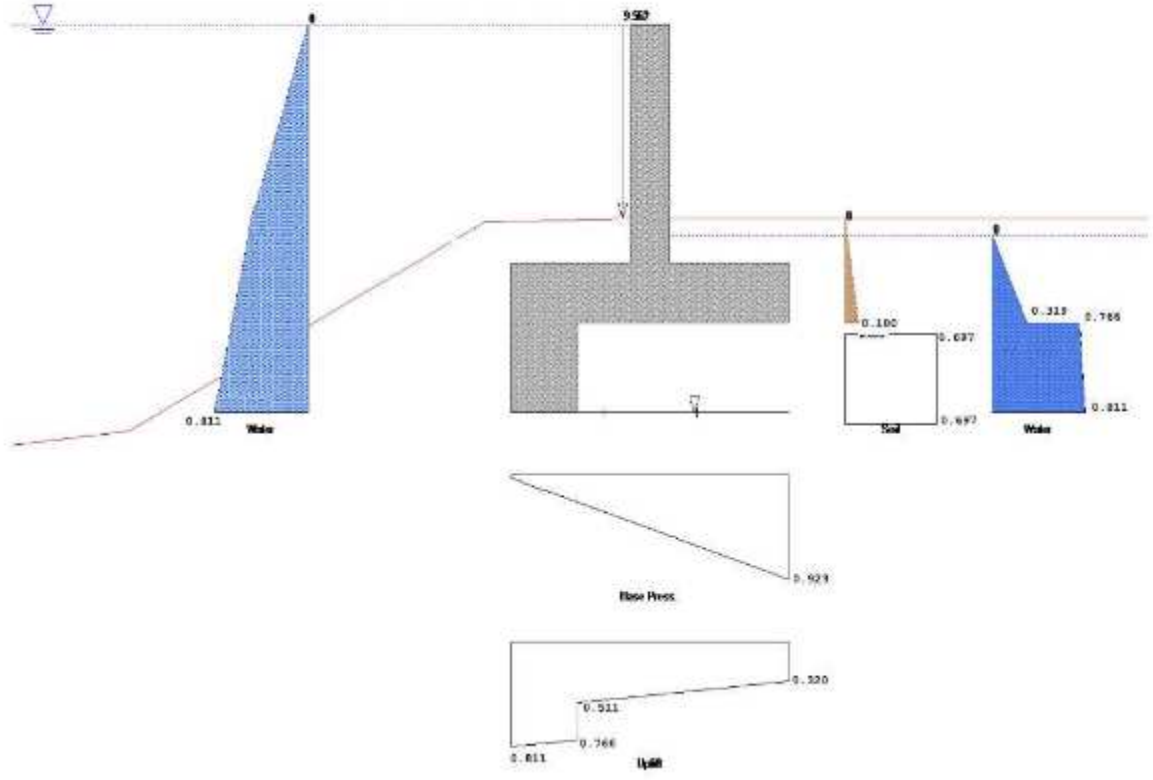
CTWall produced unhandled exception error in printing free body diagram of the wall.



Input file: RS 16+20 RB 7+80 8FT WALL _ Undrained

CT Wall - Undrained Case Output

CTWall produced unhandled exception error in printing free body diagram of the wall.



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.12 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.111 \\ 0.18 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 0.871 \\ 0.697 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 0.871 \\ 0.697 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 0.922 \\ 0.923 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 0.699 \\ 0.811 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 0.666 \\ 0.766 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.430 \\ 0.511 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.292 \\ 0.320 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.292 \\ 0.32 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.43 \\ 0.511 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 0.666 \\ 0.766 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

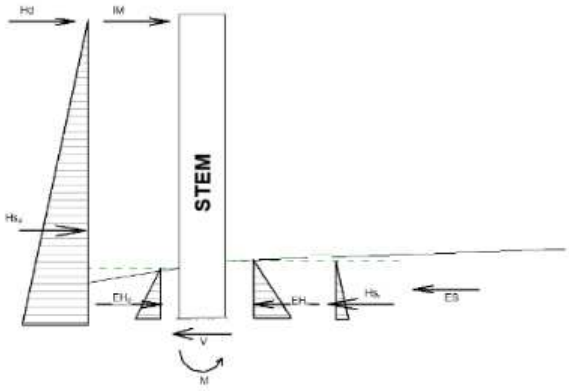
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 0.699 \\ 0.811 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 499.2 \\ 499.2 \end{bmatrix} psf$ *(hydrostatic pressure loading to driving side)*

$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 1.997 \\ 1.997 \end{bmatrix} klf$ *(hydrostatic loading to driving side)*

$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 2.7\ ft$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 96.1 \\ 96.1 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(hydrostatic loading to resisting side)*

$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.51\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.028 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side of stem)*

$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to driving side of stem)*

$(x_{EH})_d := HSD \cdot \frac{1}{3} = 0.5\ ft$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.048 \\ 0.078 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side of stem)*

$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0 \\ 0.1 \end{bmatrix} klf$ *(lateral earth loading to resisting side of stem)*

$(x_{EH})_r := HSR \cdot \frac{1}{3} = 0.5\ ft$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

$(x_{IM})_d := HTS = 8\ ft$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$p_{const} := 250\ psf$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$d_{soil} := HSR = 1.5\ ft$ *(depth of soil cover on stem)*

$(x_Q)_d := \frac{HSR}{2} = 0.8\ ft$ *(resultant height above base of stem)*

$x_Q := d_{soil} - (x_Q)_d = 0.8\ ft$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.204\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 6.5\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 6.5\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.113\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.217\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 4.8\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 1.8\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.1\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.1\ klf$

$(x_{EQ}) := HSR = 1.5\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot \langle EH_d + Hs_d + IM \rangle = \begin{bmatrix} 3.4 \\ 3.4 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \langle EH_d + EQ \rangle = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \langle EH_d + Q \rangle = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot \langle EH_r + Hs_r - W_c \rangle = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \langle EH_r \rangle = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \langle EH_r \rangle = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 3.4 \\ 3.3 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.3 \\ 0.2 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 3.4\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Hs_d \cdot (x_{Hs})_d \right) + IM \cdot (x_{IM})_d \right) = \begin{bmatrix} 9.8 \\ 9.8 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) \right) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Q \cdot \left((x_Q)_d \right) \right) \right) = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) + \left(Hs_r \cdot (x_{Hs})_r \right) \right) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0.03 \\ 0.05 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0.03 \\ 0.05 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 9.7 \\ 9.7 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0 \\ 0 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 9.7\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v\left(X_U, X_N\right) := \left| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left(X_U\right) \\ \quad \left| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left| ch_{i,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check\left(X_U, X_N\right) := \left| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left| ch_{1,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)

$$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$$

(nominal flexural strength, EM1110-2-2104, B-49)

$$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{“OK”} \quad 0.2]$$

(nominal flexural check, with Utilization ratio)

Minimum Required Steel for Flexure

Solve Constraints Given Values

$$As_{req} := 1.5\ in^2$$

$$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.151\ in^2$$

$$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.151\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.584\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.201\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{stem.req} := \begin{cases} as \leftarrow (A_{s.min})_{stem.ten} & \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \\ as \leftarrow A_{s.min} & \text{else} \end{cases} = 0.151\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{stem.vert} = 0.8\ in^2$$

(area of steel provided in the vertical direction for the stem)

$$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{“OK”} \quad 0.2]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 8 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 8 ft Floodwall Stem

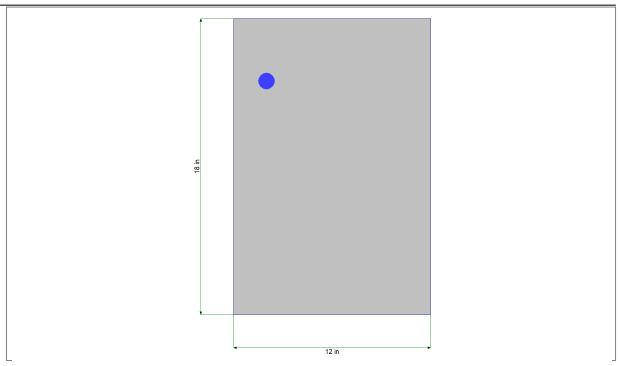
CODE REFERENCES

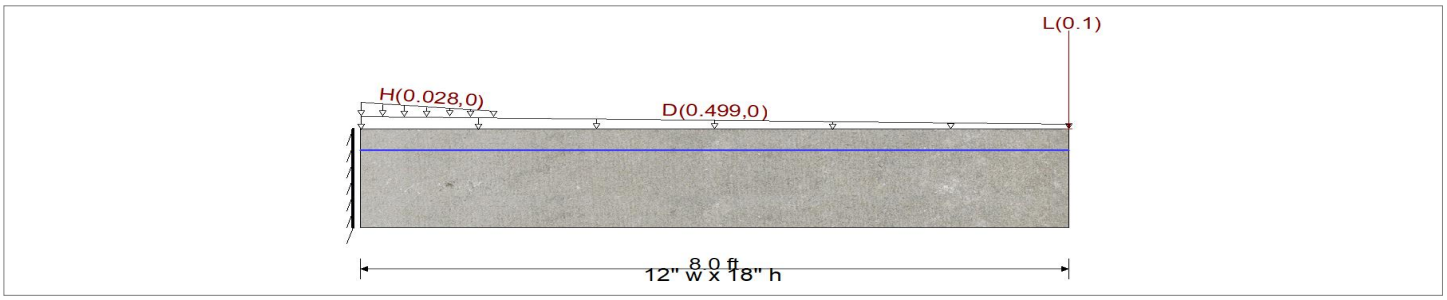
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	=	4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	503.12 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.8250
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,823.68 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		
			Number of Resisting Legs Per Stirrup =		

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.80 in from Top, from 0.0 to 8.0 ft in this span

Load for Span Number 1

Varying Uniform Load : D= 0.4990->0.0 k/ft, Extent = 0.0 -->> 8.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Driving side)

Varying Uniform Load : H= 0.0280->0.0 k/ft, Extent = 0.0 -->> 1.50 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 8.0 ft, (Impact Loading - Driving Side)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.202** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-9.813	k-ft
Mn * Phi : Allowable	48.645	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Downward Total Deflection	0.007 in	Ratio =	29072	>=180.0	Span: 1 : +D+L+H
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+L+H

Cross Section Strength & Inertia

Top & Bottom references are for tension side of sect

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.8",	0.00	48.65	5,832.00	45.19	855.63

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 8 ft Floodwall Stem

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.387	
Max Upward from Load Combinations	3.387	
+1.60D+1.60L+1.60H	3.387	
+1.60D+1.60H	3.227	
+1.60H	0.034	

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	8.000	-9.81	48.65	0.20
+1.40D+1.60H					
Span # 1	1	8.000	-7.47	48.65	0.15
+1.60D+1.60L+1.60H					
Span # 1	1	8.000	-9.81	48.65	0.20
+1.60D+1.60H					
Span # 1	1	8.000	-8.53	48.65	0.18
+1.60H					
Span # 1	1	8.000	-0.02	48.65	0.00
+1.60L					
Span # 1	1	8.000	-1.28	48.65	0.03

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+L+H	1	0.0066	8.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+H	1	0.0053	8.000	0.0000	0.000
+D+L+H	1	0.0066	8.000	0.0000	0.000
H Only	1	0.0000	8.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #6 $= 0.75\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.4\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 26.5\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 19.9\ kip$ *(factored shear-friction strength)*

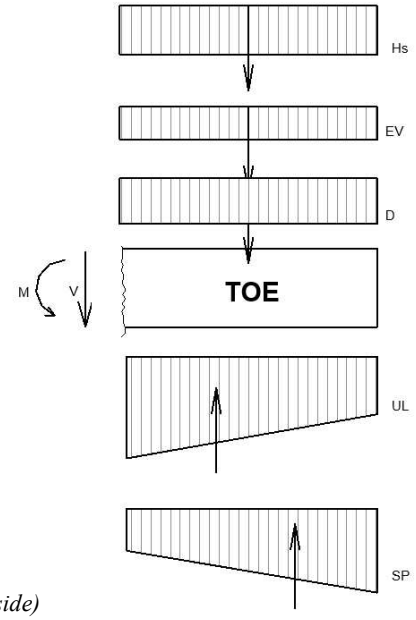
$V_{stem} := (Vu)_{stem} \cdot b = 3.4\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{"OK"} \quad 0.2]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 300\ psf$

$D := \sigma_D \cdot TWIDTH = 1.4\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 2.3\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 96.1 \\ 96.1 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 2.25\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 88.7\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.399 \\ 0.399 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 2.3\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -0.922 \\ -0.923 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 10.5\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 9.4 \\ 7.9 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 4.5\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 4.9 \\ 3.4 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -0.48 \\ -0.4 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)}{2} = \begin{bmatrix} -3.2 \\ -3 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 2.5 \\ 2.5 \end{bmatrix} ft$$

(moment arm from the connection of the toe to the resultant of the soil pressure loading)

Uplift Pressures

$$(\sigma_{UL})_{min} = \begin{bmatrix} 0.292 \\ 0.32 \end{bmatrix} ksf$$

(minimum uplift pressure at the end of the toe)

$$(\sigma_{UL})_{max} = \begin{bmatrix} 0.43 \\ 0.511 \end{bmatrix} ksf$$

(maximum uplift pressure behind the key)

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$BWIDTH = 10.5\ ft$ *(foundation width)*

$TK = 2.5\ ft$ *(key width)*

$L := BWIDTH - TK = 8\ ft$ *(length of linear uplift pressure)*

$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$ *(linear relationship)*

$$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0173 \\ 0.0239 \end{bmatrix} \frac{ksf}{ft}$$

(linear relationship)

$$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.37 \\ 0.427 \end{bmatrix} ksf$$

(uplift pressure at the connection of the toe to the foundation)

$$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -1.5 \\ -1.7 \end{bmatrix} klf$$

(uplift pressure resultant for the toe of the foundation)

$$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 2.2 \\ 2.1 \end{bmatrix} ft$$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -3.9 \\ -4 \end{bmatrix} klf$$

(load case 1 for infrequent flooding)

$$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 4\ klf$$

(maximum of the drained vs undrained cases)

Factored Shear on Toe at Face of Stem

$$(V_u)_{toe} := \Sigma F_{toe} = 4\ klf$$

(maximum factored shear in toe)

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -9.9 \\ -10 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$Mu_{toe} := \max(LC1) \cdot 1\ ft = -9.9\ kip \cdot ft$ *(resultant moment at base of floodwall stem per foot of wall)*

$Mu_{toe} := -Mu_{toe}$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := \boxed{4''\ \text{cover}}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 19.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{toe.bot} := \dfrac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \dfrac{a_{toe.bot}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{"OK"} \quad 0.15]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \dfrac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \mathbf{find}\left(As_{req}\right) = 0.2\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.785\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.8\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.785\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.2\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.153\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.2]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.4\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.6]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0034$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.43\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 0.98]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0019$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.2]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 288\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 31.4\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 23.5\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 4\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.2]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

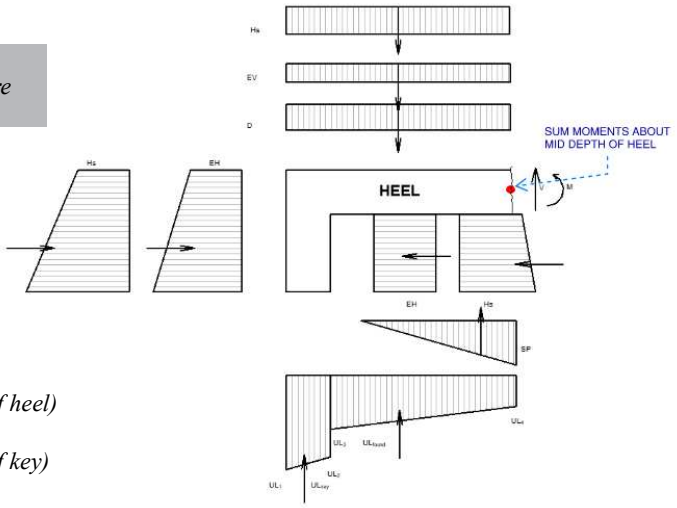
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 4.5\ ft$

(Length of heel, includes key)

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 300\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 450\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 1.4\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 1.1\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 2.5\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 2.3\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 3.3\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 2.7\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.499\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 2.2\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 2.25\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.499\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 0.811\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 5\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 3.3\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 3.7\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.666 \\ 0.766 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.699 \\ 0.811 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 2 \\ 2.4 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 2.5 \\ 2.5 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.047\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.2\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 5\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.6\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 4.016\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 0.87 \\ 0.7 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 0.87 \\ 0.7 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 2.6 \\ 2.1 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 2.5 \\ 2.5 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.089\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 0.4\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 2.25\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 1.09 \\ 2.59 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 3.41 \\ 1.91 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.33 \\ 0.22 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 0.57 \\ 0.21 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 1.1 \\ 0.6 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

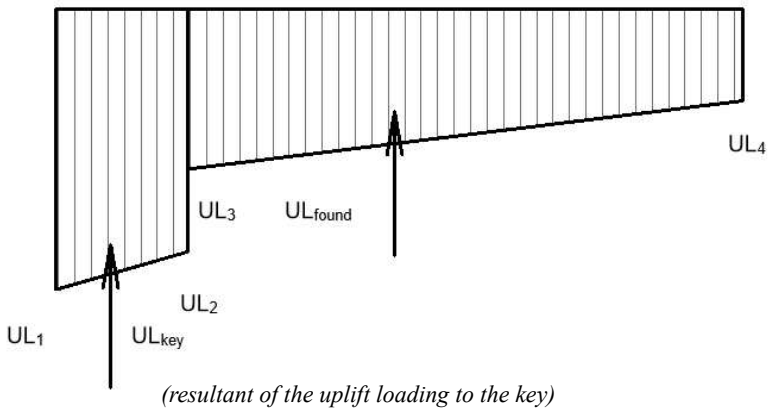
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot \langle UL_1 + UL_2 \rangle}{2} = \begin{bmatrix} 2.6 \\ 3 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 2\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot \langle 2 \cdot UL_1 + UL_2 \rangle}{3 \cdot \langle UL_1 + UL_2 \rangle} \right) = \begin{bmatrix} 3.26 \\ 3.26 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{\langle UL_4 - UL_3 \rangle}{BWIDTH - TK} = \begin{bmatrix} -0.0395 \\ -0.052 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.6 \\ 0.8 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot \langle UL_3 + UL_{stem} \rangle}{2} = \begin{bmatrix} 1.4 \\ 1.6 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot \langle 2 \cdot UL_3 + UL_{stem} \rangle}{3 \cdot \langle UL_3 + UL_{stem} \rangle} \right) = \begin{bmatrix} 1.02 \\ 1.02 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \langle (EH)_{fh} - (EH)_{bh} \rangle \\ + \langle (Hs)_{fh} - (Hs)_{bh} \rangle \end{pmatrix} = \begin{bmatrix} -1.2 \\ -0.9 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -0.9\ klf$

$(\Sigma F_{heel})_{horz} := \max\left(\overrightarrow{|LC1|}\right) = 1.2\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$
$= 0\ kip$

Forces on Heel - Vertical Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 15.4 \\ 15.9 \end{bmatrix} klf$ *(summation of the forces in the vertical direction)*

$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 15.9\ klf$ *(maximum of the drained vs undrained cases)*

Resultant Shear at Base

$Vu_{heel} := (\Sigma F_{heel})_{vert} = 15.9\ klf$ *(factored shear at connection of floodwall heel)*

Resultant Axial Load in Base

$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 1.2\ klf$ *(factored axial load at floodwall heel)*

$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$ *(tension in heel; from check above, zero if heel in compression)*

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{D \cdot (x_D)_{heel}}\right) \\ + \left(\overrightarrow{(EV)_{ah} \cdot (x_{EV})_{ah}}\right) + \left(\overrightarrow{(Hs)_{ah} \cdot (x_{Hs})_{ah}}\right) \\ + \left(\overrightarrow{(EH)_{fh} \cdot (x_{EH})_{fh}}\right) + \left(\overrightarrow{(Hs)_{fh} \cdot (x_{Hs})_{fh}}\right) \end{pmatrix} = 43.6\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{(Hs)_{bh} \cdot (x_{Hs})_{bh}}\right) \\ + \left(\overrightarrow{(EH)_{bh} \cdot (x_{EH})_{bh}}\right) \\ + \left(\overrightarrow{SP_{heel} \cdot (x_{SP})_{heel}}\right) \\ + \left(\overrightarrow{UL_{key} \cdot (x_{UL})_{key}}\right) \\ + \left(\overrightarrow{UL_{found} \cdot (x_{UL})_{found}}\right) \end{pmatrix} = \begin{bmatrix} 35.4 \\ 36.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = 8.23\ kip$

$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = 8.2\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$Mu_{heel} = 8.2\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

$(P_u)_{heel} = 1.2\ \frac{kip}{ft}$ *(maximum factored axial tension at base of floodwall stem on the heel)*

$Vu_{heel} = 15.9\ \frac{kip}{ft}$ *(maximum factored shear at base of floodwall stem on the heel)*

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 12\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$(A_s)_{heel.top} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$(A_s)_{heel.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$\rho := \dfrac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for primary tension steel)*

$\rho' := \dfrac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.003$ *(reinforcement ratio for bottom tension steel)*

$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 20.5\ in$ *(depth to top reinforcement in heel)*

$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{heel.top} := \dfrac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.62\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 94.2\ kip$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$\phi Pn := \phi_A \cdot Pn_{max} = 84.8\ kip$

$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”} \quad 0.01]$ *(tensile)*

Flexural Check

$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \dfrac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \dfrac{TTOE}{2}\right) \cdot b\right) = 42.1\ kip \cdot ft$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”} \quad 0.2]$

Minimum Required Steel for Flexure

Given

Constraints

Values

$As_{req} := 1.5\ in^2$

$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \dfrac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \dfrac{TTOE}{2}\right) \cdot b\right)\right)$

Solver

$As_{req.stem} := \text{find}\left(\mathbf{As_{req}}\right) = 0.098\ in^2$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0.1\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.825\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.82\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.825\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} :=$ if $A_{s\_1.3} < (A_s)_{heel.top}$ $as \leftarrow (A_{s.min})_{heel.top}$ else $as \leftarrow A_{s.min}$ $= 0.098\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad 0.12]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0032$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.432\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.4\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.5]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0036$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

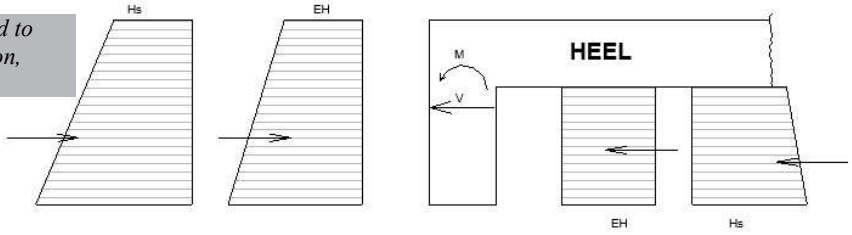
Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



$HK = 3\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 5\ ft$ *(depth of key, from top of foundation down)*

Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.624\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 0.811\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 2.2\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 1.57\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.666 \\ 0.766 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.699 \\ 0.811 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 2 \\ 2.4 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 1.5 \\ 1.5 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.306\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.397\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 5\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 1.1\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 1.565\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 0.871 \\ 0.697 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 0.871 \\ 0.697 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 2.6 \\ 2.1 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 1.5 \\ 1.5 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 8\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 11.2 \\ 10.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 3.2 \\ 2.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 3.2\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 19.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.6\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 67.8\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.05]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Solver Constraints Guess Values

$$As_{req} := 1.5\ in^2$$

$$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(As_{req}\right) = 0.036\ in^2$$

$$(A_{s.min})_{key.min} := As_{req.stem} = 0\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.8\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2}\right) = 0.54\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$ *(minimum area of steel required for flexural capacity)*

$$A_{s.min} = 0.785\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0\ in^2$$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$$(A_s)_{key.req} := \begin{array}{l} \text{if } A_{s\_1.3} < (A_s)_{key.main} \\ \quad \| \; as \leftarrow (A_{s.min})_{key.min} \\ \text{else} \\ \quad \| \; as \leftarrow A_{s.min} \end{array} = 0.036\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{key.main} = 0.8\ in^2$$ *(area of steel provided in the horizontal direction for the heel)*

$$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main}\right) = [\text{“OK”}\ \ 0.05]$$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Requirements

$$(A_{s.min})_{temp} = 0.5\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main}\right) = [\text{“OK”}\ \ 0.7]$$ *(temperature and shrinkage steel of key, check utilization ratio)*

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$ *(reinforcement ratio from EM 1110-2-2104)*

$$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main}\right)} = 0.0034$$ *(actual reinforcement ratio for concrete section)*

$$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”}\ \ 0.4]$$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{key.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} :=$ 4″ cover *(cover for lateral reinforcement in key)*

$$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$$ *(area of steel lateral reinforcement in key)*

$$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 24.6\ in$$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.54\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.5\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{“OK”} \quad 0.6]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$ *(reinforcement ratio from EM 1110-2-2104)*

$$\rho_{actual} := \frac{(A_s)_{key.lat}}{\left(b \cdot d_{key.main}\right)} = 0.0038$$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.5]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

8 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS = 930.7\ ft$

$HTS = 8\ ft$

$TTS = 1.5\ ft$

$TBS = 1.5\ ft$

$THEEL = 5\ ft$

$TTOE = 2\ ft$

$TWIDTH = 4.5\ ft$

$BWIDTH = 10.5\ ft$

$HK = 3\ ft$

$TK = 2.5\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 921.20\ ft$

$HTS := 6\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 4.5\ ft$

$TTOE := 1.5\ ft$

$TWIDTH := 4\ ft$

$BWIDTH := 9\ ft$

$HK := 3\ ft$

$TK := 1.5\ ft$

$Elev_{soil.d} := 916.7\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 916.7\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 915.2\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 913.7\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 1.5\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 1.5\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

RS 33+40 RB 25+20 _ ALT _ 6FT WALL-Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/12/16 Time: 9.21.01

Structural geometry data:
Elevation of top of stem (ELTS) = 921.20 ft
Height of stem (HTS) = 6.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 4.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 1.50 ft
Width of toe (TWIDTH) = 4.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 9.00 ft
Depth of key (HK) = 3.00 ft
Width of bottom of key (TK) = 1.50 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	910.70
0.00	915.20
3.50	915.20
3.50	921.20
5.00	921.20
5.00	915.20
9.00	915.20
9.00	913.70
1.50	913.70
1.50	910.70

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	916.70

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-1.20	10.00
2	-9.00	20.00
3	-1.33	500.00

UNDRAINED CASE

RS 33+40 RB 25+20 _ ALT _ 6FT WALL-Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/11/ 7 Time: 14.08.31

Structural geometry data:
Elevation of top of stem (ELTS) = 921.20 ft
Height of stem (HTS) = 6.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 4.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 1.50 ft
Width of toe (TWIDTH) = 3.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 6.00 ft
Depth of key (HK) = 3.00 ft
Width of bottom of key (TK) = 2.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	910.70
0.00	915.20
1.50	915.20
1.50	921.20
3.00	921.20
3.00	915.20
6.00	915.20
6.00	913.70
2.00	913.70
2.00	910.70

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	916.70

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-1.20	10.00
2	-9.00	20.00
3	-1.33	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1026.50	845.28
2	-526.50	845.28
3	-26.50	900.70
4	-6.50	915.70
5	3.50	916.70

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	916.70	0.18

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	5.00	916.70
2	505.00	924.20

Foundation property data:
- phi for soil-structure interface = 28.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 28.00 (deg)
- c for soil-soil interface = 0.000 (ksf)

Water data:
- Driving side elevation = 920.88 ft
- Resisting side elevation = 916.70 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.80	0.50

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1028.50	845.28
2	-528.50	845.28
3	-28.50	900.70
4	-8.50	915.70
5	1.50	916.70

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	916.70	0.18

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	3.00	916.70
2	503.00	924.20

Foundation property data:
- phi for soil-structure interface = 0.00 (deg)
- c for soil-structure interface = 1.000 (ksf)
- phi for soil-soil interface = 0.00 (deg)
- c for soil-soil interface = 1.000 (ksf)

Water data:
- Driving side elevation = 920.88 ft
- Resisting side elevation = 916.70 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.80	0.10

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack *is* down to bottom of heel
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

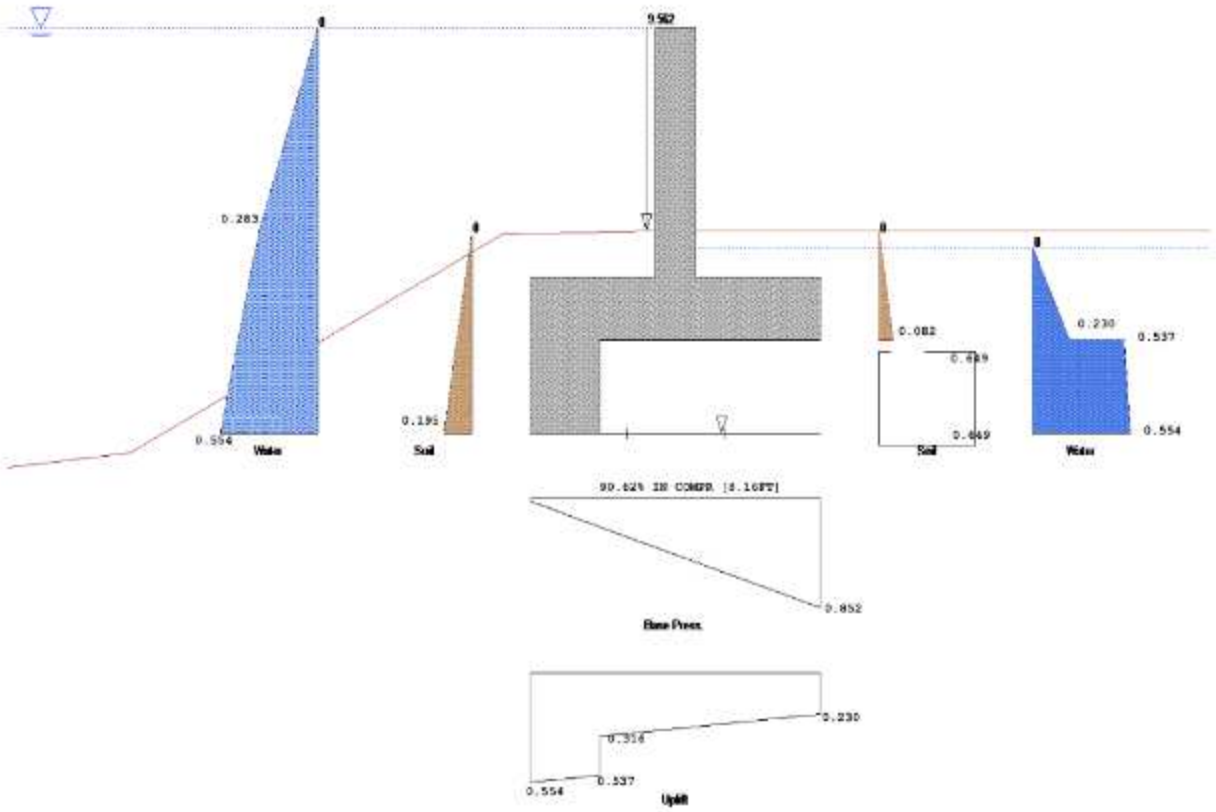
*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

g

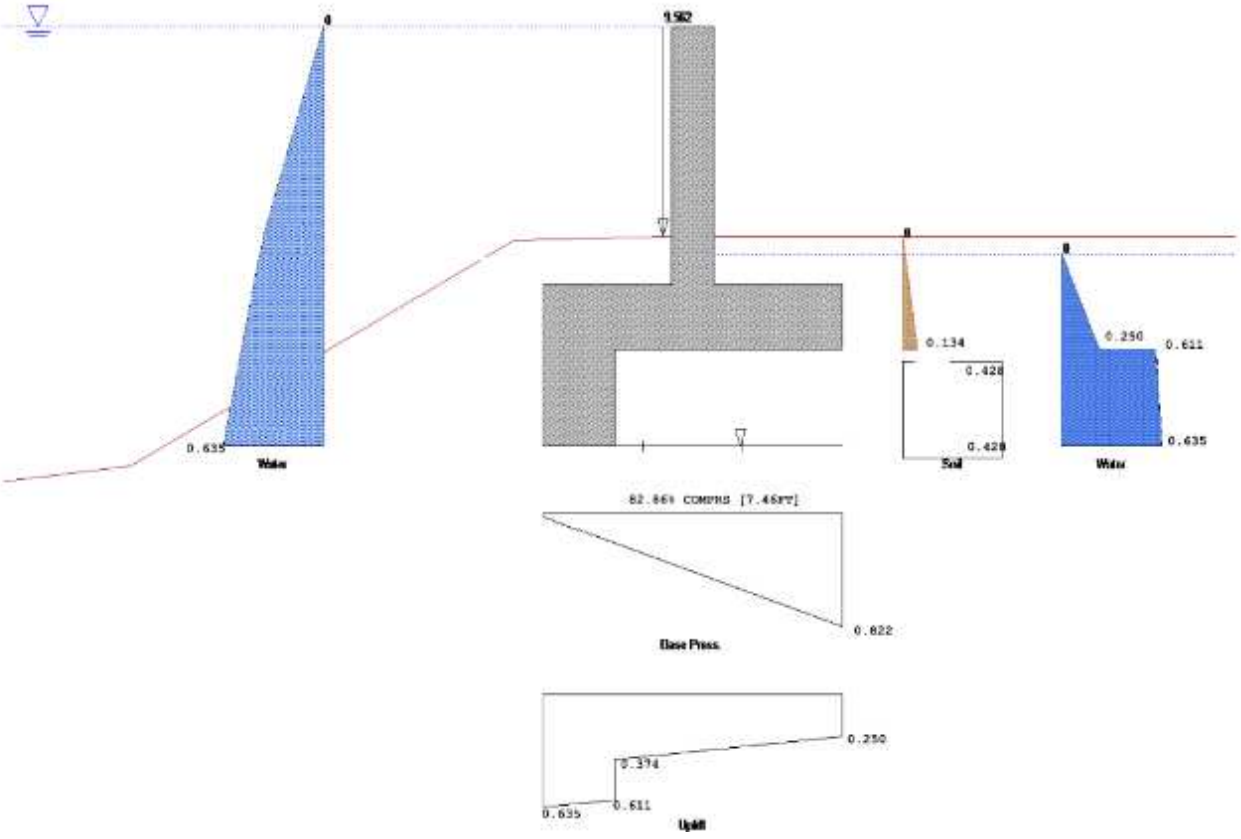
Input file: RS 33+40 RB 25+20 6FT WALL _ DRAINED

CT Wall - Drained Case Output



Input file: RS 33+40 RB 25+20 6FT WALL _ UNDRAINED

CT Wall - Undrained Case Output



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.195 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.082 \\ 0.134 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 0.649 \\ 0.428 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 0.649 \\ 0.428 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 0.852 \\ 0.822 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 0.554 \\ 0.635 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 0.537 \\ 0.611 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.316 \\ 0.374 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.230 \\ 0.250 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.23 \\ 0.25 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.316 \\ 0.374 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 0.537 \\ 0.611 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

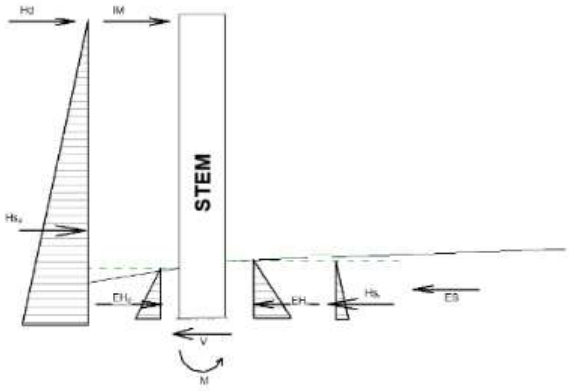
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 0.554 \\ 0.635 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 374.4 \\ 374.4 \end{bmatrix} psf$ *(hydrostatic pressure loading to driving side)*

$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 1.123 \\ 1.123 \end{bmatrix} klf$ *(hydrostatic loading to driving side)*

$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 2\ ft$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 93.6 \\ 93.6 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(hydrostatic loading to resisting side)*

$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.5\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.049 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side of stem)*

$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$ *(lateral earth loading to driving side of stem)*

$(x_{EH})_d := HSD \cdot \frac{1}{3} = 0.5\ ft$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.041 \\ 0.067 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side of stem)*

$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0 \\ 0.1 \end{bmatrix} klf$ *(lateral earth loading to resisting side of stem)*

$(x_{EH})_r := HSR \cdot \frac{1}{3} = 0.5\ ft$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

$(x_{IM})_d := HTS = 6\ ft$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$p_{const} := 250\ psf$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$d_{soil} := HSR = 1.5\ ft$ *(depth of soil cover on stem)*

$(x_Q)_d := \frac{HSR}{2} = 0.8\ ft$ *(resultant height above base of stem)*

$x_Q := d_{soil} - (x_Q)_d = 0.8\ ft$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.199\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 4.5\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 4.5\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.079\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.151\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 3.8\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 1.4\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.1\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.1\ klf$

$(x_{EQ}) := HSR = 1.5\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Hs_d + IM) = \begin{bmatrix} 2 \\ 2 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot (EH_d + EQ) = \begin{bmatrix} 0.2 \\ 0.1 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Q) = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot (EH_r + Hs_r - W_c) = \begin{bmatrix} 0 \\ 0.1 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0 \\ 0.1 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0 \\ 0.1 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 2 \\ 1.9 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.3 \\ 0.2 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 2\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Hs_d \cdot (x_{Hs})_d \right) + IM \cdot (x_{IM})_d \right) = \begin{bmatrix} 4.6 \\ 4.6 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(EQ \cdot (x_{EQ}) \right) \right) = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d \right) + \left(Q \cdot \left((x_Q)_d \right) \right) \right) = \begin{bmatrix} 0.3 \\ 0.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) + \left(Hs_r \cdot (x_{Hs})_r \right) \right) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r \right) \right) = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 4.5 \\ 4.5 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 4.5\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v(X_U, X_N) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}(X_U) \\ \quad \left\| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“N/A”} \end{array} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check(X_U, X_N) := \left\| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“N/A”} \end{array} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)

$$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$$

(nominal flexural strength, EM1110-2-2104, B-49)

$$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{"OK"} \quad 0.09]$$

(nominal flexural check, with Utilization ratio)

Minimum Required Steel for Flexure

Solve Constraints Given Values

$$As_{req} := 1.5\ in^2$$

$$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.069\ in^2$$

$$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.069\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.584\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.092\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{stem.req} := \begin{cases} as \leftarrow (A_{s.min})_{stem.ten} & \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \\ as \leftarrow A_{s.min} & \text{else} \end{cases} = 0.069\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{stem.vert} = 0.8\ in^2$$

(area of steel provided in the vertical direction for the stem)

$$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.1]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 6 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 6 ft Floodwall Stem

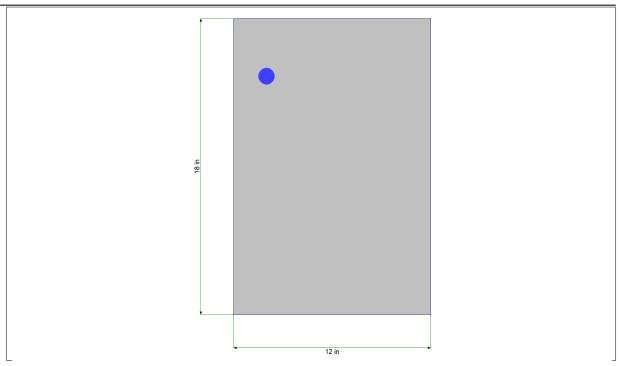
CODE REFERENCES

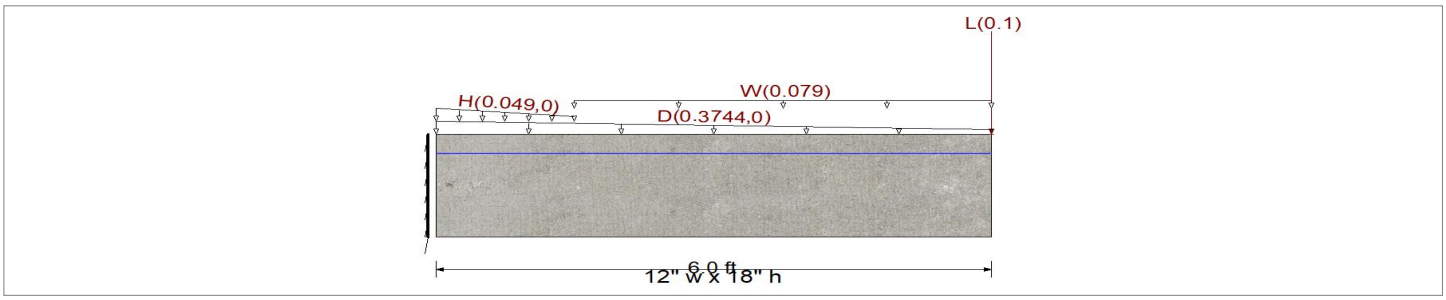
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	=	4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	503.12 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β 1	=	0.8250
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		
			Number of Resisting Legs Per Stirrup =		

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.50 in from Top, from 0.0 to 6.0 ft in this span

Load for Span Number 1

Varying Uniform Load : D= 0.3744->0.0 k/ft, Extent = 0.0 -->> 6.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Driving side)

Varying Uniform Load : H= 0.0490->0.0 k/ft, Extent = 0.0 -->> 1.50 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 6.0 ft, (Impact Loading - Driving Side)

Uniform Load : W = 0.0790 k/ft, Extent = 1.50 -->> 6.0 ft, Tributary Width = 1.0 ft, (Wind Load - Resisting Side [Negative Pressure])

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.092** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-4.584	k-ft
Mn * Phi : Allowable	49.712	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	
Max Downward Total Deflection	0.003 in	Ratio =	52752	>=180.0	Span: 1 : +D+W+H
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+W+H

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 6 ft Floodwall Stem

Cross Section Strength & Inertia

Top & Bottom references are for tension side of sect

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Bottom	Phi*Mn (k-ft) Top	Moment of Inertia (in^4) I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.5",	0.00	49.71	5,832.00	42.82	1,058.39

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.585	
Max Upward from Load Combinations	2.585	
Max Upward from Load Cases	0.356	
W Only	0.356	
+1.60D+1.60L+1.60W+1.60H	2.585	
+1.60D+1.60H	1.856	
+1.60H	0.059	
+1.60D+1.60L+1.60H	2.016	

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	6.000	-4.58	49.71	0.09
+1.40D+1.60H					
Span # 1	1	6.000	-3.17	49.71	0.06
+1.60D+1.60L+1.60H					
Span # 1	1	6.000	-4.58	49.71	0.09
+1.60D+1.60H					
Span # 1	1	6.000	-3.62	49.71	0.07
+1.60H					
Span # 1	1	6.000	-0.03	49.71	0.00
+1.60L					
Span # 1	1	6.000	-0.96	49.71	0.02

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+W+H	1	0.0027	6.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+W+H	1	0.0027	6.000	0.0000	0.000
+D+L+H	1	0.0022	6.000	0.0000	0.000
+D+H	1	0.0015	6.000	0.0000	0.000
H Only	1	0.0000	6.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #6 $= 0.75\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.4\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 26.5\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 19.9\ kip$ *(factored shear-friction strength)*

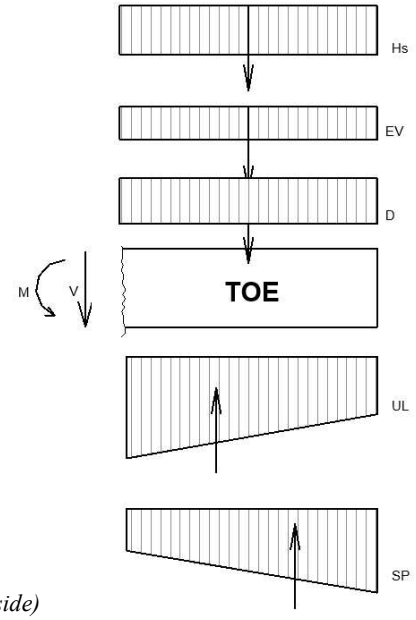
$V_{stem} := (Vu)_{stem} \cdot b = 2\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{“OK”} \quad 0.1]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 225\ psf$

$D := \sigma_D \cdot TWIDTH = 0.9\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 2\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 93.6 \\ 93.6 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 2\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 86.4\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.346 \\ 0.346 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 2\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -0.852 \\ -0.822 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 9\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 8.1 \\ 6.8 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 4\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 4.1 \\ 2.8 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -0.43 \\ -0.34 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)}{2} = \begin{bmatrix} -2.6 \\ -2.3 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 2.2 \\ 2.3 \end{bmatrix} ft$$

(moment arm from the connection of the toe to the resultant of the soil pressure loading)

Uplift Pressures

$$(\sigma_{UL})_{min} = \begin{bmatrix} 0.23 \\ 0.25 \end{bmatrix} ksf$$

(minimum uplift pressure at the end of the toe)

$$(\sigma_{UL})_{max} = \begin{bmatrix} 0.316 \\ 0.374 \end{bmatrix} ksf$$

(maximum uplift pressure behind the key)

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$BWIDTH = 9\ ft$ *(foundation width)*

$TK = 1.5\ ft$ *(key width)*

$L := BWIDTH - TK = 7.5\ ft$ *(length of linear uplift pressure)*

$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$ *(linear relationship)*

$$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0115 \\ 0.0165 \end{bmatrix} \frac{ksf}{ft}$$

(linear relationship)

$$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.276 \\ 0.316 \end{bmatrix} ksf$$

(uplift pressure at the connection of the toe to the foundation)

$$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -1 \\ -1.1 \end{bmatrix} klf$$

(uplift pressure resultant for the toe of the foundation)

$$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 1.9 \\ 1.9 \end{bmatrix} ft$$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -3.1 \\ -2.9 \end{bmatrix} klf$$

(load case 1 for infrequent flooding)

$$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 3.1\ klf$$

(maximum of the drained vs undrained cases)

Factored Shear on Toe at Face of Stem

$$(V_u)_{toe} := \Sigma F_{toe} = 3.1\ klf$$

(maximum factored shear in toe)

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -7.1 \\ -6.8 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$Mu_{toe} := \max(LC1) \cdot 1\ ft = -6.8\ kip \cdot ft$ *(resultant moment at base of floodwall stem per foot of wall)*

$Mu_{toe} := -Mu_{toe}$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := \boxed{4''\ \text{cover}}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 13.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{toe.bot} := \dfrac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.9\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \dfrac{a_{toe.bot}}{2}\right) = 46.1\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{“OK”} \quad 0.15]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \dfrac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \mathbf{find}\left(As_{req}\right) = 0.1\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.5\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.104\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.1]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.4]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.32\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 0.73]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0027$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 216\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 21.7\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 16.3\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 3.1\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.2]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

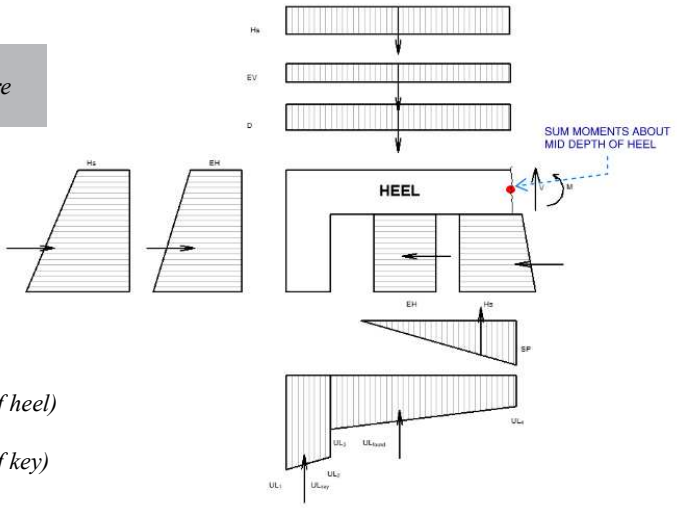
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 3.5\ ft$

(Length of heel, includes key)

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 225\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 450\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 0.8\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 0.7\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 1.5\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 1.8\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 2.8\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 2.2\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.374\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 1.3\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 1.75\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.374\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 0.655\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 4.5\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 2.3\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 3.2\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.537 \\ 0.611 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.554 \\ 0.635 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 1.6 \\ 1.9 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 2.3 \\ 2.3 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.046\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.183\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 4.5\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.5\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 3.45\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 0.65 \\ 0.43 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 0.65 \\ 0.43 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 1.9 \\ 1.3 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 2.3 \\ 2.3 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.086\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 0.3\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 1.75\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 0.94 \\ 2.22 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 2.56 \\ 1.28 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.27 \\ 0.16 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 0.35 \\ 0.1 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 0.9 \\ 0.4 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

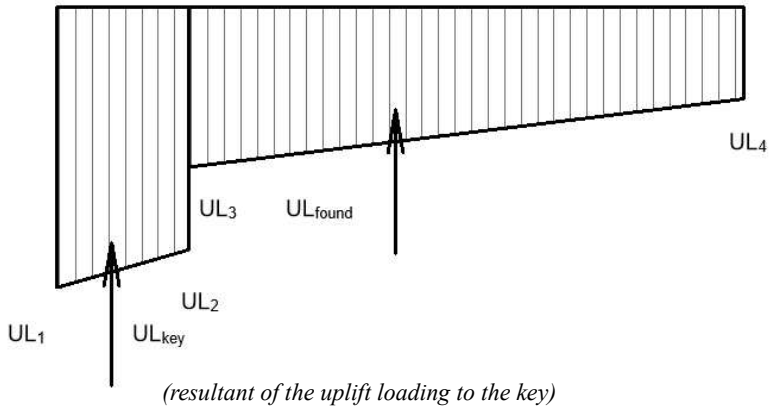
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot \langle UL_1 + UL_2 \rangle}{2} = \begin{bmatrix} 1.5 \\ 1.8 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 2\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot \langle 2 \cdot UL_1 + UL_2 \rangle}{3 \cdot \langle UL_1 + UL_2 \rangle} \right) = \begin{bmatrix} 2.75 \\ 2.76 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{\langle UL_4 - UL_3 \rangle}{BWIDTH - TK} = \begin{bmatrix} -0.0421 \\ -0.0555 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.6 \\ 0.7 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot \langle UL_3 + UL_{stem} \rangle}{2} = \begin{bmatrix} 1.4 \\ 1.6 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot \langle 2 \cdot UL_3 + UL_{stem} \rangle}{3 \cdot \langle UL_3 + UL_{stem} \rangle} \right) = \begin{bmatrix} 1.02 \\ 1.02 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \langle (EH)_{fh} - (EH)_{bh} \rangle \\ + \langle (Hs)_{fh} - (Hs)_{bh} \rangle \end{pmatrix} = \begin{bmatrix} -1.2 \\ -0.5 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -0.5\ klf$

$(\Sigma F_{heel})_{horz} := \max\left(\overrightarrow{|LC1|}\right) = 1.2\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$ $= 0\ kip$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$

Forces on Heel - Vertical Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 10.1 \\ 10.5 \end{bmatrix} klf$ *(summation of the forces in the vertical direction)*

$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 10.5\ klf$ *(maximum of the drained vs undrained cases)*

Resultant Shear at Base

$Vu_{heel} := (\Sigma F_{heel})_{vert} = 10.5\ klf$ *(factored shear at connection of floodwall heel)*

Resultant Axial Load in Base

$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 1.2\ klf$ *(factored axial load at floodwall heel)*

$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$ *(tension in heel; from check above, zero if heel in compression)*

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{D \cdot (x_D)_{heel}}\right) \\ + \left(\overrightarrow{(EV)_{ah} \cdot (x_{EV})_{ah}}\right) + \left(\overrightarrow{(Hs)_{ah} \cdot (x_{Hs})_{ah}}\right) \\ + \left(\overrightarrow{(EH)_{fh} \cdot (x_{EH})_{fh}}\right) + \left(\overrightarrow{(Hs)_{fh} \cdot (x_{Hs})_{fh}}\right) \end{pmatrix} = 24.4\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{(Hs)_{bh} \cdot (x_{Hs})_{bh}}\right) \\ + \left(\overrightarrow{(EH)_{bh} \cdot (x_{EH})_{bh}}\right) \\ + \left(\overrightarrow{SP_{heel} \cdot (x_{SP})_{heel}}\right) \\ + \left(\overrightarrow{UL_{key} \cdot (x_{UL})_{key}}\right) \\ + \left(\overrightarrow{UL_{found} \cdot (x_{UL})_{found}}\right) \end{pmatrix} = \begin{bmatrix} 22.4 \\ 21.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = 2.47\ kip$

$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = 2.5\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$Mu_{heel} = 2.5\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

$(P_u)_{heel} = 1.2\ \frac{kip}{ft}$ *(maximum factored axial tension at base of floodwall stem on the heel)*

$Vu_{heel} = 10.5\ \frac{kip}{ft}$ *(maximum factored shear at base of floodwall stem on the heel)*

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 12\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$(A_s)_{heel.top} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$(A_s)_{heel.bot} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$\rho := \frac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.004$ *(reinforcement ratio for primary tension steel)*

$\rho' := \frac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.004$ *(reinforcement ratio for bottom tension steel)*

$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 14.5\ in$ *(depth to top reinforcement in heel)*

$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{heel.top} := \frac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.87\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 94.2\ kip$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$\phi Pn := \phi_A \cdot Pn_{max} = 84.8\ kip$

$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”} \quad 0.01]$ *(tensile)*

Flexural Check

$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \frac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right) = 41.7\ kip \cdot ft$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”} \quad 0.1]$

Minimum Required Steel for Flexure

Given

Constraints

Values

$As_{req} := 1.5\ in^2$

$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \frac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \frac{TTOE}{2}\right) \cdot b\right)\right)$

Solver

$As_{req.stem} := \text{find}\left(\mathbf{As_{req}}\right) = 0.046\ in^2$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} := \text{if } A_{s\_1.3} < (A_s)_{heel.top} \parallel as \leftarrow (A_{s.min})_{heel.top}$ $\text{else} \parallel as \leftarrow A_{s.min} = 0.046\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad 0.06]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.4]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0051$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.7]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

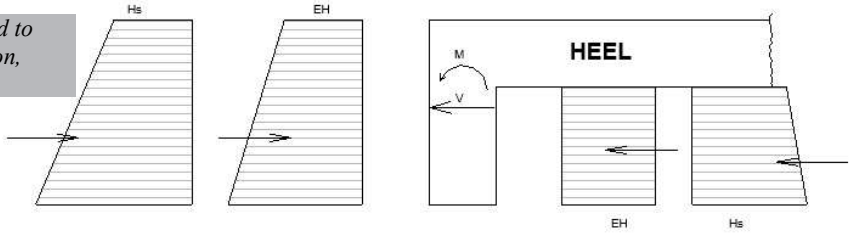
KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last

$HK = 3\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 4.5\ ft$ *(depth of key, from top of foundation down)*



Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.468\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 0.655\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 1.7\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 1.58\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.537 \\ 0.611 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.554 \\ 0.635 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 1.6 \\ 1.9 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 1.5 \\ 1.5 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.229\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.321\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 4.5\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 0.8\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 1.583\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 0.649 \\ 0.428 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 0.649 \\ 0.428 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 3\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 1.9 \\ 1.3 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 1.5 \\ 1.5 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 6.4\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 8.6 \\ 7.6 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 2.3 \\ 1.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 2.3\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 13.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.9\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 46.1\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.05]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Solve Constraints Values

$$As_{req} := 1.5\ in^2$$

$$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}}\right)$$

$$As_{req.stem} := \text{find}\left(As_{req}\right) = 0.037\ in^2$$

$$(A_{s.min})_{key.min} := As_{req.stem} = 0\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.5\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.5\ in^2$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2}\right) = 0.324\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$

(minimum area of steel required for flexural capacity)

$$A_{s.min} = 0.543\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0\ in^2$$

(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)

$$(A_s)_{key.req} := \text{if } A_{s\_1.3} < (A_s)_{key.main} \ \| \ as \leftarrow (A_{s.min})_{key.min} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.037\ in^2$$

(required steel to meet steel requirements)

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{key.main} = 0.8\ in^2$$

(area of steel provided in the horizontal direction for the heel)

$$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.05]$$

(check stem reinforcement to required reinforcement)

Temperature and Shrinkage Requirements

$$(A_{s.min})_{temp} = 0.3\ in^2$$

(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))

$$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.4]$$

(temperature and shrinkage steel of key, check utilization ratio)

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$

(reinforcement ratio from EM 1110-2-2104)

$$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main}\right)} = 0.0048$$

(actual reinforcement ratio for concrete section)

$$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.6]$$

(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)

Lateral Bars

$(d_b)_{key.lat} := \text{Bar Size: \#6} = 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in key)*

$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in key)*

$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{"OK"} \quad 0.4]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.lat}}{\left(b \cdot d_{key.main}\right)} = 0.0055$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.7]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

6 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS = 921.2\ ft$

$HTS = 6\ ft$

$TTS = 1.5\ ft$

$TBS = 1.5\ ft$

$THEEL = 4.5\ ft$

$TTOE = 1.5\ ft$

$TWIDTH = 4\ ft$

$BWIDTH = 9\ ft$

$HK = 3\ ft$

$TK = 1.5\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

T-Wall Load Cases and Strength Design

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS

- Strength Design Loading conditions and load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

FLOOD LOAD CASES

Unusual Case

Top of Wall, ACE = 100 year event, Unusual Case

WSE = Top of Protection

Usual Case

ACE = 10 year event, Usual Case

WSE elevation below the top of the foundation

Note: The Unusual Flood Case with water to the top of the wall governs the design of this floodwall. Non-flood condition loads such as construction, seismic and wind are addressed in the stem design; however, it is apparent that the top of wall flood condition governs stability and the primary steel design so only the Unusual top of wall flood load is considered for stability and the base design.

The Usual Flood Case does not have water to the top of the foundation and is not included in these calculations.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Undrained Soil Conditions

$\phi_{undrained} := 0\ deg$ *(internal friction angle for drained soil condition)*

$C_{undrained} := 1000\ psf$ *(cohesion for drained soil condition)*

$(k_0)_{undrained} := 1 - \sin\left(\phi_{undrained}\right) = 1$ *(at-rest soil pressure coefficient for the undrained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$q\lambda_c := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

CT WALL DATA

Input Data - Drained

$ELTS := 938.50\ ft$

$HTS := 4\ ft$

$TTS := 1.5\ ft$

$TBS := 1.5\ ft$

$THEEL := 2.5\ ft$

$TTOE := 1.5\ ft$

$TWIDTH := 3\ ft$

$BWIDTH := 8\ ft$

$HK := 1\ ft$

$TK := 1\ ft$

$Elev_{soil.d} := 936.05\ ft$ *(driving side soil elevation)*

$Elev_{soil.r} := 936.05\ ft$ *(resisting side soil elevation)*

$ELTF := ELTS - HTS = 934.5\ ft$ *(calculated top of foundation elevation)*

$ELTK := ELTF - THEEL + HK = 933\ ft$ *(calculated top of key, bottom of foundation elevation)*

$HSD := Elev_{soil.d} - ELTF = 1.6\ ft$ *(calculated depth of soil, driving side)*

$HSR := Elev_{soil.r} - ELTF = 1.6\ ft$ *(calculated depth of soil, resisting side)*

DRAINED CASE

RS3+33 LB 9+00 4ft Wall
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/ 9/12 Time: 9.27.55

Structural geometry data:
- *Elevation of top of stem (ELTS) = 938.50 ft*
- *Height of stem (HTS) = 4.00 ft*
- *Thickness top of stem (TTS) = 1.50 ft*
- *Thickness bottom of stem (TBS) = 1.50 ft*
- *Dist. of batter at bot. of stem (TBSR) = 0.00 ft*
- *Depth of heel (THEEL) = 2.50 ft*
- *Distance of batter for heel (BTRH) = 0.00 ft*
- *Depth of toe (TTOE) = 1.50 ft*
- *Width of toe (TWIDTH) = 3.00 ft*
- *Distance of batter for toe (BTRT) = 0.00 ft*
- *Width of base (BWIDTH) = 8.00 ft*
- *Depth of key (HK) = 1.00 ft*
- *Width of bottom of key (TK) = 1.00 ft*
- *Dist. of batter at bot. of key (BTRK) = 0.00 ft*

Structure coordinates:

x (ft)	y (ft)
0.00	932.00
0.00	934.50
3.50	934.50
3.50	938.50
5.00	938.50
5.00	934.50
8.00	934.50
8.00	933.00
1.00	933.00
1.00	932.00

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
- *Unit weight of concrete = 0.150 kcf*

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	936.05

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.52	5.00
2	-6.86	17.50
3	-0.55	500.00

UNDRAINED CASE

RS3+33 LB 9+00 4ft Wall
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********
Date: 2025/12/18 Time: 13.32.48

Structural geometry data:
- *Elevation of top of stem (ELTS) = 938.50 ft*
- *Height of stem (HTS) = 4.00 ft*
- *Thickness top of stem (TTS) = 1.50 ft*
- *Thickness bottom of stem (TBS) = 1.50 ft*
- *Dist. of batter at bot. of stem (TBSR) = 0.00 ft*
- *Depth of heel (THEEL) = 2.50 ft*
- *Distance of batter for heel (BTRH) = 0.00 ft*
- *Depth of toe (TTOE) = 1.50 ft*
- *Width of toe (TWIDTH) = 3.00 ft*
- *Distance of batter for toe (BTRT) = 0.00 ft*
- *Width of base (BWIDTH) = 8.00 ft*
- *Depth of key (HK) = 1.00 ft*
- *Width of bottom of key (TK) = 1.00 ft*
- *Dist. of batter at bot. of key (BTRK) = 0.00 ft*

Structure coordinates:

x (ft)	y (ft)
0.00	932.00
0.00	934.50
3.50	934.50
3.50	938.50
5.00	938.50
5.00	934.50
8.00	934.50
8.00	933.00
1.00	933.00
1.00	932.00

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
- *Unit weight of concrete = 0.150 kcf*

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	936.05

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.52	5.00
2	-6.86	17.50
3	-0.55	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1019.00	902.08
2	-519.00	902.08
3	-19.00	925.00
4	-1.50	935.00
5	3.50	936.05

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	936.05	0.55

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	5.00	936.05
2	505.00	958.97

Foundation property data:
- phi for soil-structure interface = 28.00 (deg)
- c for soil-structure interface = 0.000 (ksf)
- phi for soil-soil interface = 28.00 (deg)
- c for soil-soil interface = 0.000 (ksf)

Water data:
- Driving side elevation = 938.40 ft
- Resisting side elevation = 936.05 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
938.40	0.50

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack depth is to be calculated
- o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning analysis.*

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1019.00	902.08
2	-519.00	902.08
3	-19.00	925.00
4	-1.50	935.00
5	3.50	936.05

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	936.05	0.55

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	5.00	936.05
2	505.00	958.97

Foundation property data:
- phi for soil-structure interface = 0.00 (deg)
- c for soil-structure interface = 1.000 (ksf)
- phi for soil-soil interface = 0.00 (deg)
- c for soil-soil interface = 1.000 (ksf)

Water data:
- Driving side elevation = 938.40 ft
- Resisting side elevation = 936.05 ft
- Unit weight of water = 0.0624 kcf
- Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
938.40	0.50

Minimum required factors of safety:
- Sliding FS = 1.50
- Overturning = 75.00% base in compression

Crack options:
- o Crack *is* down to bottom of heel
- o Computed cracks *will* be filled with water

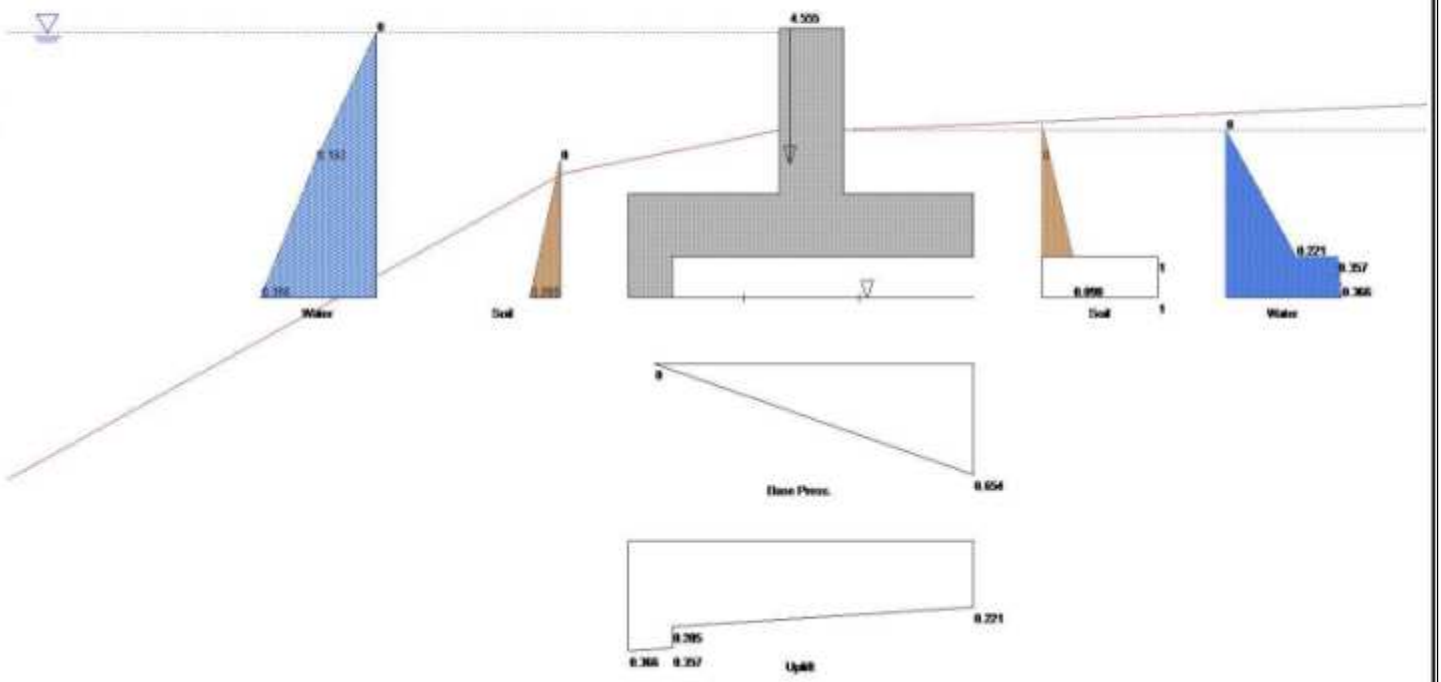
Strength mobilization factor = 0.6667

*At-rest pressures on the resisting side *are used**
in the overturning analysis.

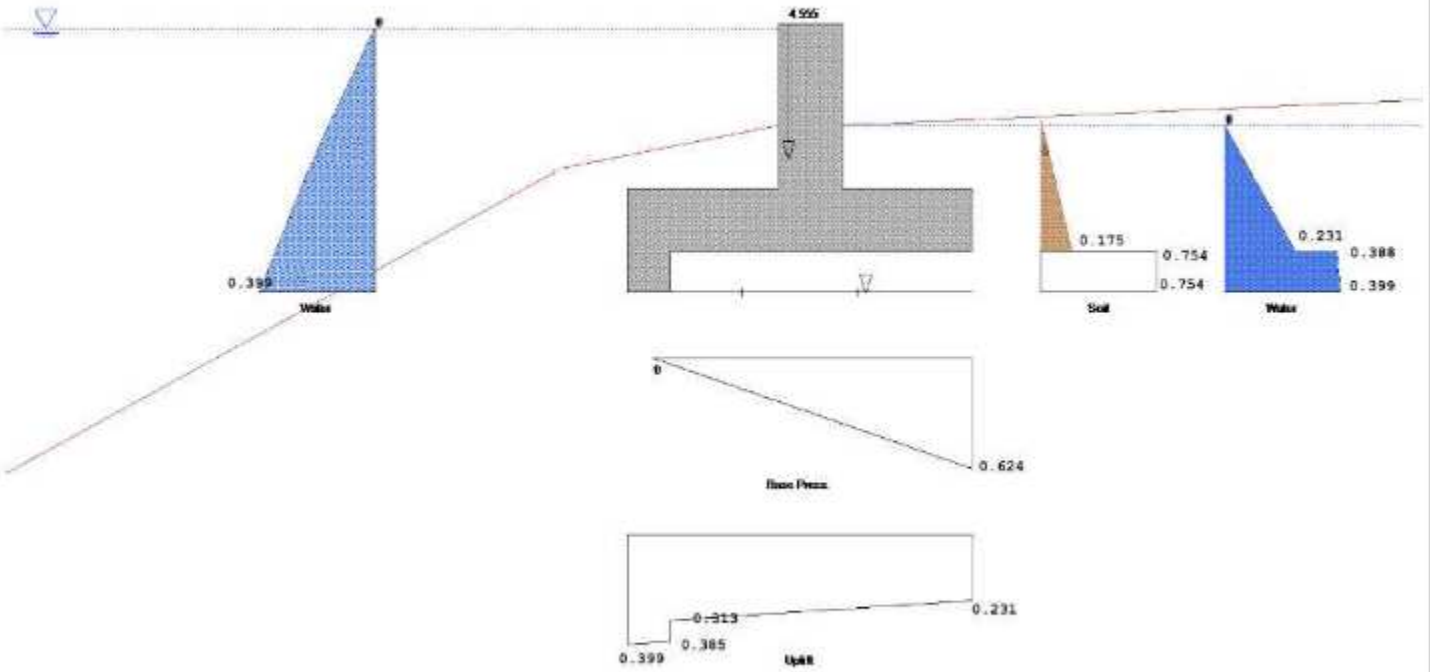
*Forces on the resisting side *are used** *in the sliding analysis.*

**Do* iterate in overturning*

CT Wall - Drained Case Output



CT Wall - Undrained Case Output



CTWALL OUTPUT

Lateral Earth Loading

$\sigma_{EHd} := \begin{bmatrix} 0.0949 \\ 0 \end{bmatrix} ksf$ *(lateral earth pressure loading to driving side, bottom of key; Drained, Undrained)*

$\sigma_{EHr} := \begin{bmatrix} 0.098 \\ 0.175 \end{bmatrix} ksf$ *(lateral earth pressure loading to resisting side, bottom of foundation at toe; Drained, Undrained)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} := \begin{bmatrix} 1.02 \\ 0.754 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation; Drained, Undrained)*

$(\sigma_{EH})_{bh.bot} := \begin{bmatrix} 1.02 \\ 0.754 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key, Drained, Undrained)*

Impact Loading

$IM := 0.1\ klf$ *(impact loading to the top of the wall)*

Soil Pressure

$(\sigma_{SP})_{max} := -\begin{bmatrix} 0.654 \\ 0.624 \end{bmatrix} ksf$ *(max base soil loading from CT Wall Output)*

Uplift Pressure

$UL_1 := \begin{bmatrix} 0.366 \\ 0.399 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 0.357 \\ 0.388 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.285 \\ 0.313 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.221 \\ 0.231 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$(\sigma_{UL})_{min} := UL_4 = \begin{bmatrix} 0.221 \\ 0.231 \end{bmatrix} ksf$ *(minimum uplift pressure at the end of the toe)*

$(\sigma_{UL})_{max} := UL_3 = \begin{bmatrix} 0.285 \\ 0.313 \end{bmatrix} ksf$ *(maximum uplift pressure behind the key)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} := \begin{bmatrix} 0.357 \\ 0.388 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

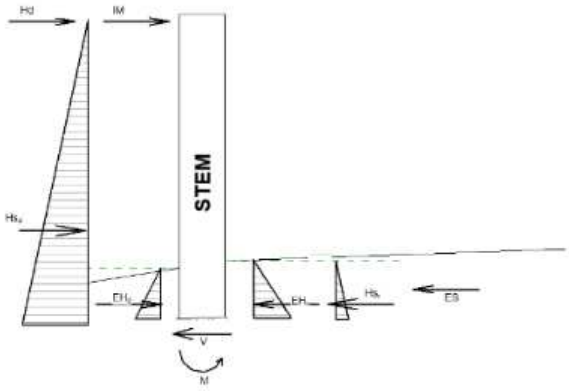
$(\sigma_{Hs})_{bh.bot} := \begin{bmatrix} 0.366 \\ 0.399 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

STEM DESIGN, RIVER SIDE

Loads to Stem

Note: largest loading from drained and undrained cases used to determine the controlling load case. In each loading condition, drained loads are listed first and undrained last.

This design for the riverside steel, along with the EnerCalc calculations, attached will be used for the landside steel design for simplicity. By inspection, the riverside reinforcement is sufficient for the landside reinforcement design.



Hydrostatic Loads

$$\sigma_{Hsd} := \begin{bmatrix} \gamma_{water} \cdot HTS \\ \gamma_{water} \cdot HTS \end{bmatrix} = \begin{bmatrix} 249.6 \\ 249.6 \end{bmatrix} psf$$ *(hydrostatic pressure loading to driving side)*

$$Hs_d := \overrightarrow{0.5 \cdot \sigma_{Hsd} \cdot HTS} = \begin{bmatrix} 0.499 \\ 0.499 \end{bmatrix} klf$$ *(hydrostatic loading to driving side)*

$$(x_{Hs})_d := HTS \cdot \frac{1}{3} = 1.3\ ft$$ *(moment arm for hydrostatic loading to the driving side from the base of the stem)*

Note: assume that hydrostatic loading from the resisting side of the T-wall provided from groundwater up to the ground surface. Assume that all additional water from the resisting side, landside, of the floodwall is conveyed away through interior drainage systems.

$$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 96.7 \\ 96.7 \end{bmatrix} psf$$ *(hydrostatic pressure loading to resisting side)*

$$Hs_r := \overrightarrow{0.5 \cdot \sigma_{Hsr} \cdot HSR} = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$$ *(hydrostatic loading to resisting side)*

$$(x_{Hs})_r := HSR \cdot \frac{1}{3} = 0.52\ ft$$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Lateral Earth Loading

$$(\sigma_{EHd})_{stem} := \sigma_{EHd} \cdot \left(\frac{HSD}{HSD + THEEL} \right) = \begin{bmatrix} 0.036 \\ 0 \end{bmatrix} ksf$$ *(lateral earth pressure loading to driving side of stem)*

$$EH_d := \overrightarrow{0.5 \cdot (\sigma_{EHd})_{stem} \cdot HSD} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} klf$$ *(lateral earth loading to driving side of stem)*

$$(x_{EH})_d := HSD \cdot \frac{1}{3} = 0.5\ ft$$ *(moment arm for lateral earth loading to the driving side from the base of the stem)*

$$(\sigma_{EHr})_{stem} := \sigma_{EHr} \cdot \frac{HSR}{HSR + TTOE} = \begin{bmatrix} 0.05 \\ 0.089 \end{bmatrix} ksf$$ *(lateral earth pressure loading to resisting side of stem)*

$$EH_r := \overrightarrow{0.5 \cdot (\sigma_{EHr})_{stem} \cdot HSR} = \begin{bmatrix} 0 \\ 0.1 \end{bmatrix} klf$$ *(lateral earth loading to resisting side of stem)*

$$(x_{EH})_r := HSR \cdot \frac{1}{3} = 0.5\ ft$$ *(moment arm for lateral earth loading to the resisting side from the base of the stem)*

Impact Loading

Note: reference DDR for discussion of impact loading on floodwalls.

$$IM := 0.1\ klf$$ *(impact loading to the top of the wall)*

$$(x_{IM})_d := HTS = 4\ ft$$ *(moment arm of the impact load from the base of the stem)*

Operating Equipment

$$p_{const} := 250\ psf$$ *(construction surcharge loading)*

Note: assume that the surcharge loading is uniform and that the resultant is mid-height of the soil depth.

$$d_{soil} := HSR = 1.6\ ft$$ *(depth of soil cover on stem)*

$$(x_Q)_d := \frac{HSR}{2} = 0.8\ ft$$ *(resultant height above base of stem)*

$$x_Q := d_{soil} - (x_Q)_d = 0.8\ ft$$ *(depth from resultant to top of soil)*

$\sigma_H := p_{const} \cdot k_0 = 0.133\ ksf$ *(lateral linear pressure applied to wall)*

$Q := \sigma_H \cdot HSR = 0.206\ klf$ *(lateral surcharge load to structure, usual case)*

Wind

Note: for conservative analysis, wind loading only applied to the driving side as a positive pressure or applied to the resisting side as a negative pressure.

$(p_w)_p := 30.4\ psf$ *(primary wind pressure, reference DDR)*

$(p_w)_c := 15.8\ psf$ *(companion wind pressure, reference DDR)*

$G := 0.85\ \frac{1}{ft}$ *(gust factor, ASCE 7-16 Section 26.11.1)*

$C_f := 1.3$ *(force coefficient from ASCE 7-16, figure 29.3-1, s/h = 1 and B/s = '>45')*

$d := HTS - HSR = 2.4\ ft$ *(exposed surface of the stem for wind pressure)*

$A_s := 1\ ft \cdot d = 2.4\ ft^2$ *(exposed surface area of the wall per linear foot)*

$W_c := (p_w)_c \cdot G \cdot C_f \cdot A_s = 0.043\ klf$ *(wind loading, usual case for companion loading)*

$W_p := (p_w)_p \cdot G \cdot C_f \cdot A_s = 0.082\ klf$ *(wind loading, unusual case)*

$(x_W)_d := HTS - \frac{d}{2} = 2.8\ ft$ *(resultant height above base of stem)*

Earthquake Loading: Equivalent Lateral Force

$D_{wall} := (TTS \cdot HTS) \cdot \gamma_{conc} = 0.9\ \frac{1}{ft} \cdot kip$ *(dead load of the stem of the wall)*

$I_e := 1.5$ *(importance factor, ASCE 7-16 Table 1.5-2)*

$R := 2.5$ *(response modification factor, ASCE 7-16, Table 12.2-1, Item G.3)*

$S_{DS} := 0.101$ *(seismic spectral acceleration for short period)*

$Cs := \frac{S_{DS}}{\frac{R}{I_e}} = 0.1$ *(seismic response coefficient, ASCE 7-16 12.8-2)*

$V := Cs \cdot D_{wall} = 0.1\ klf$ *(seismic base shear, ASCE 7-16, EQ 12.8-1)*

$EQ := V = 0.1\ klf$

$(x_{EQ}) := HSR = 1.6\ ft$ *(resultant height above base of stem)*

Load Combinations

Note: Usual design cases not considered here as the Usual pool level does not reach the base of the stem.

Driving Side Forces

$LC1_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Hs_d + IM) = \begin{bmatrix} 1 \\ 1 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot (EH_d + EQ) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot (EH_d + Q) = \begin{bmatrix} 0.4 \\ 0.3 \end{bmatrix} klf$ *(load case 3 for construction)*

Resisting Side Forces

$LC1_r := (\gamma_p)_{Serve.n} \cdot (EH_r + Hs_r - W_c) = \begin{bmatrix} 0.1 \\ 0.2 \end{bmatrix} klf$ *(load case 1 for maximum hydrostatic)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(load case 2a,2b for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot (EH_r) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} klf$ *(load case 4 for construction)*

Resultant Shear at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 0.9 \\ 0.8 \end{bmatrix} klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.1 \\ 0 \end{bmatrix} klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.3 \\ 0.2 \end{bmatrix} klf$ *(equilibrium load case 3)*

$\Sigma F_{stem} := \text{stack}(LC1, LC2, LC3)$

Note: Maximum factored shear loading to the stem is the governing loading for the river side steel design. Minimum factored shear loading is the governing landside steel design.

$(Vu)_{stem} := \max(\Sigma F_{stem}) = 0.9\ klf$ *(resultant factored shear at base of floodwall stem, driving side)*

Resultant Moment - Driving Side

$LC1_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d\right) + \left(Hs_d \cdot (x_{Hs})_d\right) + IM \cdot (x_{IM})_d\right) = \begin{bmatrix} 1.7 \\ 1.7 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum hydrostatic)*

$LC2_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d\right) + \left(EQ \cdot (x_{EQ})\right)\right) = \begin{bmatrix} 0.2 \\ 0.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_d := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_d \cdot (x_{EH})_d\right) + \left(Q \cdot \left((x_Q)_d\right)\right)\right) = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction)*

Resultant Moment - Resisting Side

$LC1_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r\right) + \left(Hs_r \cdot (x_{Hs})_r\right)\right) = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 1 for maximum flooding)*

$LC2_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r\right)\right) = \begin{bmatrix} 0.03 \\ 0.06 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 2 for earthquake loading)*

$LC3_r := (\gamma_p)_{Serve.n} \cdot \left(\left(EH_r \cdot (x_{EH})_r\right)\right) = \begin{bmatrix} 0.03 \\ 0.06 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case 3 for construction or surcharge loading)*

Resultant Moment at Base

$\Sigma F = V - \Sigma F_d + \Sigma F_r$ *(equilibrium equation)*

$LC1 := LC1_d - LC1_r = \begin{bmatrix} 1.6 \\ 1.6 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 1)*

$LC2 := LC2_d - LC2_r = \begin{bmatrix} 0.1 \\ 0.1 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 2)*

$LC3 := LC3_d - LC3_r = \begin{bmatrix} 0.2 \\ 0.2 \end{bmatrix} ft \cdot klf$ *(equilibrium load case 3)*

$\Sigma M_{stem} := \text{stack}(LC1, LC2, LC3)$ *(resultant factored moment at base of floodwall stem)*

$(Mu)_{stem} := \max(\Sigma M_{stem}) \cdot 1\ ft = 1.6\ kip \cdot ft$ *(resultant moment at base of floodwall stem, riverside steel design)*

Design Check Function

(system check for capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v\left(X_U, X_N\right) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left(X_U\right) \\ \quad \left\| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left\| ch_{i,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check\left(X_U, X_N\right) := \left\| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left\| ch_{1,1} \leftarrow \text{“N/A”} \right. \\ \text{else} \\ \quad \left\| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

$b := 12\ in$ *(strip width for analysis)*

Stem Reinforcement

Vertical Bars

$(d_b)_{stem.vert} :=$ Bar Size: #8 $= 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} :=$ 3″ cover *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$d_{stem.vert} := TTS - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in stem)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \text{if } f'_c \geq 8000\ psi \quad 0.65 \quad \text{else if } f'_c > 4000\ psi \quad 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) \quad \text{else} \quad 0.85 = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}} = 0.85\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{stem.vert} := \phi_b \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.7\ kip \cdot ft$
(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom})_{Stem} := Check\left((Mu)_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{"OK"} \quad 0.03]$ *(nominal flexural check, with Utilization ratio)*

Minimum Required Steel for Flexure

Solve Constraints Given Values

$As_{req} := 1.5\ in^2$

$(Mu)_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.025\ in^2$

$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.025\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTS \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.033\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{stem.req} := \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \quad as \leftarrow (A_{s.min})_{stem.ten} \quad \text{else} \quad as \leftarrow A_{s.min} = 0.025\ in^2$
(required steel to meet steel requirements)
(includes ACI 318-19 9.6.1.2(a) and (b))
(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{stem.vert} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.03]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$(Steel\_Check_{Vert.ts})_{Stem} := Check\left((A_{s.min})_{temp}, (A_s)_{stem.vert}\right) = [\text{"OK"} \quad 0.4]$ *(check stem reinforcement to temperature and shrinkage requirements)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{stem.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot 12\ in}{s_{stem.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in stem)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot \left(TTS \cdot b\right)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{stem.lat} = 0.4\ in^2$ *(area of steel provided in the lateral direction for the stem)*

$(Steel\_Check_{Lat})_{Stem} := Check\left(A_{s.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check stem reinforcement to required reinforcement)*

Riverside Reinforcement

Primary Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Deflection Check

Note: Reference provided justification file for loading conditions and attached EnerCalc calculations for the deflections in floodwall stem since stem thickness is less than H/8. From EnerCalc report, wall deflections limited to H/6838, which is less than expected movement capacity in waterstop and wall joints. 4 ft tall floodwalls to have stems with above detailed reinforcement.

Landside Reinforcement

For design simplicity, landside reinforcement to match both primary and lateral reinforcement.

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 4 ft Floodwall Stem

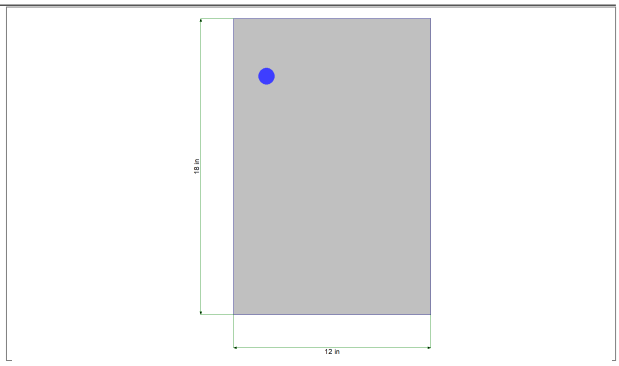
CODE REFERENCES

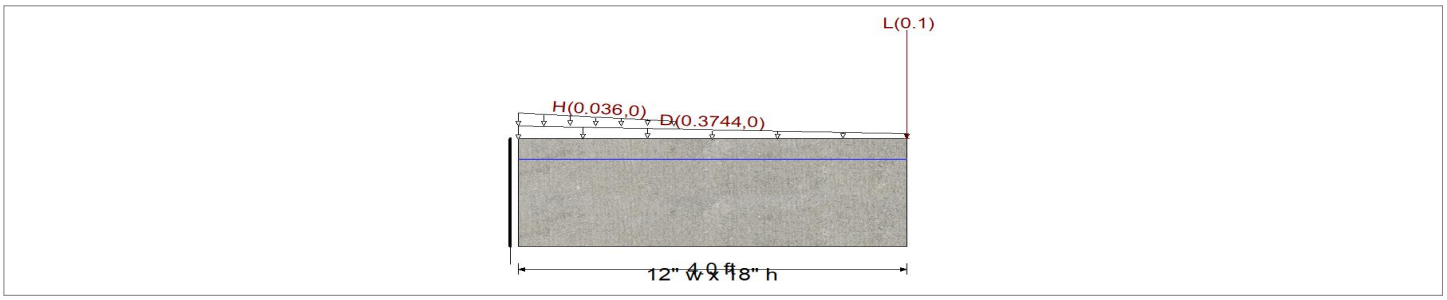
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	=	4.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	503.12 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.8250
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		
			Number of Resisting Legs Per Stirrup =		

Seismic Design Category = B





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
1-#8 at 3.50 in from Top, from 0.0 to 4.0 ft in this span

Load for Span Number 1

Varying Uniform Load : D= 0.3744->0.0 k/ft, Extent = 0.0 -->> 4.0 ft, Trib Width = 1.0 ft, (Hydrostatic Loading - Driving side)

Varying Uniform Load : H= 0.0360->0.0 k/ft, Extent = 0.0 -->> 1.60 ft, Trib Width = 1.0 ft, (Lateral Earth Pressure - Driving Side)

Point Load : L = 0.10 k @ 4.0 ft, (Impact Loading - Driving Side)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.045** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	-2.262	k-ft
Mn * Phi : Allowable	49.712	k-ft
Location of maximum on span	0.000	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	
Max Downward Total Deflection	0.000 in	Ratio =	0 <180.0	Span: 1 : +D+L+H
Max Upward Total Deflection	0.000 in	Ratio =	0 <180.0	Span: 1 : +D+L+H

Cross Section Strength & Inertia

Top & Bottom references are for tension side of sect

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Bottom	Top	I gross	Icr - Bottom	Icr - Top
Section 1	1- #8 @ d=3.5",	0.00	49.71	5,832.00	42.82	1,058.39

Project Title: Upper Turkey Creek
Engineer: John Westbrook
Project ID:
Project Descr: Floodwall Design

Concrete Beam

Project File: utc floodwall.ec6

LIC# : KW-06020622, Build:20.24.02.03 U.S. ARMY CORPS OF ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: 4 ft Floodwall Stem

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.404	
Max Upward from Load Combinations	1.404	
+1.60D+1.60L+1.60H	1.404	
+1.60D+1.60H	1.244	
+1.60H	0.046	

Shear Stirrup Requirements

Entire Beam Span Length : Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	4.000	-2.26	49.71	0.05
+1.40D+1.60H					
Span # 1	1	4.000	-1.42	49.71	0.03
+1.60D+1.60L+1.60H					
Span # 1	1	4.000	-2.26	49.71	0.05
+1.60D+1.60H					
Span # 1	1	4.000	-1.62	49.71	0.03
+1.60H					
Span # 1	1	4.000	-0.02	49.71	0.00
+1.60L					
Span # 1	1	4.000	-0.64	49.71	0.01

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft	Load Combination	Max. "+" Defl (in)	Location in Span (ft
+D+L+H	1	0.0005	4.000		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
+D+H	1	0.0003	4.000	0.0000	0.000
+D+L+H	1	0.0005	4.000	0.0000	0.000
H Only	1	0.0000	4.000	0.0000	0.000

STEM SHEAR CONNECTION TO FOUNDATION

Shear Friction

$f_y := (F_y)_{60} = 60000\ psi$ *(yield strength of reinforcing steel)*

$\mu := 1.0$

$(d_b)_{stem.vert.2} :=$ Bar Size: #6 $= 0.75\ in$ *(vertical bar size in stem)*

$s_{stem.vert2} := 12\ in$

$A_{vf} := \left(\frac{\pi}{4}\right) \cdot (d_b)_{stem.vert.2}{}^2 \cdot \left(\frac{b}{s_{stem.vert2}}\right) = 0.4\ in^2$

$V_n := \mu \cdot A_{vf} \cdot f_y = 26.5\ kip$ *(shear-friction strength, ACI 318-19, eq 22.9.4.2)*

Table 22.9.4.2—Coefficients of friction

Contact surface condition	Coefficient of friction μ[1]	
Concrete placed monolithically	1.4λ	(a)
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ	(b)
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ	(c)
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ	(d)

[1]λ = 1.0 for normalweight concrete. For lightweight concrete, λ is calculated as given in 19.2.4, but shall not exceed 0.85.

Table 22.9.4.4—Maximum V_n across the assumed shear plane

Condition	Maximum V_n		
Normalweight concrete placed monolithically or placed against hardened concrete intentionally roughened to a full amplitude of approximately 1/4 in.	Least of (a), (b), and (c)	$0.2f'_cA_c$	(a)
		$(480 + 0.08f'_c)A_c$	(b)
		$1600A_c$	(c)
Other cases	Lesser of (d) and (e)	$0.2f'_cA_c$	(d)
		$800A_c$	(e)

Maximum Shear Friction

$A_c := b \cdot TTS = 1.5\ ft^2$

$(V_n)_a := 0.2\ f'_c \cdot A_c = 194.4\ kip$ *(shear-friction maximum, equation a, Table 22.9.4.4)*

$(V_n)_b := \left(480\ psi + 0.08 \cdot f'_c\right) \cdot A_c = 181.4\ kip$ *(shear-friction maximum, equation b, Table 22.9.4.4)*

$(V_n)_c := \left(16000\ psi\right) A_c = 3456\ kip$ *(shear-friction maximum, equation c, Table 22.9.4.4)*

$(V_n)_{max} := \max\left((V_n)_a, (V_n)_b, (V_n)_c\right) = 3456\ kip$ *(shear-friction maximum, Table 22.9.4.4)*

Shear Strength

$\phi_V = 0.75$ *(reduction factor for shear capacity)*

$\phi_V V_n := \phi_V \cdot min\left((V_n)_{max}, V_n\right) = 19.9\ kip$ *(factored shear-friction strength)*

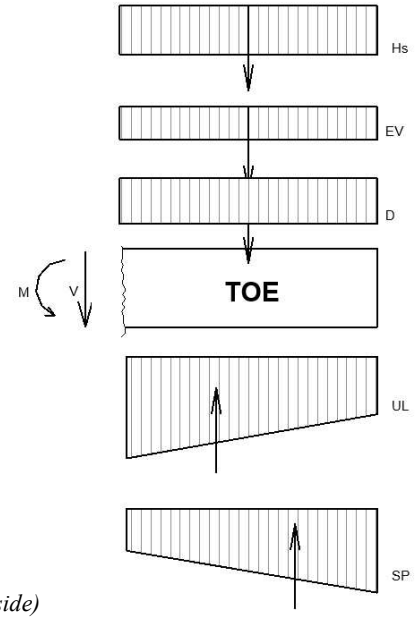
$V_{stem} := (Vu)_{stem} \cdot b = 0.9\ kip$ *(calculated shear required at base of stem)*

$V_{check} := Check\left(V_{stem}, \phi_V V_n\right) = [\text{“OK”} \quad 0.04]$ *(shear check, utilization ratio)*

Toe Check

Loads to Toe

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



Dead Load

$\sigma_D := \gamma_{conc} \cdot TTOE = 225\ psf$

$D := \sigma_D \cdot TWIDTH = 0.7\ klf$ *(resultant dead load of toe)*

$(x_D)_{toe} := \frac{TWIDTH}{2} = 1.5\ ft$ *(moment arm of the dead loading from the connection of the toe to the foundation)*

Hydrostatic Loads Above Toe

Note: Groundwater assumed to the soil surface. Potential additional seepage pressure conservatively ignored.

$\sigma_{Hsr} := \begin{bmatrix} \gamma_{water} \cdot HSR \\ \gamma_{water} \cdot HSR \end{bmatrix} = \begin{bmatrix} 96.7 \\ 96.7 \end{bmatrix} psf$ *(hydrostatic pressure loading to resisting side)*

$Hs_r := \overrightarrow{TWIDTH \cdot \sigma_{Hsr}} = \begin{bmatrix} 0.3 \\ 0.3 \end{bmatrix} klf$ *(Resultant hydrostatic loading to resisting side)*

$(x_{Hs})_r := TWIDTH \cdot 0.5 = 1.5\ ft$ *(moment arm for hydrostatic loading to the resisting side from the base of the stem)*

Vertical Earth Pressure

$\sigma_{EV} := \gamma'_{soil} \cdot HSR = 89.3\ psf$

$EV_r := \begin{bmatrix} \sigma_{EV} \cdot TWIDTH \\ \sigma_{EV} \cdot TWIDTH \end{bmatrix} = \begin{bmatrix} 0.268 \\ 0.268 \end{bmatrix} klf$ *(buoyant force of vertical soil loading)*

$(x_{EV})_r := \frac{TWIDTH}{2} = 1.5\ ft$

Soil Pressure

$(\sigma_{SP})_{max} = \begin{bmatrix} -0.654 \\ -0.624 \end{bmatrix} ksf$ *(max soil loading from CT Wall Output)*

$BWIDTH = 8\ ft$ *(total length of base of wall)*

$BIC_{percent} := \begin{bmatrix} 89.61 \\ 75.35 \end{bmatrix}\%$ *(percent of the base in compression)*

$BIC := BWIDTH \cdot BIC_{percent} = \begin{bmatrix} 7.2 \\ 6 \end{bmatrix} ft$ *(length of base in compression)*

$TWIDTH = 3\ ft$ *(length of toe of the foundation)*

note: using the length of the toe to determine the soil loading at the the connection of the toe to the stem and heel, use similar triangles to determine the loading at the connection of the toe.

$NTWIDTH := BIC - TWIDTH = \begin{bmatrix} 4.2 \\ 3 \end{bmatrix} ft$ *(length of the not-toe of the foundation in compression)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{NTWIDTH}{(\sigma_{SP})_1}$ *(relationship of similar triangles)*

$(\sigma_{SP})_1 := \overrightarrow{NTWIDTH \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} -0.38 \\ -0.31 \end{bmatrix} ksf$ *(soil pressure loading at the connection of the toe)*

$SP_{toe} := \frac{TWIDTH \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)}{2} = \begin{bmatrix} -1.6 \\ -1.4 \end{bmatrix} klf$ *(resultant soil loading to the toe of the foundation)*

$$(x_{SP})_{toe} := \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{SP})_{max}\right) + (\sigma_{SP})_1\right)}{3 \cdot \left((\sigma_{SP})_{max} + (\sigma_{SP})_1\right)} = \begin{bmatrix} 1.6 \\ 1.7 \end{bmatrix} ft$$

(moment arm from the connection of the toe to the resultant of the soil pressure loading)

Uplift Pressures

$$(\sigma_{UL})_{min} = \begin{bmatrix} 0.221 \\ 0.231 \end{bmatrix} ksf$$

(minimum uplift pressure at the end of the toe)

$$(\sigma_{UL})_{max} = \begin{bmatrix} 0.285 \\ 0.313 \end{bmatrix} ksf$$

(maximum uplift pressure behind the key)

Note: Uplift pressure drops off linearly from the back of the key to the toe of the foundation

$$BWIDTH = 8\ ft$$

(foundation width)

$$TK = 1\ ft$$

(key width)

$$L := BWIDTH - TK = 7\ ft$$

(length of linear uplift pressure)

$$(\sigma_{UL})_{max} - (\sigma_{UL})_{min} = m(L)$$

(linear relationship)

$$m := \frac{(\sigma_{UL})_{max} - (\sigma_{UL})_{min}}{L} = \begin{bmatrix} 0.0091 \\ 0.0117 \end{bmatrix} \frac{ksf}{ft}$$

(linear relationship)

$$(\sigma_{UL})_{max.toe} := \overrightarrow{(m \cdot TWIDTH) + (\sigma_{UL})_{min}} = \begin{bmatrix} 0.248 \\ 0.266 \end{bmatrix} ksf$$

(uplift pressure at the connection of the toe to the foundation)

$$UL_{toe} := \frac{-TWIDTH \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)}{2} = \begin{bmatrix} -0.7 \\ -0.7 \end{bmatrix} klf$$

(uplift pressure resultant for the toe of the foundation)

$$(x_{UL})_{toe} := TWIDTH - \frac{TWIDTH \cdot \left(\left(2 \cdot (\sigma_{UL})_{max.toe}\right) + (\sigma_{UL})_{min}\right)}{3 \cdot \left((\sigma_{UL})_{max.toe} + (\sigma_{UL})_{min}\right)} = \begin{bmatrix} 1.5 \\ 1.5 \end{bmatrix} ft$$

(distance from the connection of the toe to the resultant of the uplift pressure)

Load Combinations

Resisting Side Forces - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \left(D + EV_r + Hs_r + SP_{toe} + UL_{toe}\right) = \begin{bmatrix} -1.6 \\ -1.5 \end{bmatrix} klf$$

(load case 1 for infrequent flooding)

$$\Sigma F_{toe} := \max\left(\overrightarrow{|LC1|}\right) = 1.6\ klf$$

(maximum of the drained vs undrained cases)

Factored Shear on Toe at Face of Stem

$$(V_u)_{toe} := \Sigma F_{toe} = 1.6\ klf$$

(maximum factored shear in toe)

Resultant Moment - on Toe

$$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(D \cdot (x_D)_{toe}\right) + \left(\overrightarrow{Hs_r \cdot (x_{Hs})_r}\right) \\ + \left(\overrightarrow{EV_r \cdot (x_{EV})_r}\right) + \left(\overrightarrow{SP_{toe} \cdot (x_{SP})_{toe}}\right) \\ + \left(\overrightarrow{UL_{toe} \cdot (x_{UL})_{toe}}\right) \end{pmatrix} = \begin{bmatrix} -2.7 \\ -2.5 \end{bmatrix} \frac{kip \cdot ft}{ft}$$

(load case 1 for infrequent flooding)

Maximum Factored Moment in Toe at Face of Stem

$$Mu_{toe} := \max(LC1) \cdot 1\ ft = -2.5\ kip \cdot ft$$

(resultant moment at base of floodwall stem per foot of wall)

$$Mu_{toe} := -Mu_{toe}$$

Toe Reinforcement

Bottom Bars - Main Steel

$(d_b)_{toe.bot} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(bottom bar size in toe)*

$s_{toe.bot} := 12\ in$ *(bottom bar spacing in toe)*

$c_{toe.bot} := \boxed{4''\ \text{cover}}$ *(cover for bottom reinforcement in toe)*

$(A_s)_{toe.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{toe.bot}^{\ 2} \cdot b}{s_{toe.bot}} = 0.785\ in^2$ *(area of steel for bottom reinforcement in toe)*

$d_{toe.bot} := TTOE - c_{toe.bot} - \left(0.5 \cdot (d_b)_{toe.bot}\right) = 13.5\ in$ *(depth to reinforcement bottom reinforcement in toe)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{toe.bot} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot \langle f'_c - 4000\ psi \rangle}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{toe.bot} := \dfrac{(A_s)_{toe.bot} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{toe.bot}} = 0.9\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$(\phi M_n)_{toe.bot} := \phi_b \cdot (A_s)_{toe.bot} \cdot (F_y)_{60} \cdot \left(d_{toe.bot} - \dfrac{a_{toe.bot}}{2}\right) = 46.1\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{toe} := Check\left(Mu_{toe}, (\phi M_n)_{toe.bot}\right) = [\text{"OK"} \quad 0.06]$ *(nominal flexural check)*

Minimum Required Steel for Flexure

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$Mu_{toe} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \dfrac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{stem.vert}}\right)$

$(As_{req})_{toe} := \text{find}\left(As_{req}\right) = 0\ in^2$ *(minimum required reinforcement for flexural strength)*

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{toe.bot} = 0.5\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (As_{req})_{toe} \cdot \frac{4}{3} = 0.1\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{toe.req} := \text{if } A_{s\_1.3} < (A_s)_{toe.bot} \ \| \ as \leftarrow (As_{req})_{toe} \ \text{else} \ \| \ as \leftarrow A_{s.min} = 0.039\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{toe.bot} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the toe)*

$(Steel\_Check_{bot})_{toe} := Check\left((A_s)_{toe.req}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0]$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} = 0.3\ in^2$

$(Steel\_Check_{bot.ts})_{toe} := Check\left((A_{s.min})_{temp}, (A_s)_{toe.bot}\right) = [\text{"OK"} \quad 0.4]$

(check stem reinforcement to required t&s reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{toe.bot}}{\left(b \cdot d_{toe.bot}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{toe.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in toe)*

$s_{toe.lat} := 12\ in$ *(lateral bar spacing in toe)*

$c_{toe.lat} :=$ 4″ cover *(cover for lateral reinforcement in toe)*

$(A_s)_{toe.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{toe.lat}^{\ 2} \cdot 12\ in}{s_{toe.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in toe)*

$d_{toe.lat} := TTS - c_{toe.lat} - (d_b)_{toe.bot} - 0.5 \cdot (d_b)_{toe.lat} = 12.625\ in$ *(depth to reinforcement lateral reinforcement in toe)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.32\ in^2$

$(A_s)_{toe.lat} = 0.442\ in^2$ *(area of steel provided in the lateral direction for the toe, both faces)*

$(Steel\_Check_{Lat})_{toe} := Check\left(A_{s.req}, (A_s)_{toe.lat}\right) = [\text{"OK"} \quad 0.73]$ *(check toe reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{toe.lat}}{\left(b \cdot d_{toe.bot}\right)} = 0.0027$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Lat})_{toe} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.4]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

SHEAR CHECK

$N_u := 0\ kip$ *(compressive axial force in toe of foundation, conservatively ignored for analysis)*

$A_g := TTOE \cdot b = 216\ in^2$ *(gross area of concrete section)*

$V_c := \left(\left(2 \cdot \sqrt{f'c}\right) \cdot psi + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d_{toe.bot} = 21.7\ kip$ *(shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$\phi_V V_n := \phi_V \cdot V_c = 16.3\ kip$ *(nominal shear strength in concrete, EM 1110-2-2104, Eq 5-1)*

$V_{toe} := \left|(V_u)_{toe} \cdot b\right| = 1.6\ kip$

$V_{toe.check} := Check\left(V_{toe}, \phi_V V_n\right) = [\text{“OK”} \quad 0.1]$

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

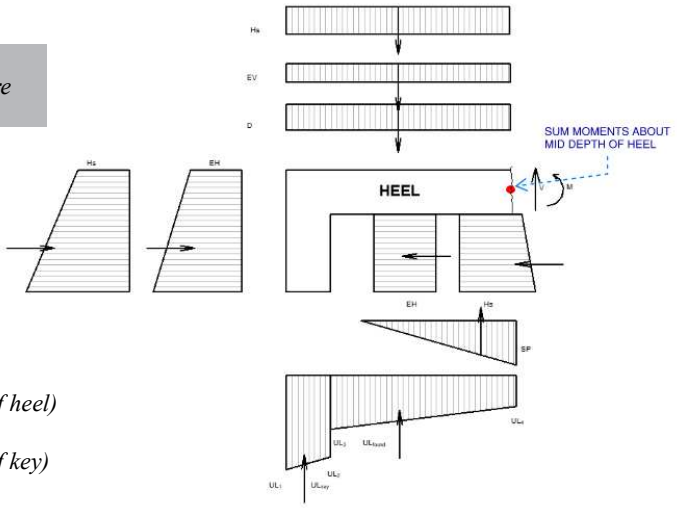
Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN

Loads to Heel

Note: largest loading from drained and undrained cases used to create conservative design for loading. In each loading condition, drained loads are listed first and undrained last



$HWIDTH := BWIDTH - TTS - TWIDTH = 3.5\ ft$

(Length of heel, includes key)

Dead Load

$(\sigma_D)_{Heel} := \gamma_{conc} \cdot (TTOE) = 225\ psf$

$(\sigma_D)_{Key} := \gamma_{conc} \cdot (HK) = 150\ psf$

$D_{Heel} := (\sigma_D)_{Heel} \cdot HWIDTH = 0.8\ klf$ *(resultant dead load of heel)*

$D_{Key} := (\sigma_D)_{Key} \cdot TK = 0.2\ klf$ *(resultant dead load of key)*

$D := D_{Heel} + D_{Key} = 0.9\ klf$ *(resultant dead load of heel including key per linear foot of wall)*

$(x_{D.heel})_{heel} := \frac{HWIDTH}{2} = 1.8\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_{D.key})_{heel} := HWIDTH - \left(\frac{TK}{2}\right) = 3\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation)*

$(x_D)_{heel} := \frac{D_{Heel} \cdot (x_{D.heel})_{heel} + D_{Key} \cdot (x_{D.key})_{heel}}{D_{Heel} + D_{Key}} = 2\ ft$ *(moment arm of the dead loading from the connection of the heel to the foundation including the key)*

Hydrostatic Loads - Top of Heel

Note: hydrostatic loads applied in three directions, notated as ah = above heel, fh = face of heel and bh = behind heel

$(\sigma_{Hs})_{ah} := \gamma_{water} \cdot HTS = 0.25\ ksf$ *(vertical water pressure from hydrostatic loading)*

$(Hs)_{ah} := (\sigma_{Hs})_{ah} \cdot HWIDTH = 0.9\ klf$ *(vertical water loading from hydrostatic loading)*

$(x_{Hs})_{ah} := \frac{HWIDTH}{2} = 1.75\ ft$ *(moment arm for vertical hydrostatic loading)*

Hydrostatic Loads - Face of Heel

$(\sigma_{Hs})_{fh.top} := \gamma_{water} \cdot HTS = 0.25\ ksf$ *(min horizontal hydrostatic loading, top of foundation)*

$(\sigma_{Hs})_{fh.bot} := \gamma_{water} \cdot (HTS + THEEL) = 0.406\ ksf$ *(min horizontal hydrostatic loading, bot of key)*

$THEEL = 2.5\ ft$ *(depth of applied loading)*

$(Hs)_{fh} := \frac{THEEL \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{2} = 0.8\ klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{fh.bot} + (\sigma_{Hs})_{fh.top}\right)}\right) = 2.1\ ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Hydrostatic Loads - Back of Heel

$(\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.357 \\ 0.388 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.366 \\ 0.399 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 1\ ft$ *(depth of key from the back side face)*

$(Hs)_{bh} := \frac{HK \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{2} = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of heel)*

$(x_{Hs})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}{3 \cdot \left((\sigma_{Hs})_{bh.bot} + (\sigma_{Hs})_{bh.top}\right)}\right) = \begin{bmatrix} 1.3 \\ 1.3 \end{bmatrix} ft$ *(moment arm for the hydrostatic loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Face of Heel

$(\sigma_{EH})_{fh.top} := \gamma'_{soil} \cdot k_0 \cdot HSD = 0.047\ ksf$ *(min horizontal soil loading, top of foundation)*

$(\sigma_{EH})_{fh.bot} := \gamma'_{soil} \cdot k_0 \cdot (HSD + THEEL) = 0.124\ ksf$ *(min horizontal soil loading, top of foundation)*

Note: conservatively the lateral earth pressure does not include the undrained case where the lateral soil pressure would be zero.

$THEEL = 2.5\ ft$ *(depth of applied loading)*

$(EH)_{fh} := \frac{THEEL \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{2} = 0.2\ klf$ *(resultant of the soil loading to the front face of heel)*

$(x_{EH})_{fh} := \frac{TTOE}{2} + \left(\frac{THEEL \cdot \left(2 \cdot (\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}{3 \cdot \left((\sigma_{EH})_{fh.bot} + (\sigma_{EH})_{fh.top}\right)}\right) = 2.186\ ft$ *(moment arm for the soil loading to the front face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Lateral Earth Pressures - Back of Heel

$(\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.02 \\ 0.75 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.02 \\ 0.75 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 1\ ft$ *(depth of key from the back side face)*

$(EH)_{bh} := \frac{HK \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{2} = \begin{bmatrix} 1 \\ 0.8 \end{bmatrix} klf$ *(resultant of the horizontal soil loading to the back face of heel)*

$(x_{EH})_{bh} := \frac{TTOE}{2} + \left(\frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}{3 \cdot \left((\sigma_{EH})_{bh.bot} + (\sigma_{EH})_{bh.top}\right)}\right) = \begin{bmatrix} 1.3 \\ 1.3 \end{bmatrix} ft$ *(moment arm for the soil loading to the back face of the heel; NOTE: negative value here indicates that resultant is below the base of the foundation)*

Vertical Earth Pressures - Top of Heel

$(\sigma_{EV})_{ah} := \gamma'_{soil} \cdot HSD = 0.089\ ksf$ *(vertical buoyant soil pressure on heel)*

$(EV)_{ah} := (\sigma_{EV})_{ah} \cdot HWIDTH = 0.31\ klf$ *(vertical buoyant soil loading on heel)*

$(x_{EV})_{ah} := \frac{HWIDTH}{2} = 1.75\ ft$ *(moment arm for vertical soil loading)*

Soil Pressure Loading - Base of Heel

$L_0 := BWIDTH - BIC = \begin{bmatrix} 0.83 \\ 1.97 \end{bmatrix} ft$ *(length of base with no soil pressure loading, measured from front face of key back toward toe)*

$L := HWIDTH - L_0 = \begin{bmatrix} 2.67 \\ 1.53 \end{bmatrix} ft$ *(length of base with soil pressure loading, measured from beginning of pressure back toward connection)*

$\frac{BIC}{(\sigma_{SP})_{max}} = \frac{L}{(\sigma_{SP})_{heel}}$ *(similar triangles relationship between the total soil pressure load and the length of the base in compression to the length of the heel in compression to the maximum soil pressure loading at the connection)*

$(\sigma_{SP})_{heel} := \overrightarrow{-L \cdot \frac{(\sigma_{SP})_{max}}{BIC}} = \begin{bmatrix} 0.24 \\ 0.16 \end{bmatrix} ksf$ *(maximum soil pressure loading at the connection of the heel)*

$SP_{heel} := \overrightarrow{0.5 \cdot (\sigma_{SP})_{heel} \cdot L} = \begin{bmatrix} 0.32 \\ 0.12 \end{bmatrix} klf$ *(resultant soil pressure loading on the heel)*

$(x_{SP})_{heel} := \frac{1}{3} L = \begin{bmatrix} 0.9 \\ 0.5 \end{bmatrix} ft$ *(moment arm for the soil pressure loading, measured from the connection of the heel)*

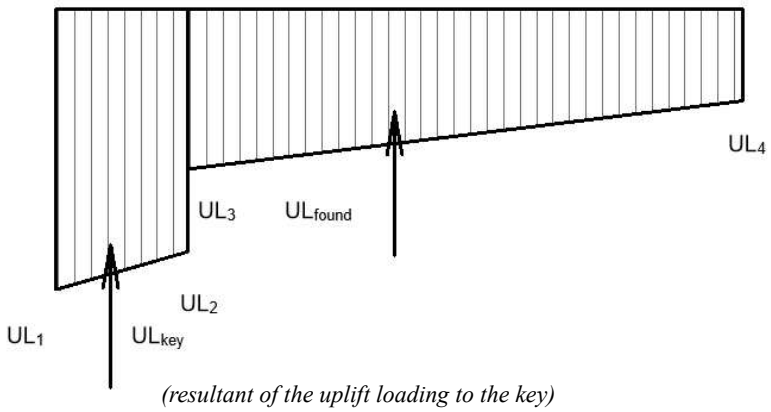
Uplifting Pressure

$UL_1 := \begin{bmatrix} 1.049 \\ 1.217 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_2 := \begin{bmatrix} 1.009 \\ 1.165 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_3 := \begin{bmatrix} 0.722 \\ 0.856 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*

$UL_4 := \begin{bmatrix} 0.406 \\ 0.44 \end{bmatrix} ksf$ *(uplift pressure, from CT Wall output)*



$UL_{key} := \frac{TK \cdot (UL_1 + UL_2)}{2} = \begin{bmatrix} 1 \\ 1.2 \end{bmatrix} klf$ *(resultant of the uplift loading to the key)*

$Heel\_Width := HWIDTH - TK = 2.5\ ft$ *(length of the heel; from the connection of the stem to the back face of the key)*

$(x_{UL})_{key} := Heel\_Width + \left(\frac{TK \cdot (2 \cdot UL_1 + UL_2)}{3 \cdot (UL_1 + UL_2)} \right) = \begin{bmatrix} 3 \\ 3 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

$m := \frac{(UL_4 - UL_3)}{BWIDTH - TK} = \begin{bmatrix} -0.0451 \\ -0.0594 \end{bmatrix} \frac{ksf}{ft}$ *(linear relationship between the uplift pressure from the foundation behind the key)*

$UL_{stem} := m \cdot (HWIDTH - TK) + UL_3 = \begin{bmatrix} 0.6 \\ 0.7 \end{bmatrix} ksf$ *(uplift pressure at the stem)*

$UL_{found} := \frac{Heel\_Width \cdot (UL_3 + UL_{stem})}{2} = \begin{bmatrix} 1.7 \\ 2 \end{bmatrix} klf$ *(resultant of the uplift loading to the foundation; less the key)*

$(x_{UL})_{found} := \left(\frac{Heel\_Width \cdot (2 \cdot UL_3 + UL_{stem})}{3 \cdot (UL_3 + UL_{stem})} \right) = \begin{bmatrix} 1.29 \\ 1.29 \end{bmatrix} ft$ *(moment arm for the uplift force on the key)*

Load Combinations

Note: Forces that are oriented to the left and up are considered negative forces and moments that would rotate the heel counterclockwise are positive for the purpose of summing forces and moments.

Forces on Heel - Horizontal Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} ((EH)_{fh} - (EH)_{bh}) \\ + ((Hs)_{fh} - (Hs)_{bh}) \end{pmatrix} = \begin{bmatrix} -0.6 \\ -0.2 \end{bmatrix} klf$ *(Summation of the forces in the horizontal direction note positive forces represent heel in compression)*

$(\Sigma F_{heelc})_{horz} := \max(LC1) = -0.2\ klf$

$(\Sigma F_{heel})_{horz} := \max(\overrightarrow{|LC1|}) = 0.6\ klf$ *(maximum of the drained vs undrained cases)*

$P\_Check :=$ if $(\Sigma F_{heelc})_{horz} > 0$
"Heel is in compression, conservatively ignore axial loads in moment capcacity check"
else
"Heel in tension, factored into moment capcacity check"

$P\_Check =$ "Heel in tension, factored into moment capcacity check"

$(\Sigma F_{heel})_{Tension} :=$ if $(\Sigma F_{heel})_{horz} > 0$
$0\ kip$
else
$|(\Sigma F_{heel})_{horz}| \cdot b$
$= 0\ kip$

Forces on Heel - Vertical Forces

$LC1 := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} D + (EV)_{ah} + (Hs)_{ah} \\ + \left(UL_{key} + UL_{found}\right) + \left(SP_{heel}\right) \end{pmatrix} = \begin{bmatrix} 8.2 \\ 8.6 \end{bmatrix} klf$ *(summation of the forces in the vertical direction)*

$(\Sigma F_{heel})_{vert} := \max\left(LC1\right) = 8.6\ klf$ *(maximum of the drained vs undrained cases)*

Resultant Shear at Base

$Vu_{heel} := (\Sigma F_{heel})_{vert} = 8.6\ klf$ *(factored shear at connection of floodwall heel)*

Resultant Axial Load in Base

$(P_u)_{heel} := (\Sigma F_{heel})_{horz} = 0.6\ klf$ *(factored axial load at floodwall heel)*

$(\Sigma F_{heel})_{Tension} = 0\ ft \cdot klf$ *(tension in heel; from check above, zero if heel in compression)*

Note: Axial loads with positive values put the heel of the floodwall in compression and negative values put the heel of the floodwall in tension.

Resultant Moment - on Heel at Stem Face

$LC1_{pos} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{D \cdot (x_D)_{heel}}\right) \\ + \left(\overrightarrow{(EV)_{ah} \cdot (x_{EV})_{ah}}\right) + \left(\overrightarrow{(Hs)_{ah} \cdot (x_{Hs})_{ah}}\right) \\ + \left(\overrightarrow{(EH)_{fh} \cdot (x_{EH})_{fh}}\right) + \left(\overrightarrow{(Hs)_{fh} \cdot (x_{Hs})_{fh}}\right) \end{pmatrix} = 9.7\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{neg} := (\gamma_p)_{Serve.n} \cdot \begin{pmatrix} \left(\overrightarrow{(Hs)_{bh} \cdot (x_{Hs})_{bh}}\right) \\ + \left(\overrightarrow{(EH)_{bh} \cdot (x_{EH})_{bh}}\right) \\ + \left(\overrightarrow{SP_{heel} \cdot (x_{SP})_{heel}}\right) \\ + \left(\overrightarrow{UL_{key} \cdot (x_{UL})_{key}}\right) \\ + \left(\overrightarrow{UL_{found} \cdot (x_{UL})_{found}}\right) \end{pmatrix} = \begin{bmatrix} 11.6 \\ 12.2 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$\Sigma LC := \max\left(LC1_{pos} - LC1_{neg}\right) = -1.85\ kip$

$Mu_{heel} := \max\left(LC1_{pos} - LC1_{neg}\right) \cdot b = -1.8\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

Note: since section is under tension and flexure, heel to be designed per section B-4 of EM 1110-2-2104.

Heel Reinforcement

$Mu_{heel} = -1.8\ kip \cdot ft$ *(maximum factored moment at base of floodwall stem on the heel)*

$(P_u)_{heel} = 0.6\ \frac{kip}{ft}$ *(maximum factored axial tension at base of floodwall stem on the heel)*

$Vu_{heel} = 8.6\ \frac{kip}{ft}$ *(maximum factored shear at base of floodwall stem on the heel)*

Top Bars - Main Steel

$(d_b)_{heel.top} :=$ Bar Size: #8 $= 1\ in$ *(top bar size in heel)*

$(d_b)_{heel.bot} :=$ Bar Size: #8 $= 1\ in$ *(bottom bar size in heel)*

$s_{heel.top} := 12\ in$ *(top bar spacing in heel)*

$s_{heel.bot} := 12\ in$ *(bottom bar spacing in heel)*

$c_{heel.top} := 3''\ \text{cover}$ *(cover for top reinforcement in heel)*

$c_{heel.bot} := 4''\ \text{cover}$ *(cover for bottom reinforcement in heel)*

$(A_s)_{heel.top} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{heel.top}^{\ 2} \cdot 12\ in}{s_{heel.top}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$(A_s)_{heel.bot} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{heel.bot}^{\ 2} \cdot 12\ in}{s_{heel.bot}} = 0.785\ in^2$ *(area of steel for top reinforcement in heel)*

$\rho := \dfrac{(A_s)_{heel.top}}{b \cdot TTOE} = 0.004$ *(reinforcement ratio for primary tension steel)*

$\rho' := \dfrac{(A_s)_{heel.bot}}{b \cdot TTOE} = 0.004$ *(reinforcement ratio for bottom tension steel)*

$d_{heel.top} := TTOE - c_{heel.top} - 0.5 \cdot (d_b)_{heel.top} = 14.5\ in$ *(depth to top reinforcement in heel)*

$a \cdot 0.85 \cdot f'c \cdot (d_b)_{heel.top} = (A_s)_{heel.top} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\dfrac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{heel.top} := \dfrac{(A_s)_{heel.top} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}} = 0.86\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

Axial Strength Check

$Pn_{max} := \left((A_s)_{heel.top} \cdot (F_y)_{60} + (A_s)_{heel.bot} \cdot (F_y)_{60}\right) = 94.2\ kip$ *(design axial strength, EM 1110-2-2104, Eq B-44)*

$\phi Pn := \phi_A \cdot Pn_{max} = 84.8\ kip$

$AxialCheck := Check\left((\Sigma F_{heel})_{horz} \cdot b, \phi Pn\right) = [\text{“OK”} \quad 0.01]$ *(tensile)*

Flexural Check

$\phi M_{n.Heel} := \phi_b \cdot \left(0.85 \cdot f'_c \cdot a_{heel.top} \cdot b \cdot \left(d_{heel.top} - \dfrac{a_{heel.top}}{2}\right) - (P_u)_{heel} \cdot \left(d_{heel.top} - \dfrac{TTOE}{2}\right) \cdot b\right) = 41.4\ kip \cdot ft$

(moment capacity for beam in flexure and axial tension, EM 2104, Eq D-1)

$(M_{Flex\_Axial})_{Heel} := Check\left(Mu_{heel}, \phi M_{n.Heel}\right) = [\text{“OK”} \quad -0.045]$

Minimum Required Steel for Flexure

Given

Constraints

Values

$As_{req} := 0.5\ in^2$

$Mu_{heel} = \phi_b \cdot \left(\left(As_{req} \cdot (F_y)_{60} \cdot \left(d_{heel.top} - \dfrac{As_{req} \cdot (F_y)_{60} + (P_u)_{heel} \cdot b}{0.85 \cdot f'_c \cdot d_{heel.top}}\right)\right) - \left((P_u)_{heel} \cdot \left(d_{heel.top} - \dfrac{TTOE}{2}\right) \cdot b\right)\right)$

Solver

$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = -0.025\ in^2$

(minimum steel required for flexural strength)

$(A_{s.min})_{heel.top} := As_{req.stem} = 0\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{heel.top} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (TTOE \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{heel.top} \cdot \frac{4}{3} = 0\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{heel.req} := \text{if } A_{s\_1.3} < (A_s)_{heel.top} \parallel as \leftarrow (A_{s.min})_{heel.top}$ $\text{else} \parallel as \leftarrow A_{s.min} = -0.025\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{heel.top} = 0.8\ in^2$ *(area of steel provided in the horizontal direction for the heel)*

$(Steel\_Check_{Top})_{heel} := Check\left((A_s)_{heel.req}, (A_s)_{heel.top}\right) = [\text{"OK"} \quad -0.03]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.top}}{\left(b \cdot d_{heel.top}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{top})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Note: Current version of EM 1110-2-2104 provides this limit as a guidance. This is ok

Lateral Bars

$(d_b)_{heel.lat} :=$ Bar Size: #6 $= 0.75\ in$ *(lateral bar size in heel)*

$s_{heel.lat} := 12\ in$ *(lateral bar spacing in heel)*

$c_{heel.lat} :=$ 4″ cover *(cover for lateral reinforcement in heel)*

$(A_s)_{heel.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{heel.lat}^2 \cdot 12\ in}{s_{heel.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in heel)*

$d_{heel.lat} := TTS - c_{heel.lat} - (d_b)_{heel.top} - 0.5 \cdot (d_b)_{heel.lat} = 12.6\ in$ *(depth to reinforcement lateral reinforcement in heel)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TTOE \cdot b)}{2} = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.3\ in^2$

$(A_s)_{heel.lat} := (A_s)_{heel.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the heel, both faces)*

$(Steel\_Check_{Lat})_{heel} := Check\left(A_{s.req}, (A_s)_{heel.lat}\right) = [\text{"OK"} \quad 0.4]$ *(check heel reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{heel.lat}}{\left(b \cdot d_{heel.top}\right)} = 0.0051$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{heel} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.7]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

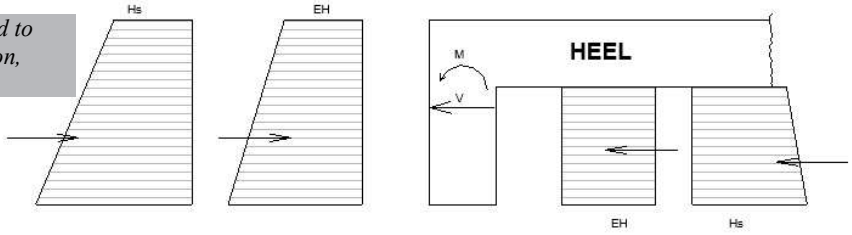
Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN

Loads to Key

Note: Drained and undrained loading cases are checked to determine the controlling case. In each loading condition, drained loads are listed first and undrained last



$HK = 1\ ft$ *(depth of key, from bottom of foundation down)*

$THEEL = 2.5\ ft$ *(depth of key, from top of foundation down)*

Hydrostatic Loads - Front of Key

Note: hydrostatic loads applied in two directions, notated as fk = front of key and bk = back of key

$(\sigma_{Hs})_{fk1} := \gamma_{water} \cdot (HTS + TTOE) = 0.343\ ksf$ *(horizontal water loading, top of key)*

$(\sigma_{Hs})_{fk2} := \gamma_{water} \cdot (HTS + THEEL) = 0.406\ ksf$ *(horizontal water loading, bottom of key)*

$(Hs)_{fk} := \frac{HK \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})}{2} = 0.4\ klf$ *(hydrostatic loading to the front of key)*

$(x_{Hs})_{fk} := \frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{fk2}) + (\sigma_{Hs})_{fk1})}{3 \cdot ((\sigma_{Hs})_{fk2} + (\sigma_{Hs})_{fk1})} = 0.51\ ft$ *(moment arm for hydrostatic loading, measured down from key connection to foundation)*

Hydrostatic Loads - Back of key

$(\sigma_{Hs})_{bk.top} := (\sigma_{Hs})_{bh.top} = \begin{bmatrix} 0.357 \\ 0.388 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of foundation)*

$(\sigma_{Hs})_{bk.bot} := (\sigma_{Hs})_{bh.bot} = \begin{bmatrix} 0.366 \\ 0.399 \end{bmatrix} ksf$ *(hydrostatic loading to back of key, bot of key)*

$HK = 1\ ft$ *(depth of key from the back side face)*

$(Hs)_{bk} := \frac{HK \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{2} = \begin{bmatrix} 0.4 \\ 0.4 \end{bmatrix} klf$ *(resultant of the hydrostatic loading to the front face of the key)*

$(x_{Hs})_{bk} := \left(\frac{HK \cdot ((2 \cdot (\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})}{3 \cdot ((\sigma_{Hs})_{bk.bot} + (\sigma_{Hs})_{bk.top})} \right) = \begin{bmatrix} 0.5 \\ 0.5 \end{bmatrix} ft$ *(moment arm of the hydrostatic loading to the front face of the key, measured down from the connection to foundation)*

Lateral Earth Pressures - Face of Key

$(\sigma_{EH})_{fk.top} := \gamma'_{soil} \cdot k_0 \cdot (HTS + TTOE) = 0.168\ ksf$ *(min horizontal lateral earth loading, top of foundation)*

$(\sigma_{EH})_{fk.bot} := \gamma'_{soil} \cdot k_0 \cdot (HTS + THEEL) = 0.199\ ksf$ *(min horizontal lateral earth loading, bottom of foundation)*

$THEEL = 2.5\ ft$ *(depth of applied loading)*

$(EH)_{fk} := \frac{HK \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{2} = 0.2\ klf$ *(resultant of the lateral earth loading to the front face of key)*

$(x_{EH})_{fk} := \frac{HK \cdot (2 \cdot (\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})}{3 \cdot ((\sigma_{EH})_{fk.bot} + (\sigma_{EH})_{fk.top})} = 0.514\ ft$ *(moment arm of the lateral earth loading to to the front face of the key, measured from the the connection of the key to the foundation)*

Lateral Earth Pressures - Back of Key

$(\sigma_{EH})_{bk.top} := (\sigma_{EH})_{bh.top} = \begin{bmatrix} 1.02 \\ 0.754 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of foundation)*

$(\sigma_{EH})_{bk.bot} := (\sigma_{EH})_{bh.bot} = \begin{bmatrix} 1.02 \\ 0.754 \end{bmatrix} ksf$ *(lateral earth loading to back of key, bot of key)*

$HK = 1\ ft$ *(depth of key from the back side face)*

$(EH)_{bk} := \frac{HK \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{2} = \begin{bmatrix} 1 \\ 0.8 \end{bmatrix} klf$ *(resultant of the lateral earth loading to the back face of key)*

$(x_{EH})_{bk} := \frac{HK \cdot \left(2 \cdot (\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)}{3 \cdot \left((\sigma_{EH})_{bk.bot} + (\sigma_{EH})_{bk.top}\right)} = \begin{bmatrix} 0.5 \\ 0.5 \end{bmatrix} ft$ *(moment arm for the lateral earth loading to the back face of the key, measured from the connection of the key to the foundation)*

Resultant Moment - on Toe

$LC1_{fk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{fk} \cdot (x_{EH})_{fk}}\right) + \left(\overrightarrow{(Hs)_{fk} \cdot (x_{Hs})_{fk}}\right)\right) = 0.5\ \frac{kip \cdot ft}{ft}$ *(load case for positive bending moments applied to heel)*

$LC1_{bk} := (\gamma_p)_{Serve.n} \cdot \left(\left(\overrightarrow{(EH)_{bk} \cdot (x_{EH})_{bk}}\right) + \left(\overrightarrow{(Hs)_{bk} \cdot (x_{Hs})_{bk}}\right)\right) = \begin{bmatrix} 1.1 \\ 0.9 \end{bmatrix} \frac{kip \cdot ft}{ft}$ *(load case for negative bending moments applied to heel)*

Resultant Moment at Base

$LC1_{key} := LC1_{bk} - LC1_{fk} = \begin{bmatrix} 0.6 \\ 0.5 \end{bmatrix} \frac{kip \cdot ft}{ft}$

$Mu_{key} := \max\left(LC1_{bk} - LC1_{fk}\right) \cdot 1\ ft = 0.6\ kip \cdot ft$ *(factored maximum moment at connection of floodwall key to the heel)*

Key Reinforcement

Back Bars - Main Steel

$(d_b)_{key.main} :=$ Bar Size: #8 $= 1\ in$ *(main bar size in key)*

$s_{key.main} := 12\ in$ *(main bar spacing in key)*

$c_{key.main} :=$ 4″ cover *(cover for main reinforcement in key)*

$(A_s)_{key.main} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.main}^{\ 2} \cdot 12\ in}{s_{key.main}} = 0.785\ in^2$ *(area of steel for top reinforcement in key)*

$d_{key.main} := TTOE - c_{key.main} - 0.5 \cdot (d_b)_{key.main} = 13.5\ in$ *(depth to main reinforcement in key)*

$a \cdot 0.85 \cdot f'c \cdot d = (A_s)_{key.main} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$a_{key.main} := \frac{(A_s)_{key.main} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}} = 0.9\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$(\phi M_n)_{key.main} := \phi_b \cdot (A_s)_{key.main} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{a_{key.main}}{2}\right) = 46.1\ kip \cdot ft$ *(nominal flexural strength, ACI 318-19)*

$(Flex_{nom})_{key} := Check\left(Mu_{key}, (\phi M_n)_{key.main}\right) = [\text{“OK”} \quad 0.01]$ *(nominal flexural check, utilization ratio)*

Minimum Required Steel for Flexure

Solve Constraints Values

$$As_{req} := 1.5\ in^2$$

$$Mu_{key} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{key.main} - \frac{As_{req} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot d_{key.main}}\right)$$

$$As_{req.stem} := \mathbf{find}\left(As_{req}\right) = 0.011\ in^2$$

$$(A_{s.min})_{key.min} := As_{req.stem} = 0\ in^2$$

Minimum Required Steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.5\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{key.main} = 0.5\ in^2$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Minimum Requirements for Reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{TK \cdot b}{2}\right) = 0.216\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b\right)$$ *(minimum area of steel required for flexural capacity)*

$$A_{s.min} = 0.543\ in^2$$

$$A_{s\_1.3} := (A_{s.min})_{key.min} \cdot \frac{4}{3} = 0\ in^2$$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$$(A_s)_{key.req} := \begin{array}{l} \text{if } A_{s\_1.3} < (A_s)_{key.main} \\ \quad as \leftarrow (A_{s.min})_{key.min} \\ \text{else} \\ \quad as \leftarrow A_{s.min} \end{array} = 0.011\ in^2$$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$$(A_s)_{key.main} = 0.8\ in^2$$ *(area of steel provided in the horizontal direction for the heel)*

$$(Steel\_Check_{Top})_{key} := Check\left((A_s)_{key.req}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.01]$$ *(check stem reinforcement to required reinforcement)*

Temperature and Shrinkage Requirements

$$(A_{s.min})_{temp} = 0.2\ in^2$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$$(Steel\_Check_{Top.ts})_{key} := Check\left((A_{s.min})_{temp}, (A_s)_{key.main}\right) = [\text{“OK”} \quad 0.3]$$

(temperature and shrinkage steel of key, check utilization ratio)

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$$ *(reinforcement ratio from EM 1110-2-2104)*

$$\rho_{actual} := \frac{(A_s)_{key.main}}{\left(b \cdot d_{key.main}\right)} = 0.0048$$ *(actual reinforcement ratio for concrete section)*

$$(\rho\_check_{main})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.6]$$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Lateral Bars

$(d_b)_{key.lat} := \text{Bar Size: \#6} = 0.75\ in$ *(lateral bar size in key)*

$s_{key.lat} := 12\ in$ *(lateral bar spacing in key)*

$c_{key.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in key)*

$(A_s)_{key.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{key.lat}^{\ 2} \cdot 12\ in}{s_{key.lat}} = 0.442\ in^2$ *(area of steel lateral reinforcement in key)*

$d_{key.lat} := TK - c_{key.lat} - (d_b)_{key.main} - 0.5 \cdot (d_b)_{key.lat} = 6.6\ in$ *(depth to reinforcement lateral reinforcement in key)*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := \frac{0.003 \cdot (TK \cdot b)}{2} = 0.216\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.req} := (A_{s.min})_{temp} = 0.2\ in^2$

$(A_s)_{key.lat} := (A_s)_{key.lat} \cdot 2 = 0.9\ in^2$ *(area of steel provided in the lateral direction for the key, both faces)*

$(Steel\_Check_{Lat})_{key} := Check\left(A_{s.req}, (A_s)_{key.lat}\right) = [\text{"OK"} \quad 0.2]$

(check key reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{actual} := \frac{(A_s)_{key.lat}}{(b \cdot d_{key.main})} = 0.0055$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{key} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.7]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

4 FT T-WALL SUMMARY

WALL GEOMETRY

$ELTS = 938.5\ ft$

$HTS = 4\ ft$

$TTS = 1.5\ ft$

$TBS = 1.5\ ft$

$THEEL = 2.5\ ft$

$TTOE = 1.5\ ft$

$TWIDTH = 3\ ft$

$BWIDTH = 8\ ft$

$HK = 1\ ft$

$TK = 1\ ft$

STEM DESIGN STEEL

Driving Side Reinforcement

$(d_b)_{stem.vert} = 1\ in$ *bars located at* $s_{stem.vert} = 12\ in$ *centers*

Driving Side Lateral Reinforcement

$(d_b)_{stem.lat} = 0.75\ in$ *bars located at* $s_{stem.lat} = 12\ in$ *centers*

Resisting Side Vertical Reinforcement

Same as Driving Side Reinforcement

Resisting Side Lateral Reinforcement

Same as Driving Side Reinforcement

TOE DESIGN STEEL

Toe - Bottom Reinforcement

$(d_b)_{toe.bot} = 1\ in$ *bars located at* $s_{toe.bot} = 12\ in$ *centers*

Toe - Lateral Reinforcement

$(d_b)_{toe.lat} = 0.75\ in$ *bars located at* $s_{toe.lat} = 12\ in$ *centers*

Toe - Top Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

Toe - Top Lateral Reinforcement

Reference below calculations for the top steel for the heel design, top steel from heel carried through toe of base

HEEL DESIGN STEEL

Heel - Top Reinforcement

$(d_b)_{heel.top} = 1\ in$ *bars located at* $s_{heel.top} = 12\ in$ *centers*

Heel - Lateral Reinforcement

$(d_b)_{heel.lat} = 0.75\ in$ *bars located at* $s_{heel.lat} = 12\ in$ *centers*

Heel - Bottom Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

Heel - Bottom Lateral Reinforcement

Reference above calculations for the bottom steel from the toe design, toe steel is transferred through the base into the heel.

KEY DESIGN STEEL

Key - Front Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key- Lateral Reinforcement

$(d_b)_{key.main} = 1\ in$ *bars located at* $s_{key.main} = 12\ in$ *centers*

Key - Back Reinforcement

Key reinforcement in back of key same as front of key

Key - Back Lateral Reinforcement

Key reinforcement in back of key same as front of key

RS32+61 RB24+40 ALT 14ft Wall - Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 7 Time: 15.18.50

Structural geometry data:
Elevation of top of stem (ELTS) = 921.76 ft
Height of stem (HTS) = 14.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 4.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 15.50 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 2.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	902.26
0.00	907.76
9.50	907.76
9.50	921.76
11.00	921.76
11.00	907.76
15.50	907.76
15.50	905.76
2.00	905.76
2.00	902.26

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	912.73

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	0.00	4.00
2	-8.25	16.00
3	-0.89	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1010.50	864.65
2	-510.50	864.65
3	-10.50	901.73
4	5.50	912.73
5	9.50	912.73

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	912.73	0.97

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	11.00	912.73
2	511.00	953.15

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)

phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 921.27 ft
Resisting side elevation = 912.73 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
921.20	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 82.55 %
Overturning ratio = 1.43

Xr (measured from toe) = 4.27 ft
Resultant ratio = 0.2752
Stem ratio = 0.2903
Base pressure at x= 12.80 ft from toe = 0.0000 ksf

Base pressure at toe = 1.7141 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 3.96

****
* Bearing *
****

Net ultimate bearing pressure = 5.5627 (ksf)
Factor of safety = 4.129

********** Output Results **********

Date: 2025/11/ 7 Time: 15.18.50

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = -84.7591
Calculated earth pressure coefficients:
Driving side at rest K = 0.1989
Driving side at rest Kc = 1.1300
Resisting side at rest K = 0.5733
Resisting side at rest Kc = 0.7571
At-rest K's for resisting side calculated.

Depth of cracking = 0.00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
921.27	0.0000
908.95	0.7688
902.26	1.0656

Earth pressures:

Elevation (ft)	Pressure (ksf)
908.95	0.0000
902.26	0.0978

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
912.73	0.0000
905.76	0.5606
905.76	1.0354
902.26	1.0656

Earth pressures:

Elevation (ft)	Pressure (ksf)
913.09	0.0000
905.76	0.2031

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
905.76	1.4067
902.26	1.4067

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	1.0656
2.00	1.0354
2.00	0.7642
15.50	0.5606

** Forces and moments **

==========

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)
Structure:				
Structure weight.....	8.850		-7.66	-67.80
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	4.602		-9.93	-45.68
Water above structure.....	0.000		0.00	0.00
Water above soil.....	5.711		-11.08	-63.29
External vertical loads....	0.000		0.00	0.00
Ext. horz. pressure loads..		0.000	0.00	0.00
Ext. horz. line loads.....		0.100	15.44	1.54
Structure, resisting side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	2.898		-2.22	-6.44
Water above structure.....	0.000		0.00	0.00
Water above soil.....	-0.051		-1.50	0.08
Driving side:				
Effective earth loads.....		0.327	-1.28	-0.42
Shear (due to delta).....	0.000		0.00	0.00
Horiz. surcharge effects...		0.000	0.00	0.00
Water loads.....		10.872	2.99	32.47
Resisting side:				
Effective earth loads.....		-0.745	2.44	-1.82
Balancing earth load.....		-4.923	-1.76	8.66
Water loads.....		-5.631	-0.35	1.99
Foundation:				
Vertical force on base.....	-10.966		-4.27	46.77
Uplift.....	-11.044		-8.51	93.93

** Statics Check ** SUMS = 0.000 0.000 0.00

Angle of base = 12.72 degrees
Normal force on base = 11.782 kips
Shear force on base = 2.387 kips
Max. available shear force = 7.569 kips

Base pressure at x= 12.80 ft from toe = 0.0000 ksf
Base pressure at toe = 1.7141 ksf

Xr (measured from toe) = 4.27 ft
Resultant ratio = 0.2752
Stem ratio = 0.2903
Base in compression = 82.55 %
Overturning ratio = 1.43

Volume of concrete = 2.19 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.528
2	4.837	5.660
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.7978	1.0656		
2		0.0000		1.0656
2		15.5000		0.5606
3	0.0000	0.5606		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 902.26 ft
Point 2 (right), x = 15.50 ft, y = 905.76 ft

Depth of cracking = 0.00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)

```
 ========================================================
   1  -83.824    6.262    0.282   6.262    5.834
   2   12.724   15.890   18.865  15.890   12.921
   3   53.852    9.652    2.594   8.632    2.420

 Wedge    Net force
 number    (kips)
 ===================
   1      -6.538
   2       2.437
   3       4.103
 ===================
   SUM =    0.001

+----------------------------+
| Factor of safety =   3.960 |
+----------------------------+




**********************
**  Bearing Results  **
**********************

        Base width =     15.890 (ft)
             Xr =      4.265 (ft)
 Effective base width =      8.745 (ft)
  (measured along slope)
        Base slope =    12.7244 (deg)

             phi =     28.000 (deg)
               c =      0.000 (ksf)
     Effective gamma =     0.0626 (kcf)

        Normal load =     11.782 (kips)
    Load inclination =     11.453 (deg)
   Load eccentricity =      3.573 (ft)

         Surcharge =     0.4174 (ksf)
         Embedment =      6.689 (ft)
       Ground slope =    -0.0807 (deg)


       Bearing Capacity Factors
==========================================
             C       Q       G
```

==========

Bearing	25.8033	14.7199	11.1897
Embedment	1.2546	1.1273	1.1273
Inclination	0.7617	0.7617	0.3492
Base Tilt	0.7616	0.7778	0.7778
Ground Slope	1.1805	1.1682	1.1682

Net ultimate bearing pressure = 5.5627 (ksf)

Factor of safety = 4.129

RS32+61 RB24+40 ALT 14ft Wall - Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 7 Time: 14.48.39

Structural geometry data:
Elevation of top of stem (ELTS) = 921.76 ft
Height of stem (HTS) = 14.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 4.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 15.50 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 2.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	902.26
0.00	907.76
9.50	907.76
9.50	921.76
11.00	921.76
11.00	907.76
15.50	907.76
15.50	905.76
2.00	905.76
2.00	902.26

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	912.73

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	0.00	4.00
2	-8.25	16.00
3	-0.89	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1010.50	864.65
2	-510.50	864.65
3	-10.50	901.73
4	5.50	912.73
5	9.50	912.73

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	912.73	0.97

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	11.00	912.73
2	511.00	953.15

Foundation property data:
phi for soil-structure interface = 0.00 (deg)
c for soil-structure interface = 1.000 (ksf)

phi for soil-soil interface = 0.00 (deg)
c for soil-soil interface = 1.000 (ksf)

Water data:
Driving side elevation = 921.27 ft
Resisting side elevation = 912.73 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
921.20	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 77.14 %
Overturning ratio = 1.33

Xr (measured from toe) = 3.99 ft
Resultant ratio = 0.2571
Stem ratio = 0.2903
Base pressure at x= 11.96 ft from toe = 0.0000 ksf

Base pressure at toe = 1.6451 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 14.27

****
* Bearing *
****

Net ultimate bearing pressure = 4.5467 (ksf)
Factor of safety = 3.551

********** Output Results **********

Date: 2025/11/ 7 Time: 14.48.39

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = 0.0000
Calculated earth pressure coefficients:
Driving side at rest K = 0.0000
Driving side at rest Kc = 0.0000
Resisting side at rest K = 1.0806
Resisting side at rest Kc = 1.0395
At-rest K's for resisting side calculated.

Depth of cracking = 6.69 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
921.27	0.0000
902.26	1.1862

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
912.73	0.0000
905.76	0.5974
905.76	1.1472
902.26	1.1862

Earth pressures:

Elevation (ft)	Pressure (ksf)
913.09	0.0000
905.76	0.3431

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
905.76	1.1290
902.26	1.1290

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	1.1862
2.00	1.1472
2.00	0.8606
15.50	0.5974

** Forces and moments **

Part	Force (kips)		Mom. Arm (ft)	Moment (ft-k)
	Vert.	Horiz.		
Structure:				
Structure weight.....	8.850		-7.66	-67.80
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00

```
 Saturated soil.....   4.602          -9.93   -45.68
 Water above structure.....  0.000          0.00    0.00
 Water above soil.....   5.711          -11.08   -63.29
 External vertical loads....  0.000          0.00    0.00
 Ext. horz. pressure loads..      0.000    0.00    0.00
 Ext. horz. line loads.....      0.100    15.44    1.54
Structure, resisting side:
 Moist soil.....  0.000          0.00    0.00
 Saturated soil.....   2.898          -2.22   -6.44
 Water above structure.....  0.000          0.00    0.00
 Water above soil.....   -0.051          -1.50    0.08
Driving side:
 Effective earth loads.....      0.000    0.00    0.00
 Shear (due to delta).....  0.000          0.00    0.00
 Horiz. surcharge effects...      0.000    0.00    0.00
 Water loads.....      11.275    2.83   31.92
Resisting side:
 Effective earth loads.....      -1.258    2.46   -3.09
 Balancing earth load.....      -3.952    -1.74    6.89
 Water loads.....      -6.166    -0.39    2.39
Foundation:
 Vertical force on base.....  -9.835          -3.99   39.20
 Uplift.....  -12.175          -8.56   104.28
=========================================================================
** Statics Check **  SUMS =    0.000    0.000          0.00

Angle of base     =   12.72 degrees
Normal force on base =   10.464 kips
Shear force on base  =   1.688 kips
Max. available shear force =   15.890 kips

Base pressure at x=  11.96 ft from toe =   0.0000 ksf
Base pressure at toe          =   1.6451 ksf

Xr (measured from toe) =    3.99 ft
Resultant ratio     =   0.2571
Stem ratio        =   0.2903
Base in compression   =   77.14 %
Overturning ratio    =    1.33

Volume of concrete =    2.19 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
```

do not result in excessive differential settlement of
the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.000
2	11.375	5.660
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.0000	0.0000		
2		0.0000		1.1862
2		15.5000		0.5974
3	0.0000	0.5974		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 902.26 ft
Point 2 (right), x = 15.50 ft, y = 905.76 ft

Depth of cracking = 6.69 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	0.000	0.000	0.000	0.000	0.000
2	12.724	15.890	18.865	15.890	14.171
3	52.375	9.875	2.746	8.800	2.629

Wedge	Net force

```
 number     (kips)
 ===================
   1       0.000
   2      -4.696
   3       4.696
 ===================
   SUM =    0.000

+----------------------------+
| Factor of safety =   14.273 |
+----------------------------+




**********************
**  Bearing Results  **
**********************

        Base width =     15.890 (ft)
              Xr =      3.986 (ft)
 Effective base width =      8.172 (ft)
  (measured along slope)
        Base slope =    12.7244 (deg)

             phi =      0.000 (deg)
               c =      1.000 (ksf)
     Effective gamma =     0.0626 (kcf)

        Normal load =     10.464 (kips)
    Load inclination =      9.165 (deg)
   Load eccentricity =      3.859 (ft)

         Surcharge =     0.4174 (ksf)
         Embedment =      6.689 (ft)
       Ground slope =    -0.0807 (deg)


       Bearing Capacity Factors
===============================================
             C       Q       G
===============================================
Bearing         5.1400    1.0000    0.1611
Embedment       1.1637    1.0000    1.0000
Inclination     0.8067    0.8067    0.0000
Base Tilt       0.9136    1.0000    1.0000
Ground Slope    1.0314    1.1682    1.1682
```

Net ultimate bearing pressure = 4.5467 (ksf)

+--------------------------------+
| Factor of safety = 3.551 |
+--------------------------------+

+--------------------------------+
| Factor of safety = 4.331 |
+--------------------------------+

RS32+61 RB24+40 ALT 12ft Wall-D
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 9 Time: 8.07.34

Structural geometry data:
Elevation of top of stem (ELTS) = 921.50 ft
Height of stem (HTS) = 12.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 4.00 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 4.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 12.50 ft
Depth of key (HK) = 2.00 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	905.50
0.00	909.50
7.00	909.50
7.00	921.50
8.50	921.50
8.50	909.50
12.50	909.50
12.50	907.50
3.00	907.50
3.00	905.50

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	0.00	11.00
2	-10.29	17.50
3	-0.89	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1021.50	859.91
2	-521.50	859.91
3	-21.50	896.99
4	-4.00	912.00
5	7.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	915.00	0.65

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	8.50	915.00
2	508.50	942.08

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)

phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 921.27 ft
Resisting side elevation = 915.00 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
921.20	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 77.78 %
Overturning ratio = 1.33

Xr (measured from toe) = 3.24 ft
Resultant ratio = 0.2593
Stem ratio = 0.3200
Base pressure at x= 9.72 ft from toe = 0.0000 ksf

Base pressure at toe = 1.5811 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 3.26

****
* Bearing *
****

Net ultimate bearing pressure = 4.5228 (ksf)
Factor of safety = 3.638

********** Output Results **********

Date: 2025/11/ 9 Time: 8.07.34

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = -58.3933
Calculated earth pressure coefficients:
Driving side at rest K = 0.4961
Driving side at rest Kc = 0.7108
Resisting side at rest K = 0.5592
Resisting side at rest Kc = 0.7478
At-rest K's for resisting side calculated.

Depth of cracking = 0.00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
921.27	0.0000
912.00	0.5784
905.50	0.8887

Earth pressures:

Elevation (ft)	Pressure (ksf)
912.00	0.0000
905.50	0.2492

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
915.00	0.0000
907.50	0.5781
907.50	0.8502
905.50	0.8887

Earth pressures:

Elevation (ft)	Pressure (ksf)
915.22	0.0000
907.50	0.2155

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
907.50	1.8104
905.50	1.8104

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.8887
3.00	0.8502
3.00	0.6998
12.50	0.5781

** Forces and moments **

==========

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)
Structure:				
Structure weight.....	7.350		-6.28	-46.16
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	2.188		-9.00	-19.69
Water above structure.....	0.000		0.00	0.00
Water above soil.....	4.049		-9.00	-36.44
External vertical loads....	0.000		0.00	0.00
Ext. horz. pressure loads..		0.000	0.00	0.00
Ext. horz. line loads.....		0.100	13.70	1.37
Structure, resisting side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	2.804		-1.99	-5.57
Water above structure.....	0.000		0.00	0.00
Water above soil.....	-0.027		-1.33	0.04
Driving side:				
Effective earth loads.....		0.810	0.16	0.13
Shear (due to delta).....	0.000		0.00	0.00
Horiz. surcharge effects...		0.000	0.00	0.00
Water loads.....		7.449	3.38	25.17
Resisting side:				
Effective earth loads.....		-0.832	2.58	-2.14
Balancing earth load.....		-3.621	-0.98	3.56
Water loads.....		-3.907	0.93	-3.63
Foundation:				
Vertical force on base.....	-7.686		-3.24	24.91
Uplift.....	-8.678		-6.74	58.47

** Statics Check ** SUMS = 0.000 0.000 0.00

Angle of base = 9.09 degrees
Normal force on base = 8.161 kips
Shear force on base = 2.361 kips
Max. available shear force = 4.766 kips

Base pressure at x= 9.72 ft from toe = 0.0000 ksf
Base pressure at toe = 1.5811 ksf

Xr (measured from toe) = 3.24 ft
Resultant ratio = 0.2593
Stem ratio = 0.3200
Base in compression = 77.78 %
Overturning ratio = 1.33

Volume of concrete = 1.81 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	2.314
2	2.781	4.022
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.5784	0.8887		
2			0.0000	0.8887
2			12.5000	0.5781
3	0.0000	0.5781		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 905.50 ft
Point 2 (right), x = 12.50 ft, y = 907.50 ft

Depth of cracking = 0.00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)

```
 ========================================================
   1  -58.393    7.632     1.625   7.632    5.599
   2    9.090   12.659    13.154  12.659    9.284
   3   48.763   10.773     3.411   9.974    2.883

 Wedge    Net force
 number    (kips)
 ===================
   1      -5.929
   2       1.342
   3       4.588
 ===================
   SUM =    0.000

+----------------------------+
| Factor of safety =   3.263 |
+----------------------------+




**********************
**  Bearing Results  **
**********************

        Base width =     12.659 (ft)
             Xr =      3.241 (ft)
 Effective base width =      6.564 (ft)
  (measured along slope)
        Base slope =     9.0903 (deg)

             phi =     28.000 (deg)
               c =      0.000 (ksf)
     Effective gamma =     0.0626 (kcf)

        Normal load =      8.161 (kips)
    Load inclination =     16.135 (deg)
   Load eccentricity =      3.048 (ft)

         Surcharge =     0.4063 (ksf)
         Embedment =      6.500 (ft)
       Ground slope =    -0.0541 (deg)


       Bearing Capacity Factors
==========================================
             C       Q       G
```

==========

Bearing	25.8033	14.7199	11.1897
Embedment	1.3296	1.1648	1.1648
Inclination	0.6736	0.6736	0.1795
Base Tilt	0.8266	0.8384	0.8384
Ground Slope	1.1194	1.1113	1.1113

Net ultimate bearing pressure = 4.5228 (ksf)

Factor of safety = 3.638

Factor of safety = 2.210

RS32+61 RB24+40 ALT 12ft Wall-Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 9 Time: 8.10.12

Structural geometry data:
Elevation of top of stem (ELTS) = 921.50 ft
Height of stem (HTS) = 12.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 4.00 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 4.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 12.50 ft
Depth of key (HK) = 2.00 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	905.50
0.00	909.50
7.00	909.50
7.00	921.50
8.50	921.50
8.50	909.50
12.50	909.50
12.50	907.50
3.00	907.50
3.00	905.50

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	0.00	11.00
2	-10.29	17.50
3	-0.89	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1021.50	859.91
2	-521.50	859.91
3	-21.50	896.99
4	-4.00	912.00
5	7.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	915.00	0.65

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	8.50	915.00
2	508.50	942.08

Foundation property data:
phi for soil-structure interface = 0.00 (deg)
c for soil-structure interface = 1.000 (ksf)

phi for soil-soil interface = 0.00 (deg)
c for soil-soil interface = 1.000 (ksf)

Water data:
Driving side elevation = 921.27 ft
Resisting side elevation = 915.00 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
921.20	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 77.62 %
Overturning ratio = 1.27

Xr (measured from toe) = 3.23 ft
Resultant ratio = 0.2587
Stem ratio = 0.3200
Base pressure at x= 9.70 ft from toe = 0.0000 ksf

Base pressure at toe = 1.4212 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 23.73

****
* Bearing *
****

Net ultimate bearing pressure = 4.8107 (ksf)
Factor of safety = 4.402

********** Output Results **********

Date: 2025/11/ 9 Time: 8.10.12

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = 0.0000
Calculated earth pressure coefficients:
Driving side at rest K = 0.0000
Driving side at rest Kc = 0.0000
Resisting side at rest K = 1.0541
Resisting side at rest Kc = 1.0267
At-rest K's for resisting side calculated.

Depth of cracking = 6.50 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
921.27	0.0000
905.50	0.9840

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
915.00	0.0000
907.50	0.6136
907.50	0.9332
905.50	0.9840

Earth pressures:

Elevation (ft)	Pressure (ksf)
915.22	0.0000
907.50	0.3689

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
907.50	1.1090
905.50	1.1090

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.9840
3.00	0.9332
3.00	0.7745
12.50	0.6136

** Forces and moments **

Part	Force (kips)		Mom. Arm (ft)	Moment (ft-k)
	Vert.	Horiz.		
Structure:				
Structure weight.....	7.350		-6.28	-46.16
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00

```
  Saturated soil.............    2.188              -9.00    -19.69
  Water above structure......    0.000               0.00      0.00
  Water above soil...........    4.049              -9.00    -36.44
  External vertical loads....    0.000               0.00      0.00
  Ext. horz. pressure loads..            0.000      0.00      0.00
  Ext. horz. line loads......            0.100     13.70      1.37
Structure, resisting side:
  Moist soil.................    0.000               0.00      0.00
  Saturated soil.............    2.804              -1.99     -5.57
  Water above structure......    0.000               0.00      0.00
  Water above soil...........   -0.027              -1.33      0.04
Driving side:
  Effective earth loads......            0.000      0.00      0.00
  Shear (due to delta).......    0.000               0.00      0.00
  Horiz. surcharge effects...            0.000      0.00      0.00
  Water loads................            7.759      3.26     25.29
Resisting side:
  Effective earth loads......           -1.423      2.59     -3.69
  Balancing earth load.......           -2.218     -1.01      2.23
  Water loads................           -4.218      0.92     -3.87
Foundation:
  Vertical force on base.....   -6.894              -3.23     22.30
  Uplift.....................   -9.469              -6.78     64.20
=====================================================================
** Statics Check **   SUMS =     0.000      0.000              0.00

Angle of base        =     9.09 degrees
Normal force on base =    7.158 kips
Shear force on base  =    1.101 kips
Max. available shear force =   12.659 kips

Base pressure at x=   9.70 ft from toe =   0.0000 ksf
Base pressure at toe                =   1.4212 ksf

Xr (measured from toe) =     3.23 ft
Resultant ratio        =   0.2587
Stem ratio             =   0.3200
Base in compression    =    77.62 %
Overturning ratio      =     1.27

Volume of concrete =     1.81 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
```

do not result in excessive differential settlement of
the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.000
2	7.859	4.022
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.0000	0.0000		
2		0.0000	0.9840	
2		12.5000	0.6136	
3	0.0000	0.6136		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 905.50 ft
Point 2 (right), x = 12.50 ft, y = 907.50 ft

Depth of cracking = 6.50 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	0.000	0.000	0.000	0.000	0.000
2	9.090	12.659	13.154	12.659	10.112
3	52.367	10.169	2.983	9.470	2.905

Wedge	Net force

number	(kips)
1	0.000
2	-4.571
3	4.571
SUM =	0.000

+----------+
| Factor of safety = 23.728 |
+----------+

****
** Bearing Results **
****

Base width = 12.659 (ft)
Xr = 3.234 (ft)
Effective base width = 6.550 (ft)
(measured along slope)
Base slope = 9.0903 (deg)

phi = 0.000 (deg)
c = 1.000 (ksf)
Effective gamma = 0.0626 (kcf)

Normal load = 7.158 (kips)
Load inclination = 8.743 (deg)
Load eccentricity = 3.054 (ft)

Surcharge = 0.4063 (ksf)
Embedment = 6.500 (ft)
Ground slope = -0.0541 (deg)

Bearing Capacity Factors

	C	Q	G
Bearing	5.1400	1.0000	0.1082
Embedment	1.1985	1.0000	1.0000
Inclination	0.8152	0.8152	0.0000
Base Tilt	0.9383	1.0000	1.0000
Ground Slope	1.0210	1.1113	1.1113

Net ultimate bearing pressure = 4.8107 (ksf)

+--------------------------------+
| Factor of safety = 4.402 |
+--------------------------------+

+--------------------------------+
| Factor of safety = 6.564 |
+--------------------------------+

+--------------------------------+
| Factor of safety = 2.210 |
+--------------------------------+

RS36+75 RB28+40 Alternate 10ft Wall - Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 8 Time: 7.44.51

Structural geometry data:
Elevation of top of stem (ELTS) = 920.31 ft
Height of stem (HTS) = 10.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 2.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 13.00 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	904.81
0.00	910.31
9.00	910.31
9.00	920.31
10.50	920.31
10.50	910.31
13.00	910.31
13.00	908.31
3.00	908.31
3.00	904.81

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-0.50	10.00
2	-6.00	24.00
3	-1.26	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1025.00	847.08
2	-525.00	847.08
3	-25.00	899.58
4	-1.00	911.58
5	9.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	912.00	0.16

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	10.50	912.00
2	510.50	918.67

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)

phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 920.17 ft
Resisting side elevation = 911.82 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.17	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 91.39 %
Overturning ratio = 1.62

Xr (measured from toe) = 3.96 ft
Resultant ratio = 0.3046
Stem ratio = 0.1923
Base pressure at x= 11.88 ft from toe = 0.0000 ksf

Base pressure at toe = 1.4209 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 2.37

****
* Bearing *
****

Net ultimate bearing pressure = 2.9173 (ksf)
Factor of safety = 2.589

********** Output Results **********

Date: 2025/11/ 8 Time: 7.44.51

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = -77.2714
Calculated earth pressure coefficients:
Driving side at rest K = 0.3581
Driving side at rest Kc = 0.8971
Resisting side at rest K = 0.5376
Resisting side at rest Kc = 0.7332
At-rest K's for resisting side calculated.

Depth of cracking = 0.00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
920.17	0.0000
911.62	0.5332
904.81	0.8092

Earth pressures:

Elevation (ft)	Pressure (ksf)
911.62	0.0000
907.16	0.1334
904.81	0.1942

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
911.82	0.0000
908.31	0.2959
908.31	0.7556
904.81	0.8092

Earth pressures:

Elevation (ft)	Pressure (ksf)
912.03	0.0000
908.31	0.0906

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
908.31	1.2024
904.81	1.2024

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.8092
3.00	0.7556
3.00	0.4746
13.00	0.2959

** Forces and moments **

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)
Structure:				
Structure weight.....	7.725		-6.57	-50.77
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	1.690		-8.31	-14.05
Water above structure.....	0.000		0.00	0.00
Water above soil.....	4.694		-8.53	-40.05
External vertical loads....	0.000		0.00	0.00
Ext. horz. pressure loads..		0.000	0.00	0.00
Ext. horz. line loads.....		0.100	11.86	1.19
Structure, resisting side:				
Moist soil.....	0.059		-1.21	-0.07
Saturated soil.....	0.472		-1.25	-0.59
Water above structure.....	0.000		0.00	0.00
Water above soil.....	0.000		0.00	0.00
Driving side:				
Effective earth loads.....		0.682	-1.21	-0.82
Shear (due to delta).....	0.000		0.00	0.00
Horiz. surcharge effects...		0.000	0.00	0.00
Water loads.....		6.852	1.83	12.54
Resisting side:				
Effective earth loads.....		-0.169	1.25	-0.21
Balancing earth load.....		-4.209	-1.76	7.42
Water loads.....		-3.258	-1.29	4.22
Foundation:				
Vertical force on base.....	-8.440		-3.96	33.42
Uplift.....	-6.200		-7.71	47.78
** Statics Check ** SUMS =	0.000	0.000		0.00

Angle of base = 15.07 degrees
Normal force on base = 9.244 kips
Shear force on base = 1.870 kips
Max. available shear force = 5.701 kips

Base pressure at x= 11.88 ft from toe = 0.0000 ksf
Base pressure at toe = 1.4209 ksf

Xr (measured from toe) = 3.96 ft
Resultant ratio = 0.3046
Stem ratio = 0.1923
Base in compression = 91.39 %
Overturning ratio = 1.62

Volume of concrete = 1.91 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.828
2	2.378	4.694
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.5527	0.8092		
2			0.0000	0.8092
2			13.0000	0.2959
3	0.0000	0.2959		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 904.81 ft
Point 2 (right), x = 13.00 ft, y = 908.31 ft

Depth of cracking = 0.00 ft

Wedge	Failure angle	Total length	Weight of wedge	Submerged length	Uplift force

number	(deg)	(ft)	(kips)	(ft)	(kips)
1	-76.678	6.683	0.671	6.683	4.551
2	15.068	13.463	11.477	13.463	7.439
3	41.515	5.703	0.989	5.296	0.783

Wedge number	Net force (kips)
1	-5.354
2	4.278
3	1.076
SUM =	0.000

Factor of safety =	2.369

****
** Bearing Results **
****

Base width = 13.463 (ft)
Xr = 3.960 (ft)
Effective base width = 8.202 (ft)
(measured along slope)
Base slope = 15.0685 (deg)

phi = 28.000 (deg)
c = 0.000 (ksf)
Effective gamma = 0.0626 (kcf)

Normal load = 9.244 (kips)
Load inclination = 11.434 (deg)
Load eccentricity = 2.630 (ft)

Surcharge = 0.2453 (ksf)
Embedment = 3.723 (ft)
Ground slope = -0.0133 (deg)

Bearing Capacity Factors
==========

	C	Q	G
Bearing	25.8033	14.7199	11.1897
Embedment	1.1511	1.0755	1.0755
Inclination	0.7620	0.7620	0.3500
Base Tilt	0.7209	0.7399	0.7399
Ground Slope	1.0288	1.0268	1.0268

Net ultimate bearing pressure = 2.9173 (ksf)

Factor of safety = 2.589

RS36+75 RB28+40 Alternate 10ft Wall - Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 8 Time: 7.41.33

Structural geometry data:
Elevation of top of stem (ELTS) = 920.31 ft
Height of stem (HTS) = 10.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 2.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 13.00 ft
Depth of key (HK) = 3.50 ft
Width of bottom of key (TK) = 3.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	904.81
0.00	910.31
9.00	910.31
9.00	920.31
10.50	920.31
10.50	910.31
13.00	910.31
13.00	908.31
3.00	908.31
3.00	904.81

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	912.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-0.50	10.00
2	-6.00	24.00
3	-1.26	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1025.00	847.08
2	-525.00	847.08
3	-25.00	899.58
4	-1.00	911.58
5	9.00	912.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	912.00	0.16

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	10.50	912.00
2	510.50	918.67

Foundation property data:
phi for soil-structure interface = 0.00 (deg)
c for soil-structure interface = 1.000 (ksf)

phi for soil-soil interface = 0.00 (deg)
c for soil-soil interface = 1.000 (ksf)

Water data:
Driving side elevation = 920.17 ft
Resisting side elevation = 911.82 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.17	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 79.10 %
Overturning ratio = 1.42

Xr (measured from toe) = 3.43 ft
Resultant ratio = 0.2637
Stem ratio = 0.1923
Base pressure at x= 10.28 ft from toe = 0.0000 ksf

Base pressure at toe = 1.4309 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 9.62

****
* Bearing *
****

Net ultimate bearing pressure = 4.0727 (ksf)
Factor of safety = 3.619

********** Output Results **********

Date: 2025/11/ 8 Time: 7.41.33

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = 0.0000
Calculated earth pressure coefficients:
Driving side at rest K = 0.0000
Driving side at rest Kc = 0.0000
Resisting side at rest K = 1.0133
Resisting side at rest Kc = 1.0066
At-rest K's for resisting side calculated.

Depth of cracking = 6.82 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
920.17	0.0000
904.81	0.9585

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
911.82	0.0000
908.31	0.3268
908.31	0.8833
904.81	0.9585

Earth pressures:

Elevation (ft)	Pressure (ksf)
912.03	0.0000
908.31	0.1394

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
908.31	0.9728
904.81	0.9728

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.9585
3.00	0.8833
3.00	0.5773
13.00	0.3268

** Forces and moments **

Part	Force (kips)		Mom. Arm (ft)	Moment (ft-k)
	Vert.	Horiz.		
Structure:				
Structure weight.....	7.725		-6.57	-50.77
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00

Saturated soil..... 1.690 -8.31 -14.05
Water above structure..... 0.000 0.00 0.00
Water above soil..... 4.694 -8.53 -40.05
External vertical loads.... 0.000 0.00 0.00
Ext. horz. pressure loads.. 0.000 0.00 0.00
Ext. horz. line loads..... 0.100 11.86 1.19
Structure, resisting side:
Moist soil..... 0.059 -1.21 -0.07
Saturated soil..... 0.472 -1.25 -0.59
Water above structure..... 0.000 0.00 0.00
Water above soil..... 0.000 0.00 0.00
Driving side:
Effective earth loads..... 0.000 0.00 0.00
Shear (due to delta)..... 0.000 0.00 0.00
Horiz. surcharge effects... 0.000 0.00 0.00
Water loads..... 7.361 1.62 11.93
Resisting side:
Effective earth loads..... -0.260 1.27 -0.33
Balancing earth load..... -3.405 -1.75 5.97
Water loads..... -3.797 -1.33 5.05
Foundation:
Vertical force on base..... -7.357 -3.43 25.22
Uplift..... -7.283 -7.76 56.52
==========
** Statics Check ** SUMS = 0.000 0.000 0.00

Angle of base = 15.07 degrees
Normal force on base = 7.989 kips
Shear force on base = 1.375 kips
Max. available shear force = 13.463 kips

Base pressure at x= 10.28 ft from toe = 0.0000 ksf
Base pressure at toe = 1.4309 ksf

Xr (measured from toe) = 3.43 ft
Resultant ratio = 0.2637
Stem ratio = 0.1923
Base in compression = 79.10 %
Overturning ratio = 1.42

Volume of concrete = 1.91 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures

do not result in excessive differential settlement of
the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.000
2	7.461	4.694
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.0000	0.0000		
2		0.0000	0.9585	
2		13.0000	0.3268	
3	0.0000	0.3268		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 904.81 ft
Point 2 (right), x = 13.00 ft, y = 908.31 ft

Depth of cracking = 6.82 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	0.000	0.000	0.000	0.000	0.000
2	15.068	13.463	11.477	13.463	8.652
3	45.913	5.251	0.846	4.887	0.798

Wedge	Net force

number	(kips)
1	0.000
2	-1.658
3	1.658
SUM =	0.001

+----------------------------+
| Factor of safety = 9.617 |
+----------------------------+

****
** Bearing Results **
****

Base width = 13.463 (ft)
Xr = 3.428 (ft)
Effective base width = 7.100 (ft)
(measured along slope)
Base slope = 15.0685 (deg)

phi = 0.000 (deg)
c = 1.000 (ksf)
Effective gamma = 0.0626 (kcf)

Normal load = 7.989 (kips)
Load inclination = 9.766 (deg)
Load eccentricity = 3.182 (ft)

Surcharge = 0.2453 (ksf)
Embedment = 3.723 (ft)
Ground slope = -0.0133 (deg)

Bearing Capacity Factors

	C	Q	G
Bearing	5.1400	1.0000	0.0267
Embedment	1.1049	1.0000	1.0000
Inclination	0.7947	0.7947	0.0000
Base Tilt	0.8977	1.0000	1.0000
Ground Slope	1.0052	1.0268	1.0268

Net ultimate bearing pressure = 4.0727 (ksf)

```
+--------------------------------+
| Factor of safety =      3.619 |
+--------------------------------+
```

```
+--------------------------------+
| Factor of safety =      1.793 |
+--------------------------------+
```

RS16+28 RB7+80 8ft Wall Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/ 8/21 Time: 10.07.27

Structural geometry data:
Elevation of top of stem (ELTS) = 930.66 ft
Height of stem (HTS) = 8.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.00 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 4.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 10.50 ft
Depth of key (HK) = 3.00 ft
Width of bottom of key (TK) = 2.50 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	917.66
0.00	922.66
4.50	922.66
4.50	930.66
6.00	930.66
6.00	922.66
10.50	922.66
10.50	920.66
2.50	920.66
2.50	917.66

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	924.20

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.48	14.50
2	-7.20	10.00
3	-0.53	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1020.00	893.12
2	-520.00	893.12
3	-20.00	915.20
4	-10.00	921.20
5	4.50	924.20

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	924.20	0.23

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	6.00	924.20
2	506.00	933.78

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)

phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 929.86 ft
Resisting side elevation = 924.16 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Strip load data:

Dist. to left end (ft)	Width (ft)	Magnitude (k/ft)
-20.00	-10.00	0.00

Horizontal line load data:

Elevation (ft)	Force (kips)
930.30	0.50

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***

* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 87.98 %
Overturning ratio = 1.50

Xr (measured from toe) = 3.08 ft
Resultant ratio = 0.2933
Stem ratio = 0.4286
Base pressure at x= 9.24 ft from toe = 0.0000 ksf
Base pressure at toe = 1.0771 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 2.36

****
* Bearing *
****

Net ultimate bearing pressure = 2.4471 (ksf)
Factor of safety = 2.826

********** Output Results **********

Date: 2025/ 8/21 Time: 10.07.27

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = -69.7514
Calculated earth pressure coefficients:
Driving side at rest K = 0.4433
Driving side at rest Kc = 0.7296
Resisting side at rest K = 0.5407
Resisting side at rest Kc = 0.7353
At-rest K's for resisting side calculated.

Depth of cracking = 0.00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
929.86	0.0000
923.27	0.4112
917.66	0.6617

Earth pressures:

Elevation (ft)	Pressure (ksf)
923.27	0.0000
917.66	0.1787

Surcharge pressures:

Elev. (ft)	Press. (ksf)
923.27	0.000
917.66	0.000

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
924.16	0.0000
920.66	0.2806
920.66	0.6258
917.66	0.6617

Earth pressures:

Elevation (ft)	Pressure (ksf)
924.29	0.0000
920.66	0.0931

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
920.66	0.9249
917.66	0.9249

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.6617
2.50	0.6258
2.50	0.3955
10.50	0.2806

** Forces and moments **

Part	Force (kips)		Mom. Arm (ft)	Moment (ft-k)
	Vert.	Horiz.		
Structure:				
Structure weight.....	6.075		-5.99	-36.39
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	0.605		-7.93	-4.79
Water above structure.....	0.000		0.00	0.00
Water above soil.....	1.720		-8.31	-14.29
External vertical loads....	0.000		0.00	0.00
Ext. horz. pressure loads..		0.000	0.00	0.00
Ext. horz. line loads.....		0.500	9.64	4.82
Structure, resisting side:				
Moist soil.....	0.045		-1.86	-0.08
Saturated soil.....	0.844		-2.25	-1.90
Water above structure.....	0.000		0.00	0.00
Water above soil.....	0.000		0.00	0.00
Driving side:				
Effective earth loads.....		0.501	-1.14	-0.57
Shear (due to delta).....	0.000		0.00	0.00
Horiz. surcharge effects...		0.000	-920.66	0.00
Water loads.....		4.364	1.21	5.30
Resisting side:				
Effective earth loads.....		-0.169	1.19	-0.20
Balancing earth load.....		-2.775	-1.50	4.15
Water loads.....		-2.422	-0.95	2.30
Foundation:				
Vertical force on base.....	-4.975		-3.08	15.32
Uplift.....	-4.313		-6.11	26.33

** Statics Check ** SUMS = 0.000 0.000 0.00

Angle of base = 15.95 degrees
Normal force on base = 5.546 kips
Shear force on base = 1.301 kips
Max. available shear force = 3.476 kips

Base pressure at x= 9.24 ft from toe = 0.0000 ksf
Base pressure at toe = 1.0771 ksf

Xr (measured from toe) = 3.08 ft
Resultant ratio = 0.2933
Stem ratio = 0.4286
Base in compression = 87.98 %
Overturning ratio = 1.50

Volume of concrete = 1.50 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.823
2	1.855	1.720
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)

```
 ================================================
   1   0.4363   0.6617
   2            0.0000    0.6617
   2           10.5000    0.2806
   3   0.0000   0.2806

Points of sliding plane:
 Point 1 (left),  x =    0.00 ft,   y =   917.66 ft
 Point 2 (right), x =   10.50 ft,   y =   920.66 ft

Depth of cracking =    0.00 ft

      Failure  Total   Weight   Submerged  Uplift
 Wedge  angle  length of wedge  length    force
 number  (deg)   (ft)  (kips)    (ft)     (kips)
 ======================================================
   1  -69.550   5.559    0.681   5.559    3.052
   2   15.945  10.920    8.600  10.920    5.145
   3   41.032   5.648    0.962   5.331    0.748

 Wedge    Net force
 number    (kips)
 ==================
   1      -3.529
   2       2.495
   3       1.034
 ==================
   SUM =   0.000

+----------------------------+
| Factor of safety =   2.356 |
+----------------------------+




**********************
**  Bearing Results  **
**********************

        Base width =    10.920 (ft)
             Xr =     3.079 (ft)
 Effective base width =     6.405 (ft)
  (measured along slope)
        Base slope =   15.9454 (deg)

            phi =    28.000 (deg)
```

c = 0.000 (ksf)
Effective gamma = 0.0626 (kcf)

Normal load = 5.546 (kips)
Load inclination = 13.206 (deg)
Load eccentricity = 2.258 (ft)

Surcharge = 0.2342 (ksf)
Embedment = 3.626 (ft)
Ground slope = -0.0192 (deg)

Bearing Capacity Factors

	C	Q	G
Bearing	25.8033	14.7199	11.1897
Embedment	1.1885	1.0942	1.0942
Inclination	0.7281	0.7281	0.2792
Base Tilt	0.7060	0.7259	0.7259
Ground Slope	1.0415	1.0387	1.0387

Net ultimate bearing pressure = 2.4471 (ksf)

Factor of safety = 2.826

RS16+28 RB7+80 8ft Wall Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/12/15 Time: 9.22.35

Structural geometry data:
Elevation of top of stem (ELTS) = 930.66 ft
Height of stem (HTS) = 8.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 5.00 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 2.00 ft
Width of toe (TWIDTH) = 4.50 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 10.50 ft
Depth of key (HK) = 3.00 ft
Width of bottom of key (TK) = 2.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	917.66
0.00	922.66
4.50	922.66
4.50	930.66
6.00	930.66
6.00	922.66
10.50	922.66
10.50	920.66
2.00	920.66
2.00	917.66

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	924.20

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.48	14.50
2	-7.20	10.00
3	-0.53	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1020.00	893.12
2	-520.00	893.12
3	-20.00	915.20
4	-10.00	921.20
5	4.50	924.20

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	924.20	0.97

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	6.00	924.20
2	506.00	964.62

Foundation property data:
phi for soil-structure interface = 0.00 (deg)
c for soil-structure interface = 1.000 (ksf)

phi for soil-soil interface = 0.00 (deg)
c for soil-soil interface = 1.000 (ksf)

Water data:
Driving side elevation = 930.66 ft
Resisting side elevation = 924.20 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
930.66	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 86.81 %
Overturning ratio = 1.38

Xr (measured from toe) = 3.04 ft
Resultant ratio = 0.2894
Stem ratio = 0.4286
Base pressure at x= 9.12 ft from toe = 0.0000 ksf

Base pressure at toe = 0.9234 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 14.70

****
* Bearing *
****

Net ultimate bearing pressure = 4.1482 (ksf)
Factor of safety = 5.674

********** Output Results **********

Date: 2025/12/15 Time: 9.22.35

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = 0.0000
Calculated earth pressure coefficients:
Driving side at rest K = 0.0000
Driving side at rest Kc = 0.0000
Resisting side at rest K = 1.0806
Resisting side at rest Kc = 1.0395
At-rest K's for resisting side calculated.

Depth of cracking = 5.61 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
930.66	0.0000
917.66	0.8112

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
924.20	0.0000
920.66	0.3196
920.66	0.7661
917.66	0.8112

Earth pressures:

Elevation (ft)	Pressure (ksf)
924.56	0.0000
920.66	0.1800

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
920.66	0.6966
917.66	0.6966

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.8112
2.00	0.7661
2.00	0.5113
10.50	0.3196

** Forces and moments **

Part	Force (kips)		Mom. Arm (ft)	Moment (ft-k)
	Vert.	Horiz.		
Structure:				
Structure weight.....	5.850		-5.90	-34.54
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00

```
 Saturated soil.....  0.605          -7.93    -4.79
 Water above structure.....  0.000          0.00    0.00
 Water above soil.....  1.945          -8.30   -16.14
 External vertical loads....  0.000          0.00    0.00
 Ext. horz. pressure loads..  0.000    0.00    0.00
 Ext. horz. line loads.....  0.100    10.00    1.00
Structure, resisting side:
 Moist soil.....  0.000          0.00    0.00
 Saturated soil.....  0.969          -2.17    -2.10
 Water above structure.....  0.000          0.00    0.00
 Water above soil.....  -0.051          -1.50    0.08
Driving side:
 Effective earth loads.....  0.000    0.00    0.00
 Shear (due to delta).....  0.000          0.00    0.00
 Horiz. surcharge effects...  0.000    0.00    0.00
 Water loads.....  5.273    1.34    7.04
Resisting side:
 Effective earth loads.....  -0.351    1.30   -0.46
 Balancing earth load.....  -2.090   -1.50    3.13
 Water loads.....  -2.932   -0.97    2.83
Foundation:
 Vertical force on base.....  -4.208          -3.04    12.79
 Uplift.....  -5.108          -6.10    31.16
=========================================================================
** Statics Check **  SUMS =   0.000    0.000          0.00

Angle of base     =  15.95 degrees
Normal force on base =  4.621 kips
Shear force on base  =  0.853 kips
Max. available shear force =  10.920 kips

Base pressure at x=  9.12 ft from toe =  0.0000 ksf
Base pressure at toe          =  0.9234 ksf

Xr (measured from toe) =  3.04 ft
Resultant ratio     =  0.2894
Stem ratio          =  0.4286
Base in compression  =  86.81 %
Overturning ratio   =  1.38

Volume of concrete =  1.44 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
```

do not result in excessive differential settlement of
the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.000
2	5.373	1.893
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.0000	0.0000		
2		0.0000		0.8112
2		10.5000		0.3196
3	0.0000	0.3196		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 917.66 ft
Point 2 (right), x = 10.50 ft, y = 920.66 ft

Depth of cracking = 5.61 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	0.000	0.000	0.000	0.000	0.000
2	15.945	10.920	8.642	10.920	6.174
3	50.279	5.441	0.840	4.602	0.735

Wedge	Net force

number	(kips)
1	0.000
2	-1.590
3	1.591
SUM =	0.000

+----------------------------+
| Factor of safety = 14.703 |
+----------------------------+

****
** Bearing Results **
****

Base width = 10.920 (ft)
Xr = 3.038 (ft)
Effective base width = 6.320 (ft)
(measured along slope)
Base slope = 15.9454 (deg)

phi = 0.000 (deg)
c = 1.000 (ksf)
Effective gamma = 0.0626 (kcf)

Normal load = 4.621 (kips)
Load inclination = 10.464 (deg)
Load eccentricity = 2.300 (ft)

Surcharge = 0.2644 (ksf)
Embedment = 3.904 (ft)
Ground slope = -0.0807 (deg)

Bearing Capacity Factors

	C	Q	G
Bearing	5.1400	1.0000	0.1611
Embedment	1.1235	1.0000	1.0000
Inclination	0.7810	0.7810	0.0000
Base Tilt	0.8917	1.0000	1.0000
Ground Slope	1.0314	1.1682	1.1682

Net ultimate bearing pressure = 4.1482 (ksf)

+--------------------------------+
| Factor of safety = 5.674 |
+--------------------------------+

RS 33+40 RB 25+20 _ ALT _ 6FT WALL-Drained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/12/16 Time: 9.21.01

Structural geometry data:
Elevation of top of stem (ELTS) = 921.20 ft
Height of stem (HTS) = 6.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 4.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 1.50 ft
Width of toe (TWIDTH) = 4.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 9.00 ft
Depth of key (HK) = 3.00 ft
Width of bottom of key (TK) = 1.50 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	910.70
0.00	915.20
3.50	915.20
3.50	921.20
5.00	921.20
5.00	915.20
9.00	915.20
9.00	913.70
1.50	913.70
1.50	910.70

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	916.70

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-1.20	10.00
2	-9.00	20.00
3	-1.33	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1026.50	845.28
2	-526.50	845.28
3	-26.50	900.70
4	-6.50	915.70
5	3.50	916.70

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	916.70	0.18

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	5.00	916.70
2	505.00	924.20

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)

phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 920.88 ft
Resisting side elevation = 916.70 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.80	0.50

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 90.62 %
Overturning ratio = 1.56

Xr (measured from toe) = 2.72 ft
Resultant ratio = 0.3021
Stem ratio = 0.4444
Base pressure at x= 8.16 ft from toe = 0.0000 ksf

Base pressure at toe = 0.8516 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 2.57

****
* Bearing *
****

Net ultimate bearing pressure = 2.1750 (ksf)
Factor of safety = 3.188

********** Output Results **********

Date: 2025/12/16 Time: 9.21.01

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = -62.1229
Calculated earth pressure coefficients:
Driving side at rest K = 0.4865
Driving side at rest Kc = 0.6879
Resisting side at rest K = 0.5385
Resisting side at rest Kc = 0.7338
At-rest K's for resisting side calculated.

Depth of cracking = 0.00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
920.88	0.0000
916.35	0.2827
910.70	0.5540

Earth pressures:

Elevation (ft)	Pressure (ksf)
916.35	0.0000
910.70	0.1950

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
916.70	0.0000
913.70	0.2303
913.70	0.5369
910.70	0.5540

Earth pressures:

Elevation (ft)	Pressure (ksf)
916.76	0.0000
913.70	0.0818

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
913.70	0.6492
910.70	0.6492

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.5540
1.50	0.5369
1.50	0.3156
9.00	0.2303

** Forces and moments **

==========

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)
Structure:				
Structure weight.....	4.050		-5.21	-21.09
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	0.580		-7.17	-4.16
Water above structure.....	0.000		0.00	0.00
Water above soil.....	0.951		-7.27	-6.92
External vertical loads....	0.000		0.00	0.00
Ext. horz. pressure loads..		0.000	0.00	0.00
Ext. horz. line loads.....		0.500	7.10	3.55
Structure, resisting side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	0.765		-1.99	-1.52
Water above structure.....	0.000		0.00	0.00
Water above soil.....	-0.007		-1.33	0.01
Driving side:				
Effective earth loads.....		0.551	-1.13	-0.62
Shear (due to delta).....	0.000		0.00	0.00
Horiz. surcharge effects...		0.000	0.00	0.00
Water loads.....		3.004	0.51	1.54
Resisting side:				
Effective earth loads.....		-0.125	0.99	-0.12
Balancing earth load.....		-1.948	-1.51	2.94
Water loads.....		-1.982	-1.07	2.12
Foundation:				
Vertical force on base.....	-3.473		-2.72	9.44
Uplift.....	-2.866		-5.18	14.83

** Statics Check ** SUMS = 0.000 0.000 0.00

Angle of base = 18.43 degrees
Normal force on base = 3.911 kips
Shear force on base = 0.750 kips
Max. available shear force = 2.647 kips

Base pressure at x= 8.16 ft from toe = 0.0000 ksf
Base pressure at toe = 0.8516 ksf

Xr (measured from toe) = 2.72 ft
Resultant ratio = 0.3021
Stem ratio = 0.4444
Base in compression = 90.62 %
Overturning ratio = 1.56

Volume of concrete = 1.00 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.859
2	1.140	0.944
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.3010	0.5540		
2			0.0000	0.5540
2			9.0000	0.2303
3	0.0000	0.2303		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 910.70 ft
Point 2 (right), x = 9.00 ft, y = 913.70 ft

Depth of cracking = 0.00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)

```
 ========================================================
   1  -61.196    6.112     1.040   6.112    2.613
   2   18.435    9.487     6.520   9.487    3.720
   3   41.556    4.692     0.670   4.523    0.521

 Wedge     Net force
 number     (kips)
 ===================
   1       -3.040
   2        2.319
   3        0.721
 ===================
    SUM =    0.000

+----------------------------+
| Factor of safety =    2.570 |
+----------------------------+




**********************
**  Bearing Results  **
**********************

         Base width =      9.487 (ft)
               Xr =      2.719 (ft)
 Effective base width =      5.731 (ft)
  (measured along slope)
         Base slope =    18.4349 (deg)

              phi =     28.000 (deg)
                c =      0.000 (ksf)
     Effective gamma =     0.0626 (kcf)

        Normal load =      3.911 (kips)
    Load inclination =     10.851 (deg)
   Load eccentricity =      1.878 (ft)

          Surcharge =     0.1950 (ksf)
          Embedment =      3.060 (ft)
       Ground slope =    -0.0150 (deg)


        Bearing Capacity Factors
==========================================
               C       Q       G
```

==========

Bearing	25.8033	14.7199	11.1897
Embedment	1.1777	1.0889	1.0889
Inclination	0.7734	0.7734	0.3751
Base Tilt	0.6643	0.6871	0.6871
Ground Slope	1.0324	1.0302	1.0302

Net ultimate bearing pressure = 2.1750 (ksf)

Factor of safety = 3.188

RS 33+40 RB 25+20 _ ALT _ 6FT WALL-Undrained
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/11/ 7 Time: 14.08.31

Structural geometry data:
Elevation of top of stem (ELTS) = 921.20 ft
Height of stem (HTS) = 6.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 4.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 1.50 ft
Width of toe (TWIDTH) = 3.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 6.00 ft
Depth of key (HK) = 3.00 ft
Width of bottom of key (TK) = 2.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	910.70
0.00	915.20
1.50	915.20
1.50	921.20
3.00	921.20
3.00	915.20
6.00	915.20
6.00	913.70
2.00	913.70
2.00	910.70

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
0.00	1.000	0.120	0.125	0.00	916.70

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-1.20	10.00
2	-9.00	20.00
3	-1.33	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1028.50	845.28
2	-528.50	845.28
3	-28.50	900.70
4	-8.50	915.70
5	1.50	916.70

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
0.00	1.000	0.120	0.125	916.70	0.18

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	3.00	916.70
2	503.00	924.20

Foundation property data:
phi for soil-structure interface = 0.00 (deg)
c for soil-structure interface = 1.000 (ksf)

phi for soil-soil interface = 0.00 (deg)
c for soil-soil interface = 1.000 (ksf)

Water data:
Driving side elevation = 920.88 ft
Resisting side elevation = 916.70 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
920.80	0.10

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 77.32 %
Overturning ratio = 1.38

Xr (measured from toe) = 1.55 ft
Resultant ratio = 0.2577
Stem ratio = 0.5000
Base pressure at x= 4.64 ft from toe = 0.0000 ksf

Base pressure at toe = 1.0218 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 95.13

****
* Bearing *
****

Net ultimate bearing pressure = 4.3912 (ksf)
Factor of safety = 6.007

********** Output Results **********

Date: 2025/11/ 7 Time: 14.08.31

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = 0.0000
Calculated earth pressure coefficients:
Driving side at rest K = 0.0000
Driving side at rest Kc = 0.0000
Resisting side at rest K = 1.0150
Resisting side at rest Kc = 1.0075
At-rest K's for resisting side calculated.

Depth of cracking = 5.85 ft
Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
920.88	0.0000
910.70	0.6352

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
916.70	0.0000
913.70	0.2678
913.70	0.5952
910.70	0.6352

Earth pressures:

Elevation (ft)	Pressure (ksf)
916.74	0.0000
913.70	0.1143

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
913.70	0.3040
910.70	0.3040

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.6352
2.00	0.5952
2.00	0.3479
6.00	0.2678

** Forces and moments **

Part	Force (kips)		Mom. Arm (ft)	Moment (ft-k)
	Vert.	Horiz.		
Structure:				
Structure weight.....	3.600		-3.78	-13.61
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00

```
  Saturated soil.............    0.267              -5.24     -1.40
  Water above structure......    0.000               0.00      0.00
  Water above soil...........    0.398              -5.25     -2.09
  External vertical loads....    0.000               0.00      0.00
  Ext. horz. pressure loads..            0.000       0.00      0.00
  Ext. horz. line loads......            0.100       7.10      0.71
Structure, resisting side:
  Moist soil.................    0.000               0.00      0.00
  Saturated soil.............    0.571              -1.49     -0.85
  Water above structure......    0.000               0.00      0.00
  Water above soil...........   -0.004              -0.99      0.00
Driving side:
  Effective earth loads......            0.000       0.00      0.00
  Shear (due to delta).......    0.000               0.00      0.00
  Horiz. surcharge effects...            0.000       0.00      0.00
  Water loads................            3.233       0.39      1.26
Resisting side:
  Effective earth loads......           -0.174       1.03     -0.18
  Balancing earth load.......           -0.912      -1.49      1.36
  Water loads................           -2.247      -1.07      2.40
Foundation:
  Vertical force on base.....   -2.370              -1.55      3.67
  Uplift.....................   -2.462              -3.55      8.74
=====================================================================
** Statics Check **   SUMS =     0.000      0.000              0.00

Angle of base        =    26.57 degrees
Normal force on base =    2.528 kips
Shear force on base  =   -0.244 kips
Max. available shear force =    6.708 kips

Base pressure at x=   4.64 ft from toe =   0.0000 ksf
Base pressure at toe                =   1.0218 ksf

Xr (measured from toe) =     1.55 ft
Resultant ratio        =   0.2577
Stem ratio             =   0.5000
Base in compression    =    77.32 %
Overturning ratio      =     1.38

Volume of concrete =     0.89 cubic yds/ft of wall

NOTE:  The engineer shall verify that the computed
bearing pressures below the wall do not exceed the
allowable foundation bearing pressure, or, perform a
bearing capacity analysis using the program CBEAR.
Also, the engineer shall verify that the base pressures
```

do not result in excessive differential settlement of
the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.000
2	3.333	0.394
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.0000	0.0000		
2		0.0000	0.6352	
2		6.0000	0.2678	
3	0.0000	0.2678		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 910.70 ft
Point 2 (right), x = 6.00 ft, y = 913.70 ft

Depth of cracking = 5.85 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	0.000	0.000	0.000	0.000	0.000
2	26.565	6.708	4.813	6.708	3.029
3	48.584	4.115	0.517	4.000	0.536

Wedge	Net force

number	(kips)
1	0.000
2	-0.651
3	0.652
SUM =	0.001

+----------------------------+
| Factor of safety = 95.135 |
+----------------------------+

****
** Bearing Results **
****

Base width = 6.708 (ft)
Xr = 1.546 (ft)
Effective base width = 3.458 (ft)
(measured along slope)
Base slope = 26.5651 (deg)

phi = 0.000 (deg)
c = 1.000 (ksf)
Effective gamma = 0.0626 (kcf)

Normal load = 2.528 (kips)
Load inclination = 5.521 (deg)
Load eccentricity = 1.625 (ft)

Surcharge = 0.1932 (ksf)
Embedment = 3.045 (ft)
Ground slope = -0.0150 (deg)

Bearing Capacity Factors

	C	Q	G
Bearing	5.1400	1.0000	0.0300
Embedment	1.1761	1.0000	1.0000
Inclination	0.8811	0.8811	0.0000
Base Tilt	0.8196	1.0000	1.0000
Ground Slope	1.0058	1.0302	1.0302

Net ultimate bearing pressure = 4.3912 (ksf)

+--------------------------------+
| Factor of safety = 6.007 |
+--------------------------------+

+--------------------------------+
| Factor of safety = 2.649 |
+--------------------------------+

RS3+33 LB 9+00 4ft Wall
Updated for 75% Compression and 0.5k impact

********** Echoprint of Input Data **********

Date: 2025/ 9/12 Time: 9.27.55

Structural geometry data:
Elevation of top of stem (ELTS) = 938.50 ft
Height of stem (HTS) = 4.00 ft
Thickness top of stem (TTS) = 1.50 ft
Thickness bottom of stem (TBS) = 1.50 ft
Dist. of batter at bot. of stem (TBSR)= 0.00 ft
Depth of heel (THEEL) = 2.50 ft
Distance of batter for heel (BTRH) = 0.00 ft
Depth of toe (TTOE) = 1.50 ft
Width of toe (TWIDTH) = 3.00 ft
Distance of batter for toe (BTRT) = 0.00 ft
Width of base (BWIDTH) = 8.00 ft
Depth of key (HK) = 1.00 ft
Width of bottom of key (TK) = 1.00 ft
Dist. of batter at bot. of key (BTRK) = 0.00 ft

Structure coordinates:

x (ft)	y (ft)
0.00	932.00
0.00	934.50
3.50	934.50
3.50	938.50
5.00	938.50
5.00	934.50
8.00	934.50
8.00	933.00
1.00	933.00
1.00	932.00

NOTE: X=0 is located at the left-hand side
of the structure. The Y values correspond
to the actual elevation used.

Structural property data:
Unit weight of concrete = 0.150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
28.00	0.000	0.120	0.125	0.00	936.05

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	-2.52	5.00
2	-6.86	17.50
3	-0.55	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1019.00	902.08
2	-519.00	902.08
3	-19.00	925.00
4	-1.50	935.00
5	3.50	936.05

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
28.00	0.000	0.120	0.125	936.05	0.55

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	5.00	936.05
2	505.00	958.97

Foundation property data:
phi for soil-structure interface = 28.00 (deg)
c for soil-structure interface = 0.000 (ksf)

phi for soil-soil interface = 28.00 (deg)
c for soil-soil interface = 0.000 (ksf)

Water data:
Driving side elevation = 938.40 ft
Resisting side elevation = 936.05 ft
Unit weight of water = 0.0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

Elevation (ft)	Force (kips)
938.40	0.50

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = 0.6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

***** Summary of Results *****

********** *** Satisfied ***
* Overturning * Required base in comp. = 75.00 %
********** Actual base in comp. = 92.51 %
Overturning ratio = 1.48

Xr (measured from toe) = 2.47 ft
Resultant ratio = 0.3084
Stem ratio = 0.3750
Base pressure at x= 7.40 ft from toe = 0.0000 ksf

Base pressure at toe = 0.6544 ksf

********** *** Satisfied ***
* Sliding * Min. Required = 1.50
********** Actual FS = 2.92

****
* Bearing *
****

Net ultimate bearing pressure = 2.3876 (ksf)
Factor of safety = 4.692

********** Output Results **********

Date: 2025/ 9/12 Time: 9.27.55

****
** Overturning Results **
****

Solution converged in 1 iterations.

SMF used to calculate K's = 0.6667
Alpha for the SMF = -69.1098
Calculated earth pressure coefficients:
Driving side at rest K = 0.4483
Driving side at rest Kc = 0.7204
Resisting side at rest K = 0.5548
Resisting side at rest Kc = 0.7449
At-rest K's for resisting side calculated.

Depth of cracking = 0.00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
938.40	0.0000
935.32	0.1925
932.00	0.3657

Earth pressures:

Elevation (ft)	Pressure (ksf)
935.32	0.0000
932.00	0.0949

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
936.05	0.0000
933.00	0.2213
933.00	0.3566
932.00	0.3657

Earth pressures:

Elevation (ft)	Pressure (ksf)
936.19	0.0000
933.00	0.0979

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
933.00	1.0248
932.00	1.0248

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
0.00	0.3657
1.00	0.3566
1.00	0.2851
8.00	0.2213

** Forces and moments **

==========

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)
Structure:				
Structure weight.....	2.850		-4.11	-11.70
Structure, driving side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	0.517		-6.07	-3.14
Water above structure.....	0.000		0.00	0.00
Water above soil.....	0.594		-6.33	-3.76
External vertical loads....	0.000		0.00	0.00
Ext. horz. pressure loads..		0.000	0.00	0.00
Ext. horz. line loads.....		0.500	5.40	2.70
Structure, resisting side:				
Moist soil.....	0.000		0.00	0.00
Saturated soil.....	0.607		-1.48	-0.90
Water above structure.....	0.000		0.00	0.00
Water above soil.....	-0.013		-1.01	0.01
Driving side:				
Effective earth loads.....		0.157	0.16	0.03
Shear (due to delta).....	0.000		0.00	0.00
Horiz. surcharge effects...		0.000	0.00	0.00
Water loads.....		1.222	1.18	1.45
Resisting side:				
Effective earth loads.....		-0.156	1.06	-0.17
Balancing earth load.....		-1.025	-0.48	0.50
Water loads.....		-0.699	0.24	-0.17
Foundation:				
Vertical force on base.....	-2.422		-2.47	5.97
Uplift.....	-2.133		-4.30	9.17

** Statics Check ** SUMS = 0.000 0.000 0.00

Angle of base = 7.13 degrees
Normal force on base = 2.530 kips
Shear force on base = 0.717 kips
Max. available shear force = 1.543 kips

Base pressure at x= 7.40 ft from toe = 0.0000 ksf
Base pressure at toe = 0.6544 ksf

Xr (measured from toe) = 2.47 ft
Resultant ratio = 0.3084
Stem ratio = 0.3750
Base in compression = 92.51 %
Overturning ratio = 1.48

Volume of concrete = 0.70 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	0.000	0.234
2	0.797	0.581
3	0.000	0.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	0.2078	0.3657		
2			0.0000	0.3657
2			8.0000	0.2213
3	0.0000	0.2213		

Points of sliding plane:
Point 1 (left), x = 0.00 ft, y = 932.00 ft
Point 2 (right), x = 8.00 ft, y = 933.00 ft

Depth of cracking = 0.00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)

```
 ======================================================
   1  -69.148    3.285     0.242    3.285     0.942
   2    7.125    8.062     4.349    8.062     2.366
   3   45.796    4.654     0.643    4.255     0.471

 Wedge     Net force
 number     (kips)
 ===================
   1       -1.113
   2        0.308
   3        0.806
 ===================
    SUM =    0.001

+----------------------------+
| Factor of safety =    2.922 |
+----------------------------+




**********************
**  Bearing Results  **
**********************

         Base width =      8.062 (ft)
               Xr =      2.467 (ft)
 Effective base width =      4.972 (ft)
  (measured along slope)
         Base slope =     7.1250 (deg)

              phi =     28.000 (deg)
                c =      0.000 (ksf)
     Effective gamma =     0.0626 (kcf)

         Normal load =      2.530 (kips)
    Load inclination =     15.813 (deg)
   Load eccentricity =      1.545 (ft)

          Surcharge =     0.2072 (ksf)
          Embedment =      3.188 (ft)
       Ground slope =    -0.0458 (deg)


        Bearing Capacity Factors
==========================================
               C       Q       G
```

==========

Bearing	25.8033	14.7199	11.1897
Embedment	1.2134	1.1067	1.1067
Inclination	0.6795	0.6795	0.1895
Base Tilt	0.8628	0.8721	0.8721
Ground Slope	1.1006	1.0938	1.0938

Net ultimate bearing pressure = 2.3876 (ksf)

Factor of safety = 4.692

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Calculation Description

PROJECT

LOCATION, STATE

ASSUMPTIONS

- Wall experiences maximum horizontal deflection of 1.5" as recommended for extreme conditions in Table 8.1, EM 1110-2-2502
- Assumption
- Assumption

References

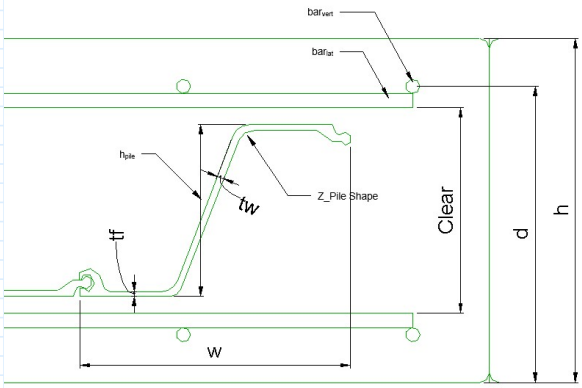
- Floodwalls and Hydraulic Retaining Walls, EM 1110-2-2502, 2022
- Strength Design for Reinforced Concrete Hydraulic Structures, EM 1110-2-2104, 2025
- Stability Analysis of Concrete Structures, EM 1110-2-2100, 2005
- Earth Pressure and Retaining Wall Basics for Non-Geotechnical Engineers, Richard P. Weber, P.E., 2012

LOAD FACTORS

$\gamma_{pile.N} := 1.4$ *(minimum load factor for sheet piling, EM 1110-2-2502, Table 9.5)*

do I multiply the hydrostatic and soil forces by this? -LKP

CONCRETE CAP



$h := 18\ in$ *(thickness of concrete cap to I wall sheet pile)*

$bar_{vert} := 0.75\ in$ *(vertical bar size)*

$bar_{lat} := 0.75\ in$ *(lateral bar size)*

$spacing_{vert} := 12\ in$ *(vertical bar spacing)*

$spacing_{lat} := 12\ in$ *(lateral bar spacing)*

$cover_{vert} := 3\ in$ *(minimum cover requirement EM 1110-2-2104, Table 2-1)*

$d := h - cover_{vert} = 15\ in$ *(depth to reinforcement)*

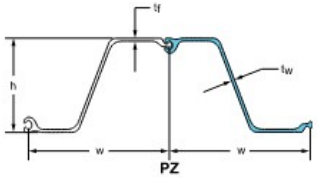
$Clear := h - \left(2 \cdot cover_{vert}\right) - \left(2 \cdot bar_{vert}\right) - \left(2 \cdot bar_{lat}\right) = 9\ in$ *(clear distance inside of reinforcement for z pile in wall)*

STEEL PILE

Steel Z Pile:

NUCOR® SKYLINE

HOT ROLLED STEEL SHEET PILE
PZ



PZ

SECTION	Width (w) in / mm	Height (h) in / mm	THICKNESS Flange (t_f) in / mm	THICKNESS Web (t_w) in / mm	Cross Sectional Area in²/ft / cm²/m	WEIGHT Single Pile lb/ft / kg/m	WEIGHT Wall Area lb/ft² / kg/m²	SECTION MODULUS Elastic in³/ft / cm³/m	SECTION MODULUS Plastic in³/ft / cm³/m	Moment of Inertia in⁴/ft / cm⁴/m	COATING AREA Both Sides ft²/ft of single / m²/m	COATING AREA Wall Surface ft²/ft² / m²/m²
PZ 22	22.00 / 559	9.0 / 229	0.375 / 9.50	0.375 / 9.50	6.47 / 136.9	40.3 / 60.0	22.0 / 107.4	18.1 / 973	21.79 / 1171.4	84.38 / 11500	4.48 / 1.37	1.22 / 1.22
PZ 27	18.00 / 457	12.0 / 305	0.375 / 9.50	0.375 / 9.50	7.94 / 168.1	40.5 / 60.3	27.0 / 131.8	30.2 / 1620	36.49 / 1961.9	184.20 / 25200	4.48 / 1.37	1.49 / 1.49
PZ 35	22.64 / 575	14.9 / 378	0.600 / 15.21	0.500 / 12.67	10.29 / 217.8	66.0 / 98.2	35.0 / 170.9	48.5 / 2608	57.17 / 3073.5	361.22 / 49300	5.37 / 1.64	1.42 / 1.42
PZ 40	19.69 / 500	16.1 / 409	0.600 / 15.21	0.500 / 12.67	11.77 / 249.1	65.6 / 97.6	40.0 / 195.3	60.7 / 3263	71.92 / 3866.7	490.85 / 67000	5.37 / 1.64	1.64 / 1.64

$Z\_Pile :=$ Size: PZ 22 *(Pile size)*

$w := Z\_Pile_2 = 22\ in$ *(width of z pile section)*

$h_{pile} := Z\_Pile_3 = 9\ in$ *(height of z pile section)*

$t_f := Z\_Pile_4 = 0.375\ in$ *(z pile flange thickness)*

$t_w := Z\_Pile_5 = 0.375\ in$ *(z pile web thickness)*

$S_{pile} := Z\_Pile_6 \cdot in^3 = 18.1\ in^3$ *(elastic section modulus)*

$Z_{pile} := Z\_Pile_7 \cdot in^3 = 21.8\ in^3$ *(plastic section modulus)*

$I_{pile} := Z\_Pile_8 \cdot in^4 = 84.4\ in^4$ *(moment of inertia)*

SHEAR CAPACITY OF SHEET PILE

$A_v := t_w \cdot \frac{h}{w} \cdot 1\ ft = 3.7\ in^2$ *(shear area per foot of z pile, EM 1110-1-2502 Eq 9.4)*

PROPERTIES OF I WALLS

I-Wall Dimensions

$b := 1\ ft$ *(unit width of I-wall)*

$Elev_{top} := 940\ ft$ *(top elevation of I-wall)*

$Elev_{watersurf} := 938.5\ ft$ *(elevation of water surface profile) -* ***why is this not top of wall?***

$Elev_{ground} := 931.5\ ft$ *(elevation of ground level at I-wall)*

$Elev_{bottom} := 921.5\ ft$ *(elevation of bottom of I-wall)*

$height := Elev_{watersurf} - Elev_{ground} = 7\ ft$ *(height of wall section -* ***only use this for calc of hydrostatic loading above ground - LKP)***

MATERIAL PROPERTIES

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\phi_{undrained} := 0\ deg$ *(internal friction angle of in undrained/quick analysis. Assume 0 (most conservative) according to section 6.7.6.5 of EM 1110-2-2502). Also given in DDR section 6.7.3*

$\gamma_{soil.sat} := 120\ pcf$ *(unit weight of soil, saturated -* ***was orig. 125 pcf, but DDR gives 120 pcf -LKP)***

$\gamma_{soil} := 115\ pcf$ *(unit weight of soil on site)*

$\gamma'_{soil} := \gamma_{soil.sat} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$\sigma'_n := 150\ psf$ *(effective stress normal to the wall)*

$(\delta'/\phi')_{drained} := 0.5$ *(interface friction angle ratio, EM 1110-2-2502 Table 6.2)*

$q_n := 2000\ psi$ *(assumed bearing pressure)*

$C_{A\_undrained} := 1000\ psf$ *(Cohesion)*

$k_{a\_undrained} := \tan\left(45\ deg - \frac{\phi_{undrained}}{2}\right)^2 = 1$ *Active earth pressure coefficient. As discussed above, reigning EM and DDR say to assume phi = 0, resulting in K_a of 1*

Drained Soil Conditions

$\phi_{frict\_soil\_drained} := 28\ deg$ *(internal fiction angle in the drained condition)*

$S_{A\_drained} := 100\ psf$ *(soil drained shear strength)*

$C_{A\_drained} := 0\ psf$ *(cohesion for drained conditions, EM 1110-2-2502)*

$k_{o\_drained} := 1 - \sin\left(\phi_{frict\_soil\_drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

$k_{p\_drained} := \tan\left(45\ deg + \frac{\phi_{frict\_soil\_drained}}{2}\right)^2 = 2.8$ *(passive soil pressure coefficient, resisting side, EM 1110-2-2100 eq. 3, section 5-3)*

$k_{a\_drained} := \tan\left(45\ deg - \frac{\phi_{frict\_soil\_drained}}{2}\right)^2 = 0.36$ *(active earth pressure coefficient, Rankine's Theory, also seen in section 5-3, EM 1110-2-2100)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(Youngs modulus of elasticity for steel)*

$(F_y)_{35} := 35\ ksi$ *(yield strength for 35 ksi steels, EM 1110-2-2104, Table 3-3)*

$(F_y)_{36} := 36\ ksi$ *(yield strength for 36 ksi steels)*

$(F_y)_{42} := 42\ ksi$ *(yield strength for 42 ksi steels)*

$(F_y)_{48} := 48\ ksi$ *(yield strength for 48 ksi steels)*

$(F_y)_{50} := 50\ ksi$ *(yield strength for 50 ksi steels)*

$(F_y)_{60} := 60\ ksi$ *(yield strength for 60 ksi steels)*

Concrete Properties

$b := 12\ in$ *(unit length of floodwall)*

$cover := 4\ in$ *(assumed cover to reinforcing steel)*

$f'_c := 4.5\ ksi$ *(assumed concrete compressive strength for all hydraulic structures)*

$\lambda_c := 1.0$ *(concrete modification factor)*

$\varepsilon_{ty} := 0.004$ *(yield strain in reinforcing steel)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

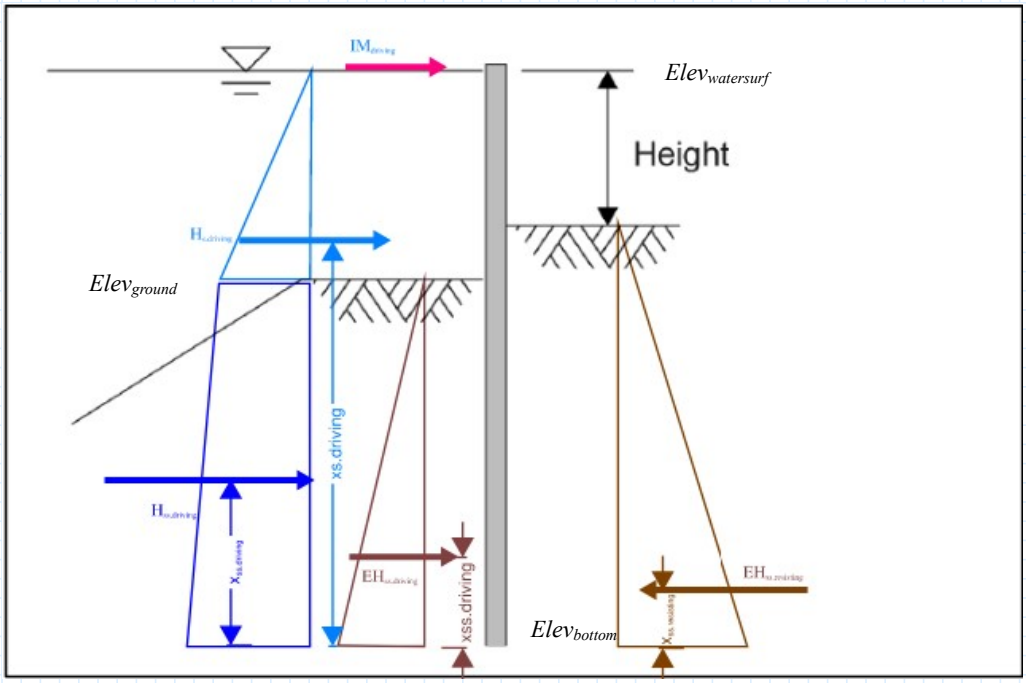
$\gamma_{concrete} := 150\ pcf$ *(unit weight of concrete)*

$f'_c := 4.5\ ksi$ *(concrete compressive strength)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength)*

$E_c := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$

LOADING CONDITIONS



Driving Side

$$P_{Hs1driving} := \frac{1}{2} \cdot \gamma_{water} \cdot \left(Elev_{watersurf} - Elev_{ground}\right) = 218.4\ psf$$

Driving side hydrostatic pressure above ground

$$P_{Hs2driving} := \frac{1}{2} \cdot \gamma_{water} \cdot \left(\left(Elev_{watersurf} - Elev_{ground}\right) + \left(Elev_{watersurf} - Elev_{bottom}\right)\right) = 748.8\ psf$$

$$F_{Htotal} := \gamma_{pile.N} \cdot \frac{1}{2} \cdot \gamma_{water} \cdot \left(Elev_{watersurf} - Elev_{bottom}\right)^2 = 12623.5\ plf$$

Driving side hydrostatic pressure below ground. Multiplied by load factor of 1.4 as given above.

$$F_{Sdriving} := \gamma_{pile.N} \cdot \frac{1}{2} \cdot \gamma'_{soil} \cdot k_{a\_undrained} \cdot \left(Elev_{ground} - Elev_{bottom}\right)^2 = 4032\ plf$$

Driving side soil force (active, soil moves away from this side. depends on degree of wall movement, assume max. 1.5" deflection). Multiplied by load factor of 1.4 as given above.
This is the item that needs work. EM 1110-2-2502 has a different equation to use if the soil is cohesive with no friction angle. But, when I use it, the value turns negative (saying that cohesion is large enough that no lateral pressures are created).

$$F_{Sresisting} := \frac{1}{2} \cdot \gamma_{soil} \cdot k_{p\_drained} \cdot \left(Elev_{ground} - Elev_{bottom}\right)^2 = 15926.5\ plf$$

Resisting side soil force (passive, soil moves towards this side) ***unrealistic, do not use - LKP***

$$F_{Sresisting} := \gamma_{pile.N} \cdot \frac{1}{2} \cdot \gamma_{soil} \cdot k_{o\_drained} \cdot \left(Elev_{ground} - Elev_{bottom}\right)^2 = 4270.8\ plf$$

Resisting side soil force (at-rest conditions)

Forces

$$F := P \cdot A$$

General equation for force caused by pressure

$$F_{Hs1driving} := P_{Hs1driving} \cdot height = 1528.8\ plf$$

Resultant hydrostatic force per linear foot of wall (good because we want per unit width)

$$F_{Hs2driving} := P_{Hs2driving} \cdot \left(Elev_{ground} - Elev_{bottom}\right) = 7488\ plf$$

Because we have an assumed horizontal deflection of 1.5", the structure has been classified as experiencing movement. Thus, active and passive earth pressures have developed.

plf is fine because we assume unit width of 1. just convert to pounds

$$F_{driving\_total} := F_{Htotal} + F_{Sdriving} = 16655.5\ plf$$

driving is lower than resisting which is not good or correct

$$F_{resisting\_total} := F_{Sresisting} = 4270.8\ plf$$

maybe we assume at rest conditions? that would make the analysis make more sense. Total angle of movement in maximum loading conditions is .018 radians, and EM 1110-2-2100 says that for passive conditions to develop, structure rotation of .02 - .2 radians required. "however, the rotation required to develop one-half the passive pressure is significantly less, as little as .005 radians"

does that mean i use half the passive pressure? or do i assume at-rest conditions???

*total horizontal force against wall = water over WHOLE length. water pressure does not change even with soil introduction. 1/2*gamma*H^2. PLUSSSSS soil force with BUOYANT gamma. buoyant gamma subtracts water from the calculation ALREADY.*

Sum the moments at the top of the wall

$d_H := \frac{2}{3} \cdot (Elev_{watersurf} - Elev_{bottom}) = 11.3\ ft$ *Point of application of hydrostatic force*

$M_H := F_{Htotal} \cdot 1\ ft \cdot d_H = 143066.6\ lbf \cdot ft$ *Moment created by hydrostatic force*

$d_S := \left((Elev_{watersurf} - Elev_{ground}) + \frac{2}{3} (Elev_{ground} - Elev_{bottom}) \right) = 13.7\ ft$ *Point of application of soil force* ***- does this differ for driving and resisting side?***

$M_S := F_{Sdriving} \cdot 1\ ft \cdot \left((Elev_{watersurf} - Elev_{ground}) + \frac{2}{3} (Elev_{ground} - Elev_{bottom}) \right) = 55104\ lbf \cdot ft$ *Moment created by soil force*

Use deflection to calculate required I

$\delta := \frac{F \cdot b^2}{6\ EI} (3\ l - b)$ *Equation for maximum deflection (at free end) in cantilevered beam, AISC*

$I := 184.20\ \frac{in^4}{ft}$ *Moment of Inertia of sheet piling per NUCOR table*

$b_h := (Elev_{watersurf} - Elev_{bottom} - d_H) = 5.7\ ft$ *B is distance of the load to the fixed end* ***- go over this with john -LKP***

$b_s := (Elev_{watersurf} - Elev_{bottom} - d_S) = 3.3\ ft$

$E_{sheet} := 48.5\ \frac{in^3}{ft}$ *Elastic modulus of sheet pile per NUCOR table* ***im confused, why is this not in a unit of force/ area ????? ksi, psi, etc.***

$\delta_H := \frac{F_{Htotal} \cdot (b_h)^2}{6\ E_{steel} \cdot I} (3\ (Elev_{top} - Elev_{bottom}) - b_h) = 1.089\ in$ *do I use E_{steel}, $E_{concrete}$, or E_{sheet}? Do i separate the deflections?*

$\delta_{S\_driving} := \frac{F_{Sdriving} \cdot (b_s)^2}{6\ E_{steel} \cdot I} (3\ (Elev_{top} - Elev_{bottom}) - b_s) = 0.126\ in$

$\delta_{S\_resisting} := \frac{F_{Sresisting} \cdot (b_s)^2}{6\ E_{steel} \cdot I} (3\ (Elev_{top} - Elev_{bottom}) - b_s) = 0.133\ in$

$\delta_{Total} := \delta_H + \delta_{S\_driving} - \delta_{S\_resisting} = 1.082\ in$ *PZ 27 is the smallest sheet pile size that does not exceed maximum deflection of 1.5"*

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Inlet and Outlet Structures

Upper Turkey Creek

Merriam, KS

ASSUMPTIONS AND REFERENCES

- References used:
  1. EM 1110-2-2104 [2025] Strength Design for Reinforced Concrete Hydraulic Structures
  2. ACI 318-19
  3. EM 1110-2-2100 [2005] Stability Analysis for Concrete Structures
  4. EM 1110-2- 2502 [2022] Floodwall and Other Hydraulic Retaining Walls
- Load factors are denoted by γ values and strength reduction factors are denoted by ϕ values
- Extreme loading cases are not required for outfall structures. Floodwall system and project defined as retaining the loading from the 100 year [1% AEP event]. Per EM 1110-2-2104 Section 3-2 d. Strength Design per EM 1110-2-2104 and ACI 318-19, Stability per EM 1110-2-2100 and assumed to be "Normal" for sliding.
- Assumed two general load conditions; 1) saturated soil to top of wall with no free water, and 2) free water to 1/2 of the height (varied along the wingwall) with the inside empty.
- Earthquake loading is ignored as the loading period for the floodwalls exceeds the loading for earthquake frequencies. Construction loads are also negated as equipment will not be allowed to pass over the outlet or inlet structures during typical construction practices.
- The design for the outlet structures will also apply to the inlet structures, design governed by the outlet structures as they are taller.

SAFETY FACTORS AND CONSTRAINTS

Load Factors

Serviceability Load Factors

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve} := 1.0$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-3)*

Strength Load Factors, Permanent Loads

$(\gamma_p)_{D.solo} := 1.4$ *(load factor for dead load, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{D.min} := 0.9$ *(load factor for dead load, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{D.max} := 1.2$ *(load factor for dead load, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{EV.min} := 1.0$ *(load factor for vertical earth pres, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{EV.max} := 1.35$ *(load factor for vertical earth pres, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{EH.d.rest} := 1.35$ *(lateral earth pressure, driving side, at rest condition, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{EH.r.rest} := 0.9$ *(lateral earth pressure, resisting side, at rest condition, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{EH.d.act} := 1.5$ *(lateral earth pressure, driving side, active condition, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{EH.r.act} := 0.5$ *(lateral earth pressure, resisting side, active condition, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{G.min} := 0$ *(load factor for gravity load, EM 1110-2-2104, 3-3d)*

$(\gamma_p)_{G.max} := 1.6$ *(load factor for gravity load, EM 1110-2-2104, 3-3d)*

Strength Load Factors, Temporary Loads

$(\gamma_{pr})_n := 1.4$ *(load factor for principal load, EM 1110-2-2104, 3-3e(2)(b))*

$(\gamma_{pr})_u := 1.5$ *(load factor for principal load, EM 1110-2-2104, 3-3e(2)(c))*

$(\gamma_{pr})_3 := 1.3$ *(load factor for principal load, condition 3,EM 1110-2-2104, 3-3e(3))*

$(\gamma_{pr})_L := 1.6$ *(load factor for live principal load, EM 1110-2-2104, 3-3e(4))*

$(\gamma_{pr})_T := 1.0$ *(load factor for self-straining principal load, EM 1110-2-2104, 3-3e(4))*

$(\gamma_{pr})_{ES} := 1.0$ *(load factor for soil Surcharge principal load, EM 1110-2-2104, Table 3-2)*

$(\gamma_c)_{Lt} := 1.0$ *(load factor for temporary, companion load, EM 1110-2-2104, 3-3f))*

Strength Load Factors, Dynamic Loads

$(\gamma_p)_{Hw} := 1.2$ *(load factor for gravity load, EM 1110-2-2104, 3-3d)*

$(\gamma_{pr})_W := 1.0$ *(load factor for wind principal load, EM 1110-2-2104, 3-3e(4))*

$(\gamma_{pr})_{EQ.min} := 1.0$ *(load factor for earthquake principal load, EM 1110-2-2104, Table 3-2)*

$(\gamma_{pr})_{EQ.max} := 1.25$ *(load factor for earthquake principal load, EM 1110-2-2104, Table 3-2)*

$(\gamma_c)_L := 1.0$ *(load factor for live companion load, EM 1110-2-2104, 3-3f))*

$(\gamma_c)_T := 0.75$ *(load factor for self-straining companion load, EM 1110-2-2104, 3-3f))*

$(\gamma_c)_W := 0.5$ *(load factor for wind companion load, EM 1110-2-2104, 3-3f))*

$(\gamma_c)_{Ld} := 1.0$ *(load factor for dynamic, companion load, EM 1110-2-2104, 3-3f))*

Standard Material Strengths

$f_y := 60\ ksi$ *(assumed rebar strength for all reinforcing steel)*

$f'_c := 4.5\ ksi$ *(assumed concrete compressive strength for all hydraulic structures)*

$\lambda_c := 1.0$ *(concrete modification factor)*

$\varepsilon_{ty} := 0.002$ *(yield strain in reinforcing steel)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{concrete} := 150\ pcf$ *(unit weight of concrete)*

$\gamma_{steel} := 490\ pcf$ *(unit weight of water)*

Site Soil Conditions

$\gamma_{soil} := 115\ pcf$ *(unit weight of soil on site)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 52.6\ pcf$ *(buoyant unit weight of soil)*

$\sigma'_n := 0\ psf$ *(effective stress normal to the wall)*

$(\delta'/\phi')_{drained} := 0.5$ *(interface friction angle ratio, EM 1110-2-2502 Table 6.2)*

Drained Soil Conditions

$\phi_{frict\_soil\_drained} := 26\ deg$ *(internal fiction angle in the drained condition)*

$S_{A\_undrained} := 100\ psf$ *(soil undrained shear strength)*

$C_{A\_drained} := 100\ psf$ *(cohesion for drained conditions)*

$k_{o\_drained} := 1 - \sin\left(\phi_{frict\_soil\_drained}\right) = 0.562$ *(at-rest soil pressure coefficient for the drained condition)*

$(s_u)_{drained} := 0\ psi$ *(drained cohesion strength of soils, EM 1110-2-2502)*

Undrained Soil Conditions

$\phi_{frict\_soil\_undrained} := 30\ deg$ *(internal fiction angle in the undrained condition)*

$S_{A\_undrained} := 0\ psf$

$C_{A\_undrained} := 750\ psf$ *(cohesion for undrained conditions)*

$k_{o\_undrained} := 1 - \sin\left(\phi_{frict\_soil\_undrained}\right) = 0.5$ *(at-rest soil pressure coefficient for the undrained condition)*

$(s_u)_{undrained} := 250\ psf$ *(undrained cohesion strength of soils, EM 1110-2-2502 Table 5.2)*

$(c_a/s_u)_{undrained} := 0.25$ *(undrained adhesion strength modification factor, EM 1110-2-2502, 6.7.5.4.2)*

$u_{soil} := 0\ psi$ *(assumed pore pressure in undrained soil conditions)*

Loading Conditions from EM 1110-2-2104

$Structure_{critical} :=$ Yes *(structure categorized as Critical or Normal)*

Serviceability Loading Conditions

$(f_{s\_U})_{M\_V} := 25\ ksi$ *(reinforcement stress, EM 2104 3-4a(1), Table 3-3)*

$(f_{s\_U})_{T} := 20\ ksi$ *(reinforcement stress, EM 2104 3-4a(1), Table 3-3)*

$(f_{s\_N})_{M\_V} := 35\ ksi$ *(reinforcement stress, EM 2104 3-4a(1), Table 3-3)*

$(f_{s\_N})_{T} := 27.5\ ksi$ *(reinforcement stress, EM 2104 3-4a(1), Table 3-3)*

Load factors for Single-Load Cases in Serviceability Design

$(\Phi_U)_{M\_V\_SLC} := 2.2$ *(single load case load factor, EM 2104 Table 3-4)*

$(\Phi_U)_{T\_SLC} := 2.8$ *(single load case load factor, EM 2104 Table 3-4)*

$(\Phi_N)_{M\_V\_SLC} := 1.6$ *(single load case load factor, EM 2104 Table 3-4)*

$(\Phi_N)_{T\_SLC} := 2.0$ *(single load case load factor, EM 2104 Table 3-4)*

Serviceability Deflection Limits

$(\rho_b)_S := 0.25$ *(flexural maximum reinforcement ratio, EM 2104 3-4b(2), limited to 25% of the balanced reinforcement ratio)*

$(\rho_b)_{T\_Max} := 0.50$ *(maximum reinforcement)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

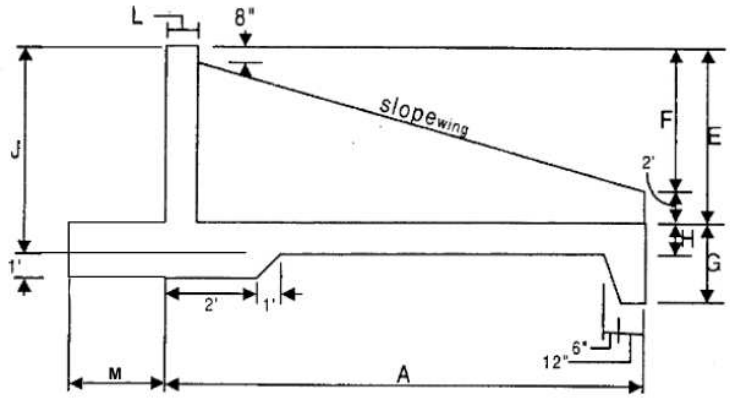
Factors of Safety

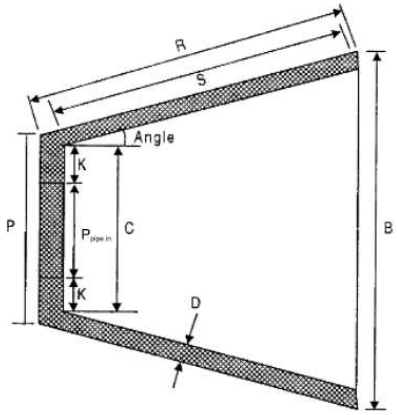
$FS := 1.0$ *(safety factor for analysis, EM1110-2-2502, Section 6.7.6.4.3)*

STRUCTURE LOCATIONS

Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Pipe Size in	Pipe Wall in	Top of Outlet Wall Ft	Wall Angle °	10 Yr Flood Elevation	100 Yr Flood Elevation
ST-02A	LB	2+75	923.17	922.15	24	4	926.67	-	933.43	937.15
ST-03A	LB	5+80	923.90	922.00	24	4	927.40	-	933.00	936.50
ST-19A	LB	36+60	897.06	894.50	48	8	902.56	-	913.50	918.75
ST-22A	RB	39+25	898.68	895.00	24	4	902.18	-	913.50	918.75
ST-26A	RB	41+80	899.11	896.20	24	4	902.61	-	913.00	918.00
ST-43A	RB	62+50	893.12	888.20	24	4	896.62	-	907.50	912.00
ST-45A	RB	64+70	896.67	891.00	24	4	900.17	-	905.00	910.00
ST-45B	LB	64+85	898.87	890.50	24	4	902.37	-	905.00	909.75

STRUCTURE LAYOUT





Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Pipe Size in	Pipe Wall in	Top of Outlet Wall Ft	A In	B	C	D	E	F	G	H	J	K	L	M	P	P pipe in	R	S
ST-02A	LB	2+75.0	923.17	922.15	24	4	926.67	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54
ST-03A	LB	5+80.0	923.9	922	24	4	927.4	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54
ST-19A	LB	36+60.0	897.06	894.5	48	8	902.56	96	124	84	8	78	54	24	12	90	18	12	24	96	48	99	91
ST-22A	RB	39+25.0	898.68	895	24	4	902.18	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54
ST-26A	RB	41+80.0	899.11	896.2	24	4	902.61	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54
ST-43A	RB	62+50.0	893.12	888.2	24	4	896.62	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54
ST-45A	RB	64+70.0	896.67	891	24	4	900.17	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54
ST-45B	LB	64+85.0	898.87	890.5	24	4	902.37	60	105	60	8	60	36	24	12	72	18	12	24	73	24	62	54

Volume of Structure Components

$(V_{strt})_{tail} := \overrightarrow{M_{outfall} \cdot (H_{outfall} + 1\ ft) \cdot P_{outfall}}$ *(area of heel of outfall wall)*

$(V_{strt})_{thk.bot} := \overrightarrow{((0.5 \cdot (2\ ft + 3\ ft) \cdot 1\ ft) \cdot P_{outfall})} \leftarrow$
$+ \overrightarrow{((0.5 \cdot (1\ ft + 18\ in) \cdot (G_{outfall} - H_{outfall})) \cdot B_{outfall})}$ *(area of thickened sections under slab; at the connection to the stem and the key in front)*

$(V_{strt})_{slab} := \overrightarrow{(\overrightarrow{A_{outfall} \cdot H_{outfall}}) \cdot (0.5 \cdot (B_{outfall} + P_{outfall}))}$ *(area of slabs)*

$(V_{strt})_{stem} := \overrightarrow{L_{outfall} \cdot E_{outfall} \cdot P_{outfall}}$ *(area of stems)*

$(V_{strt})_{w.w} := \overrightarrow{2 \cdot \left(\begin{array}{l} \overrightarrow{(0.5 \cdot (F_{outfall} - 8\ in) \cdot (A_{outfall} - L_{outfall}))} \leftarrow \\ + \overrightarrow{(2\ ft \cdot (A_{outfall} - L_{outfall}))} \end{array} \right) \cdot D_{outfall}}$ *(area of the wing walls for outfall structures)*

Center of Gravity for Structural Components

$(x_{strt})_{tail} := \overrightarrow{A_{outfall} + (0.5 \cdot M_{outfall})}$ *(distance to centroid from toe to tail of the structures)*

$(x_{strt})_{thk.bot.back} := A_{outfall} - \left(\frac{(3\ ft + 4\ ft) \cdot 1\ ft}{3 \cdot 5\ ft}\right)$ *(distance to centroid of thickened connection from toe)*

$(x_{strt})_{thk.bot.toe} := 0.5 \cdot (Toe_{top})$ *(distance to centroid of key at toe from toe)*

$(x_{strt})_{thk.bot} := 0.5 \cdot \left((x_{strt})_{thk.bot.back} + (x_{strt})_{thk.bot.toe}\right)$ *(distance from combined centroid of thickened sections of slab from toe)*

$(x_{strt})_{slab} :=$ *(distance from centroid of slab from toe)*

```
for k ∈ ORIGIN .. 19
    if P_outfall_k + B_outfall_k = 0
        (x_strt)_slab_k ← 0
    else
        (x_strt)_slab_k ← ((B_outfall_k + 2 · P_outfall_k) · A_outfall_k)  (vector)
                          / (3 · (P_outfall_k + B_outfall_k))
return (x_strt)_slab
```

$(x_{strt})_{stem} := A_{outfall} - (0.5 \cdot L_{outfall})$ *(distance from centroid of stem from toe)*

$(x_{strt})_{w.top} := \frac{2}{3} \cdot (A_{outfall} - L_{outfall})$ *(centroid location of triangular section of wingwall, from toe)*

$(x_{strt})_{w.bot} := \frac{1}{2} \cdot (A_{outfall} - L_{outfall})$ *(centroid location of rectangular section of wingwall, from toe)*

$(A_{strt})_{w.top} := \overrightarrow{0.5 \cdot (A_{outfall} - L_{outfall}) \cdot (F_{outfall})}$ *(area of triangular section of wingwall, from toe)*

$(A_{strt})_{w.bot} := (A_{outfall} - L_{outfall}) \cdot (2\ ft)$ *(area of rectangular section of wingwall, from toe)*

$(x_{strt})_{w.w} :=$ *(location of the centroid for the wingwalls from the toe)*

```
for k ∈ ORIGIN .. 19
    if (A_strt)_w.top_k + (A_strt)_w.bot_k = 0
        (x_strt)_w.w_k ← 0 ft
    else
        (x_strt)_w.w_k ← ((x_strt)_w.top_k · (A_strt)_w.top_k)  (vector)
                         + ((x_strt)_w.bot_k · (A_strt)_w.bot_k)  (vector)
                         / (A_strt)_w.top_k + (A_strt)_w.bot_k
return (x_strt)_w.w
```

Weight of Structural Components

$(W_{strt})_{tail} := \gamma_{concrete} \cdot (V_{strt})_{tail}$ *(weight of the tail of the structures)*

$(W_{strt})_{thk.bot} := \gamma_{concrete} \cdot (V_{strt})_{thk.bot}$ *(weight of the thickened slab portions)*

$(W_{strt})_{slab} := \gamma_{concrete} \cdot (V_{strt})_{slab}$ *(weight of the slab portions)*

$(W_{strt})_{stem} := \gamma_{concrete} \cdot (V_{strt})_{stem}$ *(weight of the stem portions)*

$(W_{strt})_{w.w} := \gamma_{concrete} \cdot (V_{strt})_{w.w}$ *(weight of the wingwalls portions)*

Summary Table of Components

Structure Name	Location	River Station	Top of Outlet Wall Ft	Vol $(V_{strt})_{tail}$	Vol $(V_{strt})_{thk}$	Vol $(V_{strt})_{slab}$	Vol $(V_{strt})_{stem}$	Vol $(V_{strt})_{w.w}$	Dist $(x_{strt})_{tail}$	Dist $(x_{strt})_{thk}$	Dist $(x_{strt})_{slab}$	Dist $(x_{strt})_{stem}$	Dist $(x_{strt})_{w.w}$	Weight $(W_{strt})_{tail}$	Weight $(W_{strt})_{thk}$	Weight $(W_{strt})_{slab}$	Weight $(W_{strt})_{stem}$
ST-02A	LB	275	926.67	24.33	26.15	37.08	30.42	16.89	6.00	2.64	2.35	4.50	2.29	3.65	3.92	5.56	4.56
ST-03A	LB	580	927.4	24.33	26.15	37.08	30.42	16.89	6.00	2.64	2.35	4.50	2.29	3.65	3.92	5.56	4.56
ST-19A	LB	3660	902.56	32.00	32.92	73.33	52.00	36.56	9.00	4.14	3.83	7.50	4.12	4.80	4.94	11.00	7.80
ST-22A	RB	3925	902.18	24.33	26.15	37.08	30.42	16.89	6.00	2.64	2.35	4.50	2.29	3.65	3.92	5.56	4.56
ST-26A	RB	4180	902.61	24.33	26.15	37.08	30.42	16.89	6.00	2.64	2.35	4.50	2.29	3.65	3.92	5.56	4.56
ST-43A	RB	6250	896.62	24.33	26.15	37.08	30.42	16.89	6.00	2.64	2.35	4.50	2.29	3.65	3.92	5.56	4.56
ST-45A	RB	6470	900.17	24.33	26.15	37.08	30.42	16.89	6.00	2.64	2.35	4.50	2.29	3.65	3.92	5.56	4.56

Total Weight of Structure

$$W_{strt} := (W_{strt})_{tail} + (W_{strt})_{thk.bot} + (W_{strt})_{slab} + (W_{strt})_{stem} + (W_{strt})_{w.w}$$ *(total weight of the outfall structure)*

$$D := W_{strt}$$

Centroid Location of Structures

$$(X_{strt})_{bar} := \left| \begin{array}{l} \text{for } k \in \mathbf{ORIGIN} .. 19 \\ \quad \text{if } (V_{strt})_{tail_k} + (V_{strt})_{thk.bot_k} + (V_{strt})_{slab_k} + (V_{strt})_{stem_k} + (V_{strt})_{w.w_k} = 0 \\ \quad\quad (X_{strt})_{bar_k} \leftarrow 0 \\ \quad \text{else} \\ \quad\quad (X_{strt})_{bar_k} \leftarrow \dfrac{\left(\overrightarrow{(V_{strt})_{tail_k} \cdot (x_{strt})_{tail_k}}\right) + \left(\overrightarrow{(V_{strt})_{thk.bot_k} \cdot (x_{strt})_{thk.bot_k}}\right) + \left(\overrightarrow{(V_{strt})_{slab_k} \cdot (x_{strt})_{slab_k}}\right) + \left(\overrightarrow{(V_{strt})_{stem_k} \cdot (x_{strt})_{stem_k}}\right) + \left(\overrightarrow{(V_{strt})_{w.w_k} \cdot (x_{strt})_{w.w_k}}\right)}{(V_{strt})_{tail_k} + (V_{strt})_{thk.bot_k} + (V_{strt})_{slab_k} + (V_{strt})_{stem_k} + (V_{strt})_{w.w_k}} \\ \text{return } (X_{strt})_{bar} \end{array} \right|$$

Structure Name	Location	River Station	Outlet Body Total Weight [Kips]	Outlet Body Centroid Location [Ft]
ST-02A	LB	275	20.23	3.54
ST-03A	LB	580	20.23	3.54
ST-19A	LB	3660	34.02	5.49
ST-22A	RB	3925	20.23	3.54
ST-26A	RB	4180	20.23	3.54
ST-43A	RB	6250	20.23	3.54
ST-45A	RB	6470	20.23	3.54

SOIL LOADING

Landside Layout - Resisting Side							
Structure Name	Location	River Station	Top of Wall Ft	Elevation at Resisting Side Ft	Distance Landside Offset Ft	Calc Slope	β Slope Behind Headwall [Deg]
ST-02A	LB	275	926.67	934.5	25	0.313	0.2
ST-03A	LB	580	927.4	932	15	0.307	0.2
ST-19A	LB	3660	902.56	909	8	0.805	0.5
ST-22A	RB	3925	902.18	911	25	0.353	0.2
ST-26A	RB	4180	902.61	909	20	0.319	0.2
ST-43A	RB	6250	896.62	905.8	22	0.417	0.2
ST-45A	RB	6470	900.17	908	20	0.392	0.2
ST-45B	LB	6485	902.37	908	20	0.282	0.2

Riverine Layout - Driving Side							
Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Ft	Distance to CL of Creek Ft	Calc Slope	β Slope From Outfall
ST-02A	LB	275	923.17	922.15	5	-0.20	-0.12
ST-03A	LB	580	923.9	922	6	-0.32	-0.18
ST-19A	LB	3660	897.06	894.5	10	-0.26	-0.15
ST-22A	RB	3925	898.68	895	7	-0.53	-0.30
ST-26A	RB	4180	899.11	896.2	4	-0.73	-0.42
ST-43A	RB	6250	893.12	888.2	5	-0.98	-0.56
ST-45A	RB	6470	896.67	891	12	-0.47	-0.27

$\beta_{RS} := \beta_{RS}$ *(angle of slope behind headwall, resisting side)*

$(K_{o\beta})_{RS} := \left(1 - \sin\left(\phi_{frict\_soil\_drained}\right)\right) \cdot \left(1 + \sin\left(\beta_{RS}\right)\right)$ *(at-rest soil pressure coeff. with a sloping backfill, EM 1110-2-2502 Eq 6.9)*

$(K_p)_{RS} := \frac{\left(1 + \sin\left(\phi_{frict\_soil\_drained}\right)\right)}{\left(1 - \sin\left(\phi_{frict\_soil\_drained}\right)\right)} = 2.6$ *(passive soil pressure coeff. with a sloping backfill, EM 1110-2-2502 Eq 6.16)*

$(K_A)_{RS} := \frac{\left(1 - \sin\left(\phi_{frict\_soil\_drained}\right)\right)}{\left(1 + \sin\left(\phi_{frict\_soil\_drained}\right)\right)} = 0.4$ *(active soil pressure coeff. with a sloping backfill, EM 1110-2-2502 Eq 6.15)*

Vertical Earth Pressures

$(h_{soil})_1 := ToW \cdot ft \dots - (FLElev \cdot ft + Pipe\_size \cdot in + Pipe\_Wt \cdot in)$

(height of soil behind stem; taken as the difference in elevation between the top of the pipe and the top of the stem wall)

$(h_{soil})_2 := (h_{soil})_1 + \overrightarrow{M_{outfall} \cdot \sin(\beta_{RS})}$

(height of soil behind end of pipe and t-foundation, assumed uniform slopping back to end of pipe section)

$(V_{soil})_{RS} := \overrightarrow{\left(0.5 \cdot \left((h_{soil})_2 + (h_{soil})_1\right) \cdot \overrightarrow{M_{outfall}}\right) \cdot P_{outfall}}$

(total volume of soil behind wall affecting vertical earth loading)

$(x_{soil})_{RS} :=$
for $i \in$ ORIGIN .. 19
  if $(h_{soil})_{2_i} + (h_{soil})_{1_i} = 0$
    $(x_{soil})_{RS_i} \leftarrow 0\ ft$
  else
    $(x_{soil})_{RS_i} \leftarrow A_{outfall_i} + \left(\frac{\overrightarrow{\left((h_{soil})_{1_i} + 2 \cdot (h_{soil})_{2_i}\right) \cdot M_{outfall_i}}}{3 \cdot \left((h_{soil})_{2_i} + (h_{soil})_{1_i}\right)}\right)$
return $(x_{soil})_{RS}$

(distance to centroid of the vertical earth pressure from the front face of the toe of the wall)

$EV := \gamma_{soil} \cdot (V_{soil})_{RS}$

(vertical earth loading applied to the outfall structure from landside of the structure, above the pipe.)

Summary of Vertical Soil Loads

Structure Name	Location	River Station	h_1 height behind wall Ft	h_2 height behind slope Ft	Vol of Soil Ft^3	Centroid Location Ft	EV kip
ST-02A	LB	2+75	1.17	1.17	14.23	6.00	1.64
ST-03A	LB	5+80	1.17	1.17	14.23	6.00	1.64
ST-19A	LB	36+60	0.83	0.85	13.46	9.00	1.55
ST-22A	RB	39+25	1.17	1.17	14.24	6.00	1.64
ST-26A	RB	41+80	1.17	1.17	14.23	6.00	1.64
ST-43A	RB	62+50	1.17	1.18	14.25	6.00	1.64
ST-45A	RB	64+70	1.17	1.17	14.24	6.00	1.64
ST-45B	LB	64+85	1.17	1.17	14.23	6.00	1.64
0	0	-	0.00	0.00	0.00	0.00	0.00

Hydrostatic Loading and Groundwater

$(Elev_f)_{10} := (Elev_f)_{10} \cdot ft$

(10 year flood elevation, from H&H modeling)

$(Elev_f)_{100} := (Elev_f)_{100} \cdot ft$

(100 year flood elevation, from H&H modeling)

$\beta_{DS} := \beta_{DS}$

(angle of slope behind headwall, resisting side)

$(K_{o\beta})_{DS} := \left(1 - \sin\left(\phi_{frict\_soil\_drained}\right)\right) \cdot \left(1 + \sin\left(\beta_{DS}\right)\right)$

(at-rest soil pressure coeff. with a sloping channel, EM 1110-2-2502 Eq 6.9)

$(K_p)_{DS} := \frac{\left(1 + \sin\left(\phi_{frict\_soil\_drained}\right)\right)}{\left(1 - \sin\left(\phi_{frict\_soil\_drained}\right)\right)} = 2.6$

(passive soil pressure coeff. with a sloping channel, EM 1110-2-2502 Eq 6.16)

$(K_A)_{DS} := \frac{\left(1 - \sin\left(\phi_{frict\_soil\_drained}\right)\right)}{\left(1 + \sin\left(\phi_{frict\_soil\_drained}\right)\right)} = 0.4$

(active soil pressure coeff. with a sloping channel, EM 1110-2-2502 Eq 6.15)

Crack Gap Calculations

$(d_c)_Q := 2 \cdot \frac{(s_u)_{undrained}}{\gamma'_{soil}} = 9.506\ ft$ *(crack depth for Q-case, undrained soil conditions, EM 1110-2-2502 Eq 6.1)*

$(d_c)_S := 2 \cdot \frac{C_{A\_drained}}{\gamma'_{soil} \cdot \sqrt{(K_A)_{RS}}} = 6.085\ ft$ *(crack depth for S-case, undrained soil conditions, EM 1110-2-2502 Eq 6.2)*

Riverside Hydrostatic Loading (Driving Side)

$(Hs_{10})_{ToW.DS} := \gamma_{water} \cdot \left((Elev_f)_{10} - ToW \cdot ft\right) = \begin{bmatrix} 2.9 \\ 2.4 \\ 4.7 \\ 4.9 \\ \vdots \end{bmatrix} psi$ *(hydrostatic pressure at the top of the outfall wall in the 10 year flood on the river side)*

$(Hs_{10})_{slab.DS} := \gamma_{water} \cdot \left((Elev_f)_{10} - \left(ToW \cdot ft - E_{outfall}\right)\right) = \begin{bmatrix} 5.1 \\ 4.6 \\ 7.6 \\ 7.1 \\ \vdots \end{bmatrix} psi$ *(hydrostatic pressure at the top face of the slab of outfall structure in the 10 year flood on the rivers side)*

$(Hs_{100})_{ToW.DS} := \gamma_{water} \cdot \left((Elev_f)_{100} - ToW \cdot ft\right) = \begin{bmatrix} 4.5 \\ 3.9 \\ 7 \\ 7.2 \\ \vdots \end{bmatrix} psi$ *(hydrostatic pressure at the top of the outfall wall in the 100 year flood on the river side)*

$(Hs_{100})_{slab.DS} := \gamma_{water} \cdot \left((Elev_f)_{100} - \left(ToW \cdot ft - E_{outfall}\right)\right) = \begin{bmatrix} 6.7 \\ 6.1 \\ 9.8 \\ 9.3 \\ \vdots \end{bmatrix} psi$ *(hydrostatic pressure at the top face of the slab of outfall structure in the 100 year flood on the rivers side)*

Resultant Hydrostatic Pressure

$(Hs_{10})_{press.DS} := 0.5 \cdot \overrightarrow{\left((Hs_{10})_{slab.DS} - (Hs_{10})_{ToW.DS}\right) \cdot E_{outfall}}$ *(hydrostatic loading to front face of outfall at 10 year event elevations)*

$(Hs_{100})_{press.DS} := 0.5 \cdot \overrightarrow{\left((Hs_{100})_{slab.DS} - (Hs_{100})_{ToW.DS}\right) \cdot E_{outfall}}$ *(hydrostatic loading to front face of outfall at 100 year event elevations)*

Total Hydrostatic Load to front face of Structure

$(Hs_{10})_{DS} := \overrightarrow{(Hs_{10})_{press.DS} \cdot C_{outfall}}$ *(hydrostatic resultant to structure, 10 year event elevation)*

$(Hs_{100})_{DS} := \overrightarrow{(Hs_{100})_{press.DS} \cdot C_{outfall}}$ *(hydrostatic resultant to structure, 100 year event elevation)*

Resultant Location on Front Face of Structure

$(x_{10})_{Hs.DS} := \left(\frac{E_{outfall}}{2}\right)$ *(resultant location, measured from top to slab to hydrostatic loading, 10 year event)*

$(x_{100})_{Hs.DS} := \left(\frac{E_{outfall}}{2}\right)$ *(resultant location, measured from top to slab to hydrostatic loading, 100 year event)*

(H

Stem Loading Combinations

Riverside Hydrostatic Loading Conditions

Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Top of Outlet Wall Ft	10 Yr Flood Elevation	100 Yr Flood Elevation	10 Yr Hs kips	100 Yr Hs kips	X10 feet	X100 feet
ST-02A	LB	2+75	923.17	922.15	926.67	933.43	937.15	3.90	3.90	2.50	2.50
ST-03A	LB	5+80	923.90	922.00	927.40	933.00	936.50	3.90	3.90	2.50	2.50
ST-19A	LB	36+60	897.06	894.50	902.56	913.50	918.75	9.23	9.23	3.25	3.25
ST-22A	RB	39+25	898.68	895.00	902.18	913.50	918.75	3.90	3.90	2.50	2.50
ST-26A	RB	41+80	899.11	896.20	902.61	913.00	918.00	3.90	3.90	2.50	2.50
ST-43A	RB	62+50	893.12	888.20	896.62	907.50	912.00	3.90	3.90	2.50	2.50
ST-45A	RB	64+70	896.67	891.00	900.17	905.00	910.00	3.90	3.90	2.50	2.50
ST-45B	LB	64+85	898.87	890.50	902.37	905.00	909.75	3.90	3.90	2.50	2.50
0	0	0+00	-	-	-	-	-	-	-	-	-

$(Elev_f$

Landside Hydrostatic Loading (Resisting Side)

Note: the hydrostatic loading for the back face of the stem wall is isolated to the hydrostatic pressure in the soils above the pipe connection to the back face, as well as the assumed hydrostatic pressure inside of the pipe connected to the outlet structure stem. Hydrostatic pressure assumed to be the ground surface elevation as this conservatively assumes that the ground behind the wall is saturated; conservatively the higher of the elevations is taken between the flood event elevation or the ground surface elevation at the floodwall.

10 Year Event Elevation

$$(Elev_f)_{10.RS} := \left\| \begin{array}{l} \text{for } i \in \text{ORIGIN} .. 19 \\ \quad \left\| (Elev_f)_{10.RS_i} \leftarrow \max\left((Elev_f)_{10_i}, (Elev_{Land})_{Wall_i}\right) \right. \\ \text{return } (Elev_f)_{10.RS} \end{array} \right.$$

(maximum elevation for the water surface elevation on the resisting side, take as higher of the grand surface elevation or the water elevation for the 10 year flood)

100 Year Event Elevation

$$(Elev_f)_{100.RS} := \left\| \begin{array}{l} \text{for } i \in \text{ORIGIN} .. 19 \\ \quad \left\| (Elev_f)_{100.RS_i} \leftarrow \max\left((Elev_f)_{100_i}, (Elev_{Land})_{Wall_i}\right) \right. \\ \text{return } (Elev_f)_{100.RS} \end{array} \right.$$

(maximum elevation for the water surface elevation on the resisting side, take as higher of the grand surface elevation or the water elevation for the 100 year flood)

Riverside Hydrostatic Loading (Resisting Side)

$$(Hs_{10})_{ToW.RS} := \gamma_{water} \cdot \left((Elev_f)_{10.RS} - ToW \cdot ft\right) = \begin{bmatrix} 3.4 \\ 2.4 \\ 4.7 \\ 4.9 \\ \vdots \end{bmatrix} psi$$

(hydrostatic pressure at the top of the outfall wall in the 10 year flood on the river side)

$$(Hs_{10})_{slab.RS} := \gamma_{water} \cdot \left((Elev_f)_{10.RS} - \left(ToW \cdot ft - E_{outfall}\right)\right) = \begin{bmatrix} 5.6 \\ 4.6 \\ 7.6 \\ 7.1 \\ \vdots \end{bmatrix} psi$$

(hydrostatic pressure at the top face of the slab of outfall structure in the 10 year flood on the rivers side)

$(Hs_{100})_{ToW.RS} := \gamma_{water} \cdot \langle (Elev_f)_{100.RS} - ToW \cdot ft \rangle = \begin{bmatrix} 4.5 \\ 3.9 \\ 7 \\ 7.2 \\ \vdots \end{bmatrix} psi$ *(hydrostatic pressure at the top of the outfall wall in the 100 year flood on the river side)*

$(Hs_{100})_{slab.RS} := \gamma_{water} \cdot \langle (Elev_f)_{100.RS} - \langle ToW \cdot ft - E_{outfall} \rangle \rangle = \begin{bmatrix} 6.7 \\ 6.1 \\ 9.8 \\ 9.3 \\ \vdots \end{bmatrix} psi$ *(hydrostatic pressure at the top face of the slab of outfall structure in the 100 year flood on the rivers side)*

Resultant Hydrostatic Pressure

$(Hs_{10})_{press.RS} := 0.5 \cdot \overrightarrow{\langle (Hs_{10})_{slab.RS} + (Hs_{10})_{ToW.RS} \rangle \cdot E_{outfall}}$ *(hydrostatic loading to front face of outfall at 10 year event elevations)*

$(Hs_{100})_{press.RS} := 0.5 \cdot \overrightarrow{\langle (Hs_{100})_{slab.RS} + (Hs_{100})_{ToW.RS} \rangle \cdot E_{outfall}}$ *(hydrostatic loading to front face of outfall at 100 year event elevations)*

Total Hydrostatic Load to front face of Structure

$(Hs_{10})_{RS} := \overrightarrow{(Hs_{10})_{press.RS} \cdot C_{outfall}}$ *(hydrostatic resultant to structure, 10 year event elevation)*

$(Hs_{100})_{RS} := \overrightarrow{(Hs_{100})_{press.RS} \cdot C_{outfall}}$ *(hydrostatic resultant to structure, 10 year event elevation)*

Resultant Location on Front Face of Structure

$(x_{10})_{Hs.RS} := \left(\frac{E_{outfall}}{2} \right)$ *(resultant location, measured from top to slab to hydrostatic loading, 10 year event)*

$(x_{100})_{Hs.RS} := \left(\frac{E_{outfall}}{2} \right)$ *(resultant location, measured from top to slab to hydrostatic loading, 100 year event)*

Landside Hydrostatic Loading Conditions

Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Top of Outlet Wall Ft	10 Yr Flood Elevation	100 Yr Flood Elevation	10 Yr Hs kips	100 Yr Hs kips	X10 feet	X100 feet
ST-02A	LB	275	923.17	922.15	926.67	934.5	937.15	16.11	20.249	2.5	2.5
ST-03A	LB	580	923.9	922	927.4	933	936.5	12.64	18.096	2.5	2.5
ST-19A	LB	3660	897.06	894.5	902.56	913.5	918.75	40.29	55.194	3.25	3.25
ST-22A	RB	3925	898.68	895	902.18	913.5	918.75	21.56	29.749	2.5	2.5
ST-26A	RB	4180	899.11	896.2	902.61	913	918	20.11	27.908	2.5	2.5
ST-43A	RB	6250	893.12	888.2	896.62	907.5	912	20.87	27.893	2.5	2.5
ST-45A	RB	6470	896.67	891	900.17	908	910	16.11	19.235	2.5	2.5
ST-45B	LB	6485	898.87	890.5	902.37	908	909.75	12.68	15.413	2.5	2.5
0	0	0	0	0	0	0	0	0	0	0	0

Lateral Earth Pressures

Note: Lateral earth pressures are calculated assuming that the soil behind the outfall structure will be saturated. Lateral earth pressures are accounting for the soil not present in the area of the pipe, but neglects any bedding material and conservatively assumes lateral earth pressure around the pipe, applied to the

Shear Strength

Drained Conditions

$(\tau_f)_{drained} := \sigma'_n \cdot \tan\left((\delta'/\phi')_{drained} \cdot \phi_{frict\_soil\_drained}\right) = 0\ psi$ *(shear stress on wall at failure, EM 1110-2-2502 Eq 6.6)*

Undrained Conditions

$(\tau_f)_{undrained} := (c_a/s_u)_{undrained} \cdot (s_u)_{undrained} = 0.4\ psi$ *(limiting shear stress on the wall at failure, EM 1110-2-2502, EQ 6.7)*

Active Earth Pressures

$\phi'_d := \phi_{frict\_soil\_drained} = 26\ deg$ *(developed friction angle, drained conditions, EM 1110-2-2502, Eq 6.19)*

$(\delta'/\phi')_{drained} = 0.5$ *(interface friction angle ratio, EM 1110-2-2502 Table 6.2)*

$\delta'_d := (\delta'/\phi')_{drained} \cdot \phi'_d = 13\ deg$ *(developed interface friction coefficient, EM 1110-2-2502, Eq 6.19)*

$$(K_A)_{coeff} := \frac{\left(\cos\left(\phi'_d - \Theta\right)\right)^2}{\left(\cos\left(\Theta\right)^2 \cdot \cos\left(\delta'_d + \Theta\right)\right) \cdot \left(1 + \sqrt{\frac{\sin\left(\phi'_d + \delta'_d\right) \cdot \sin\left(\phi'_d - \beta_{RS}\right)}{\cos\left(\delta'_d + \Theta\right) \cdot \cos\left(\Theta - \beta_{RS}\right)}}\right)^2}$$

(active earth pressure accounting for the developed angle of wall friction, wall angle and sloping soil, EM 1110-2-2502, Eq 6.19)

$(K_A)_{DS} := (K_A)_{coeff} \cdot (K_A)_{DS}$ *(active earth pressure accounting for slope angles, EM 1110-2-2502, Eq 6.19)*

Passive Earth Pressures

$$(K_P)_{coeff} := \frac{\left(\cos\left(\phi'_d + \Theta\right)\right)^2}{\left(\cos\left(\Theta\right)^2 \cdot \cos\left(\delta'_d - \Theta\right)\right) \cdot \left(1 - \sqrt{\frac{\sin\left(\phi'_d + \delta'_d\right) \cdot \sin\left(\phi'_d + \beta_{RS}\right)}{\cos\left(\delta'_d - \Theta\right) \cdot \cos\left(\Theta - \beta_{RS}\right)}}\right)^2}$$

(passive earth pressure accounting for the developed angle of wall friction, wall angle and sloping soil, EM 1110-2-2502, Eq 6.20)

Area of Loading for Horizontal Earth Pressure

$A_{Stem} := \overrightarrow{\left(P_{outfall} \cdot E_{outfall}\right)} - \left(0.25 \cdot \overrightarrow{\pi \cdot \left((P_{pipe.in})_{outfall}\right)^2}\right)$ *(cross sectional area of the stem wall minus the pipe)*

$A_{Key} := \overrightarrow{\left(G_{outfall} \cdot B_{outfall}\right)}$ *(cross sectional area of the key, or front face of slab)*

Lateral Earth Pressure Locations

Note: its assumed that the pipe comes out of the outfall stem at mid-height of the stem wall and thus the centroid location of the loading is half of the wall height. Conservatively, the lateral earth pressure is assumed to be at the 1/3 height location

$(y_{bar})_{Stem} := \frac{E_{outfall}}{3}$ *(centroid location of the horizontal loading to the stem wall)*

$$(y_{bar})_{Key} := \frac{-G_{outfall}}{2}$$

(centroid location of the horizontal loading to the key, from the top face of the slab)

Lateral Earth Pressures

Undrained Quick (Q-case)

$$(p_{a.ud})_{stem} := (\gamma_{soil} \cdot E_{outfall}) + (\gamma_{water} \cdot E_{outfall}) - \left(2 \cdot (s_u)_{undrained} \cdot \sqrt{1 + (c_a/s_u)_{undrained}}\right)$$

(undrained active pressure behind stem wall, EM 1110-2-2502, Eq 6.22)

$$(p_{a.ud})_{key} := (\gamma_{soil} \cdot G_{outfall}) + (\gamma_{water} \cdot ((Elev_f)_{100.RS} - ToW \cdot ft + E_{outfall})) - \left(2 \cdot (s_u)_{undrained} \cdot \sqrt{1 + (c_a/s_u)_{undrained}}\right)$$

(undrained active pressure behind key wall, EM 1110-2-2502, Eq 6.22)

$$(p_{p.ud})_{stem} := (\gamma_{soil} \cdot E_{outfall}) + (\gamma_{water} \cdot ((Elev_f)_{100.RS} - ToW \cdot ft + E_{outfall})) + \left(2 \cdot (s_u)_{undrained} \cdot \sqrt{1 + (c_a/s_u)_{undrained}}\right)$$

(undrained passive pressure behind stem wall, EM 1110-2-2502, Eq 6.24)

$$(p_{p.ud})_{key} := (\gamma_{soil} \cdot G_{outfall}) + (\gamma_{water} \cdot ((Elev_f)_{100.RS} - ToW \cdot ft + E_{outfall})) + \left(2 \cdot (s_u)_{undrained} \cdot \sqrt{1 + (c_a/s_u)_{undrained}}\right)$$

(undrained passive pressure behind key wall, EM 1110-2-2502, Eq 6.24)

Drained Quick (S-case)

$$(p'_{a.d})_{stem} := \overrightarrow{(\gamma'_{soil} \cdot E_{outfall} \cdot (K_A)_{DS})}$$

(drained active pressure behind stem wall, EM 1110-2-2502, Eq 6.12, Note that second term is zero)

$$(p'_{a.d})_{key} := \overrightarrow{(\gamma'_{soil} \cdot G_{outfall} \cdot (K_A)_{DS})}$$

(drained active pressure behind key wall, EM 1110-2-2502, Eq 6.12, Note that second term is zero)

$$(p'_{p.d})_{stem} := \overrightarrow{(\gamma'_{soil} \cdot E_{outfall} \cdot (K_p)_{RS})}$$

(drained active pressure behind stem wall, EM 1110-2-2502, Eq 6.12, Note that second term is zero)

$$(p'_{p.d})_{key} := \overrightarrow{(\gamma'_{soil} \cdot G_{outfall} \cdot (K_p)_{RS})}$$

(drained active pressure behind key wall, EM 1110-2-2502, Eq 6.12, Note that second term is zero)

Undrained Conditions			Active Pressure				Passive Pressure			
Structure Name	Location	River Station	Stem Loading (psi)	Stem Loading Location (ft)	Key Loading (psi)	Key Loading Location (ft)	Stem Loading (psi)	Stem Loading Location (ft)	Key Loading (psi)	Key Loading Location (ft)
ST-02A	LB	275	2.28	1.67	4.42	(1.00)	14.58	1.67	12.19	(1.00)
ST-03A	LB	580	2.28	1.67	3.83	(1.00)	13.99	1.67	11.59	(1.00)
ST-19A	LB	3,660	4.13	2.17	7.55	(1.00)	18.91	2.17	15.31	(1.00)
ST-22A	RB	3,925	2.28	1.67	7.06	(1.00)	17.22	1.67	14.83	(1.00)
ST-26A	RB	4,180	2.28	1.67	6.55	(1.00)	16.71	1.67	14.31	(1.00)
ST-43A	RB	6,250	2.28	1.67	6.55	(1.00)	16.71	1.67	14.31	(1.00)
ST-45A	RB	6,470	2.28	1.67	4.14	(1.00)	14.30	1.67	11.91	(1.00)
ST-45B	LB	6,485	2.28	1.67	3.08	(1.00)	13.24	1.67	10.84	(1.00)
0	0	-	(3.88)	-	(3.88)	-	3.88	-	3.88	-

Drained Conditions			Active Pressure				Passive Pressure			
Structure Name	Location	River Station	Stem Loading (psi)	Stem Loading Location (ft)	Key Loading (psi)	Key Loading Location (ft)	Stem Loading (psi)	Stem Loading Location (ft)	Key Loading (psi)	Key Loading Location (ft)
ST-02A	LB	275	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)
ST-03A	LB	580	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)
ST-19A	LB	3,660	0.34	2.17	0.10	(1.00)	6.08	2.17	1.87	(1.00)
ST-22A	RB	3,925	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)
ST-26A	RB	4,180	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)
ST-43A	RB	6,250	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)
ST-45A	RB	6,470	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)
ST-45B	LB	6,485	0.26	1.67	0.10	(1.00)	4.68	1.67	1.87	(1.00)

Soil Surcharge Loading

$q_{floodwall} := 100\ psi$ *(maximum bearing pressure from floodwalls)*

Soil Surcharge Inputs					Floodwall							
Structure Name	Location	River Station	Top of Outlet Wall Ft	Height of Outfall Wall	Elevation of Bot of Fndtn	Lat Offset to Toe of Wall	Length of Fndtn	Lat Offset to Heel of Wall	β - Beg	β - End	β	α
ST-02A	LB	275	926.7	5.00	930.7	4.00	8.00	12.00	0.50	1.02	0.52	0.26
ST-03A	LB	580	927.4	5.00	930.0	2.00	6.00	8.00	0.33	0.93	0.61	0.30
ST-19A	LB	3,660	902.6	6.50	923.0	6.00	8.00	14.00	0.24	0.51	0.28	0.14
ST-22A	RB	3,925	902.2	5.00	921.0	5.00	8.00	13.00	0.22	0.53	0.31	0.15
ST-26A	RB	4,180	902.6	5.00	918.0	2.50	10.00	12.50	0.13	0.59	0.46	0.23
ST-43A	RB	6,250	896.6	5.00	915.0	1.25	10.00	11.25	0.06	0.48	0.42	0.21
ST-45A	RB	6,470	900.2	5.00	910.0	2.00	4.00	6.00	0.15	0.43	0.28	0.14
ST-45B	LB	6,485	902.4	5.00	906.0	2.25	6.00	8.25	0.31	0.87	0.56	0.28

$\beta_{outfall} := \beta_{heel} - \beta_{toe} = \begin{bmatrix} 0.52 \\ 0.61 \\ 0.28 \\ 0.31 \\ \vdots \end{bmatrix}$ *(beta angle for surcharge loading from foundation of floodwall beyond)*

$\alpha_{outfall} := \frac{\beta_{outfall}}{2} = \begin{bmatrix} 0.26 \\ 0.3 \\ 0.14 \\ 0.15 \\ \vdots \end{bmatrix}$ *(alpha angle for surcharge loading from foundation of floodwall beyond)*

$\sigma_H := \frac{q_{floodwall}}{\pi} \cdot \left(\beta_{outfall} - \overrightarrow{\sin\left(\beta_{outfall}\right) \cdot \cos\left(2 \cdot \alpha_{outfall}\right)}\right)$ *(lateral earth pressure from surcharge, EM 1110-2-2502, equation from Figure 6.12)*

Surcharge Loading added to Drained Condition:

Loading at Stem Wall

$$(EH_{a.ud})_{stem} := \overrightarrow{\left((p_{a.ud})_{stem} + \sigma_H\right) \cdot \left(\overrightarrow{C_{outfall} \cdot E_{outfall}}\right)} = \begin{bmatrix} 18.5 \\ 24.1 \\ 29.9 \\ \vdots \end{bmatrix} kip$$ *(lateral active earth pressure, undrained)*

$$(EH_{p.ud})_{stem} := \overrightarrow{\left((p_{p.ud})_{stem} + \sigma_H\right) \cdot \left(\overrightarrow{C_{outfall} \cdot E_{outfall}}\right)} = \begin{bmatrix} 62.8 \\ 66.3 \\ 126.8 \\ \vdots \end{bmatrix} kip$$ *(lateral passive earth pressure, undrained)*

$$(EH_{a.d})_{stem} := \overrightarrow{\left((p'_{a.d})_{stem} + \sigma_H\right) \cdot \left(\overrightarrow{C_{outfall} \cdot E_{outfall}}\right)} = \begin{bmatrix} 11.3 \\ 16.8 \\ 5.1 \\ \vdots \end{bmatrix} kip$$ *(lateral active earth pressure, drained)*

$$(EH_{p.d})_{stem} := \overrightarrow{\left((p'_{p.d})_{stem} + \sigma_H\right) \cdot \left(\overrightarrow{C_{outfall} \cdot E_{outfall}}\right)} = \begin{bmatrix} 27.2 \\ 32.7 \\ 42.7 \\ \vdots \end{bmatrix} kip$$ *(lateral passive earth pressure, drained)*

Loading at Key

$$(EH_{a.ud})_{key} := \overrightarrow{\left((p_{a.ud})_{key} + \sigma_H\right) \cdot \left(\overrightarrow{G_{outfall} \cdot B_{outfall}}\right)} = \begin{bmatrix} 18.4 \\ 20.8 \\ 23.8 \\ \vdots \end{bmatrix} kip$$ *(lateral active earth pressure, undrained)*

$$(EH_{p.ud})_{key} := \overrightarrow{\left((p_{p.ud})_{key} + \sigma_H\right) \cdot \left(\overrightarrow{G_{outfall} \cdot B_{outfall}}\right)} = \begin{bmatrix} 38 \\ 40.3 \\ 46.9 \\ \vdots \end{bmatrix} kip$$ *(lateral passive earth pressure, undrained)*

$$(EH_{a.d})_{key} := \overrightarrow{\left((p'_{a.d})_{key} + \sigma_H\right) \cdot \left(\overrightarrow{G_{outfall} \cdot B_{outfall}}\right)} = \begin{bmatrix} 7.5 \\ 11.4 \\ 1.6 \\ \vdots \end{bmatrix} kip$$ *(lateral active earth pressure, undrained)*

$$(EH_{p.d})_{key} := \overrightarrow{\left((p'_{p.d})_{key} + \sigma_H\right) \cdot \left(\overrightarrow{G_{outfall} \cdot B_{outfall}}\right)} = \begin{bmatrix} 12 \\ 15.9 \\ 6.9 \\ \vdots \end{bmatrix} kip$$ *(lateral passive earth pressure, undrained)*

Resultant location of Lateral Earth Pressures

$$(x_{EH})_{stem} := \frac{1}{3} \cdot E_{outfall}$$ *(resultant of lateral earth pressure assumed to act at 1/3 the height of the wall)*

$$(x_{EH})_{key} := \frac{1}{3} \cdot -G_{outfall}$$ *(resultant of lateral earth pressure assumed to act at 1/3 the height of the stem)*

Undrained Conditions			Active Pressure				Passive Pressure			
Structure Name	Location	River Station	Stem Loading (kips)	Stem Loading Location (ft)	Key Loading (kips)	Key Loading Location (ft)	Stem Loading (kips)	Stem Loading Location (ft)	Key Loading (kips)	Key Loading Location (ft)
ST-02A	LB	275	18.54	1.67	18.38	(0.67)	62.84	1.67	37.95	(0.67)
ST-03A	LB	580	24.11	1.67	20.78	(0.67)	66.26	1.67	40.34	(0.67)

ST-19A	LB	3,660	29.93	2.17	23.78	(0.67)	126.77	2.17	46.89	(0.67)
ST-22A	RB	3,925	10.40	1.67	19.34	(0.67)	64.20	1.67	38.91	(0.67)
ST-26A	RB	4,180	15.14	1.67	21.37	(0.67)	67.10	1.67	40.94	(0.67)
ST-43A	RB	6,250	13.68	1.67	20.34	(0.67)	65.63	1.67	39.90	(0.67)
ST-45A	RB	6,470	9.80	1.67	11.55	(0.67)	53.08	1.67	31.12	(0.67)
ST-45B	LB	6,485	20.62	1.67	16.46	(0.67)	60.08	1.67	36.02	(0.67)
0	0	-	-	-	-	-	-	-	-	-

(E

Drained Conditions			Active Pressure				Passive Pressure			
Structure Name	Location	River Station	Stem Loading (kips)	Stem Loading Location (ft)	Key Loading (kips)	Key Loading Location (ft)	Stem Loading (kips)	Stem Loading Location (ft)	Key Loading (kips)	Key Loading Location (ft)
ST-02A	LB	275	11.27	1.67	7.50	(0.67)	27.18	1.67	11.95	(0.67)
ST-03A	LB	580	16.84	1.67	11.40	(0.67)	32.75	1.67	15.85	(0.67)
ST-19A	LB	3,660	5.12	2.17	1.63	(0.67)	42.74	2.17	6.89	(0.67)
ST-22A	RB	3,925	3.14	1.67	1.80	(0.67)	19.04	1.67	6.26	(0.67)
ST-26A	RB	4,180	7.88	1.67	5.12	(0.67)	23.78	1.67	9.58	(0.67)
ST-43A	RB	6,250	6.42	1.67	4.10	(0.67)	22.32	1.67	8.55	(0.67)
ST-45A	RB	6,470	2.53	1.67	1.38	(0.67)	18.44	1.67	5.83	(0.67)
ST-45B	LB	6,485	13.35	1.67	8.96	(0.67)	29.26	1.67	13.41	(0.67)

Hydrodynamic Loading

Hydro-Dynamic Loads									
Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Top of Outlet Wall Ft	Flood Elev. (100 year) (ft)	Equiv. Hydro Elev (ft)	Bot of Wall (Flowline) (ft)	Loading Height (ft)
ST-02A	LB	275	923.17	922.15	926.67	937.15	937.15	923.17	-
ST-03A	LB	580	923.90	922.00	927.40	936.50	936.50	923.90	-
ST-19A	LB	3,660	897.06	894.50	902.56	918.75	918.75	897.06	-
ST-22A	RB	3,925	898.68	895.00	902.18	918.75	918.75	898.68	-
ST-26A	RB	4,180	899.11	896.20	902.61	918.00	918.00	899.11	-
ST-43A	RB	6,250	893.12	888.20	896.62	912.00	912.00	893.12	-
ST-45A	RB	6,470	896.67	891.00	900.17	910.00	910.00	896.67	-
ST-45B	LB	6,485	898.87	890.50	902.37	909.75	909.75	898.87	-

$(Hd)_{height} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ \vdots \end{bmatrix} ft$ *(resultant height of hydraulic loading from hydrodynamic forces)*

$\sigma_{Hd} := \gamma_{water} \cdot \left(0.5 \cdot (Hd)_{height}\right) = \begin{bmatrix} 0 \\ 0 \\ 0 \\ \vdots \end{bmatrix} psi$ *(hydrodynamic pressure loading to the stem wall)*

$Hd := \overrightarrow{\sigma_{Hd} \cdot \left(C_{outfall} \cdot E_{outfall}\right)} = \begin{bmatrix} 0 \\ 0 \\ 0 \\ \vdots \end{bmatrix} kip$ *(hydrodynamic loading to the stem wall)*

$x_{Hd} := \frac{(Hd)_{height}}{3}$ *(height of the resultant hydrodynamic loading)*

Hydro-Dynamic Loads

Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Top of Outlet Wall Ft	Loading Height (ft)	Hd (kips)	Resultant Height (ft)
ST-02A	LB	275	923.17	922.15	926.67	-	-	-
ST-03A	LB	580	923.90	922.00	927.40	-	-	-
ST-19A	LB	3,660	897.06	894.50	902.56	-	-	-
ST-22A	RB	3,925	898.68	895.00	902.18	-	-	-
ST-26A	RB	4,180	899.11	896.20	902.61	-	-	-
ST-43A	RB	6,250	893.12	888.20	896.62	-	-	-
ST-45A	RB	6,470	896.67	891.00	900.17	-	-	-
ST-45B	LB	6,485	898.87	890.50	902.37	-	-	-

Wave Loading

Hydro-Dynamic Wave Load

Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Pipe Size in	Pipe Wall in	Top of Outlet Wall Ft	Wave Loading (kip)	Resultant Height (ft)
ST-02A	LB	275	923.17	922.15	24	4	926.67	-	5.0
ST-03A	LB	580	923.90	922.00	24	4	927.40	-	5.0
ST-19A	LB	3,660	897.06	894.50	48	8	902.56	-	6.5
ST-22A	RB	3,925	898.68	895.00	24	4	902.18	-	5.0
ST-26A	RB	4,180	899.11	896.20	24	4	902.61	-	5.0
ST-43A	RB	6,250	893.12	888.20	24	4	896.62	-	5.0

$Hw := (Hw)$ *(wave loading on structure)*

$x_{Hw} := E_{outfall}$ *(wave loading location on structure - conservatively assumed at the top of the stem wall)*

Impact Loading

Dynamic Impact Loading

Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Pipe Size in	Pipe Wall in	Top of Outlet Wall Ft	Impact Loading (kip)	Resultant Height (ft)
ST-02A	LB	275	923.17	922.15	24	4	926.67	-	5.0
ST-03A	LB	580	923.90	922.00	24	4	927.40	-	5.0
ST-19A	LB	3,660	897.06	894.50	48	8	902.56	-	6.5
ST-22A	RB	3,925	898.68	895.00	24	4	902.18	-	5.0
ST-26A	RB	4,180	899.11	896.20	24	4	902.61	-	5.0
ST-43A	RB	6,250	893.12	888.20	24	4	896.62	-	5.0

$IM := (IM)$ *(impact loading on structure)*

$x_{IM} := E_{outfall}$ *(impact loading location on structure - conservatively assumed at the top of the stem wall)*

(impact loading location on structure - conservatively assumed at the top of the stem wall)

Load Combinations

			Loading								
			Permanent Loads				Temporary Loads		Dynamic Loads		
Structure Name	Location	River Station	D Dead [kips]	EV Vert Earth [kips]	EH Lat Earth [kips]	Gravity [kips]	Hs Hydro-static 100 yr [kips]	Hs Hydro-static 10 yr [kips]	Hd Hyro-dynamic [kips]	Hw Wave [kips]	IM Debris [kips]
ST-02A	LB	2+75	20.2	1.6	18.5	0.0	3.9	3.9	0.0	0.0	0.0
ST-03A	LB	5+80	20.2	1.6	24.1	0.0	3.9	3.9	0.0	0.0	0.0
ST-19A	LB	36+60	34.0	1.5	29.9	0.0	9.2	9.2	0.0	0.0	0.0
ST-22A	RB	39+25	20.2	1.6	10.4	0.0	3.9	3.9	0.0	0.0	0.0
ST-26A	RB	41+80	20.2	1.6	15.1	0.0	3.9	3.9	0.0	0.0	0.0
ST-43A	RB	62+50	20.2	1.6	13.7	0.0	3.9	3.9	0.0	0.0	0.0
ST-45A	RB	64+70	20.2	1.6	9.8	0.0	3.9	3.9	0.0	0.0	0.0
ST-45B	LB	64+85	20.2	1.6	20.6	0.0	3.9	3.9	0.0	0.0	0.0

$$MaxLC_2(LC1, LC2) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}(LC1) \\ \quad LC_i \leftarrow \max\left(LC1_i, LC2_i\right) \\ \text{return } LC \end{array} \right.$$

$$MinLC_2(LC1, LC2) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}(LC1) \\ \quad LC_i \leftarrow min\left(LC1_i, LC2_i\right) \\ \text{return } LC \end{array} \right.$$

$$MaxLC_4(LC1, LC2, LC3, LC4) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}(LC1) \\ \quad LC_{i,1} \leftarrow \max\left(LC1_i, LC2_i, LC3_i, LC4_i\right) \\ \quad LC_{i,2} \leftarrow min\left(LC1_i, LC2_i, LC3_i, LC4_i\right) \\ \text{return } LC \end{array} \right.$$

$(\gamma_p)_{Serve} = 1$ *(serviceability load factor for unusual loading, EM 1110-2-2104, Table 3-3)*

$(\gamma_p)_{Serve.n} = 1.6$ *(serviceability load factor for unusual loading, EM 1110-2-2104, Table 3-4)*

$(\gamma_p)_{Serve.u} = 2.2$ *(serviceability load factor for usual loading, EM 1110-2-2104, Table 3-4)*

Serviceability Loads, Lateral

$(LC1_{Sv})_{Lat} := (\gamma_p)_{Serve.u} \cdot \left(\begin{array}{l} (EH_{a.ud})_{stem} \\ + (Hs_{100})_{DS} + MaxLC_2(Hw, IM) \end{array} \right)$ *(load case 1, serviceability for stem wall, normal operating)*

$(LC2A_{Sv})_{Lat} := (\gamma_p)_{Serve} \cdot \left((EH_{a.ud})_{stem} + (Hs_{10})_{DS} \right)$ *(load case 2A, serviceability for stem wall, earthquake)*

$(LC2B_{Sv})_{Lat} := (\gamma_p)_{Serve} \cdot \left((EH_{a.ud})_{stem} + (Hs_{10})_{DS} \right)$ *(load case 2B, serviceability for stem wall, earthquake)*

$(LC3_{Sv})_{Lat} := (\gamma_p)_{Serve.u} \cdot \left((EH_{a.ud})_{stem} \right)$ *(load case 3, serviceability for stem wall, construction)*

Note: Load Case 3 for the construction condition loading does not include soil surcharge loadings. The surcharge loading from the floodwall experienced on the outlet structure is accounted for in the lateral soil pressure for the structure [reference above]. Project specifications do not allow for heavy construction equipment near the newly constructed outfall structures, and thus the loading will not drive the design.

$(LC_{serv})_{Lat} := MaxLC_4\left((LC1_{Sv})_{Lat}, (LC2A_{Sv})_{Lat}, (LC2B_{Sv})_{Lat}, (LC3_{Sv})_{Lat}\right)$

(serviceability load cases for stem wall, lateral loads)

Serviceability Loads, Vertical

$(LC1_{Sv})_{Vert} := (\gamma_p)_{Serve.u} \cdot (D + EV)$ *(load case 1, serviceability for stem wall, normal operating)*

$(LC2A_{Sv})_{Vert} := (\gamma_p)_{Serve} \cdot (D + EV)$ *(load case 2A, serviceability for stem wall, earthquake)*

$(LC2B_{Sv})_{Vert} := (\gamma_p)_{Serve} \cdot (D + EV)$ *(load case 2B, serviceability for stem wall, earthquake)*

$(LC3_{Sv})_{Vert} := (\gamma_p)_{Serve.u} \cdot (D + EV)$ *(load case 3, serviceability for stem wall, construction)*

Note: Load Case 3 for the construction condition loading does not include soil surcharge loadings. The surcharge loading from the floodwall experienced on the outlet structure is accounted for in the lateral soil pressure for the structure [reference above]. Project specifications do not allow for heavy construction equipment near the newly constructed outfall structures, and thus the loading will not drive the design.

$(LC_{serv})_{Vert} := MaxLC_4\left((LC1_{Sv})_{Vert}, (LC2A_{Sv})_{Vert}, (LC2B_{Sv})_{Vert}, (LC3_{Sv})_{Vert}\right)$

(serviceability load cases for stem wall, vertical loads)

Strength Loads, Lateral

$(LC1_{St})_{Lat} := (\gamma_p)_{EH.d.act} \cdot (EH_{a.ud})_{stem} + (\gamma_{pr})_n \cdot (Hs_{100})_{DS} + (\gamma_c)_{Lt} \cdot MaxLC_2(Hw, IM)$ *(load case 1, strength for stem wall, normal operating)*

$(LC2A_{St})_{Lat} := (\gamma_p)_{EH.d.act} \cdot (EH_{a.ud})_{stem} + (\gamma_c)_{Lt} \cdot (Hs_{10})_{DS}$ *(load case 2A, strength for stem wall, earthquake)*

$(LC2B_{St})_{Lat} := (\gamma_c)_{Lt} \cdot (EH_{a.ud})_{stem} + (\gamma_c)_{Lt} \cdot (Hs_{10})_{DS}$ *(load case 2B, strength for stem wall, earthquake)*

$(LC3_{St})_{Lat} := (\gamma_p)_{EH.d.act} \cdot (EH_{a.ud})_{stem}$ *(load case 3, strength for stem wall, construction)*

$(LC_{strgth})_{Lat} := MaxLC_4\left((LC1_{St})_{Lat}, (LC2A_{St})_{Lat}, (LC2B_{St})_{Lat}, (LC3_{St})_{Lat}\right)$

Strength Loads, Vertical

$(LC1_{St})_{Vert} := (\gamma_p)_{D.max} \cdot D + (\gamma_p)_{EV.max} \cdot EV$ *(load case 1, strength for stem wall, normal operating)*

$(LC2A_{St})_{Vert} := (\gamma_p)_{D.max} \cdot D + (\gamma_p)_{EV.max} \cdot EV$ *(load case 2A, strength for stem wall, earthquake)*

$(LC2B_{St})_{Vert} := 1.0 \cdot D + (\gamma_p)_{EV.min} \cdot EV$ *(load case 2B, strength for stem wall, earthquake)*

$(LC3_{St})_{Vert} := (\gamma_p)_{D.max} \cdot D + (\gamma_p)_{EV.max} \cdot EV$ *(load case 3, strength for stem wall, construction)*

$(LC_{strgth})_{Vert} := MaxLC_4\left((LC1_{St})_{Vert}, (LC2A_{St})_{Vert}, (LC2B_{St})_{Vert}, (LC3_{St})_{Vert}\right)$

Lateral Loads

$(LC_{Lat})_{Max} := MaxLC_2\left((LC_{serv})_{Lat}^{\langle 1 \rangle}, (LC_{strgth})_{Lat}^{\langle 1 \rangle}\right)$ *(maximum of the lateral loading, serviceability and strength)*

$(LC_{Lat})_{Min} := MinLC_2\left((LC_{serv})_{Lat}^{\langle 2 \rangle}, (LC_{strgth})_{Lat}^{\langle 2 \rangle}\right)$ *(minimum of the lateral loading, serviceability and strength)*

$LC_{Lat} := \text{augment}\left((LC_{Lat})_{Max}, (LC_{Lat})_{Min}\right) = \begin{bmatrix} 49.4 & 22.4 \\ 61.6 & 28 \\ & \vdots \end{bmatrix} kip$

Vertical Loads

$(LC_{Vert})_{Max} := MaxLC_2\left((LC_{serv})_{Vert}^{\langle 1\rangle}, (LC_{strgth})_{Vert}^{\langle 1\rangle}\right)$ *(maximum of the lateral loading, serviceability and strength)*

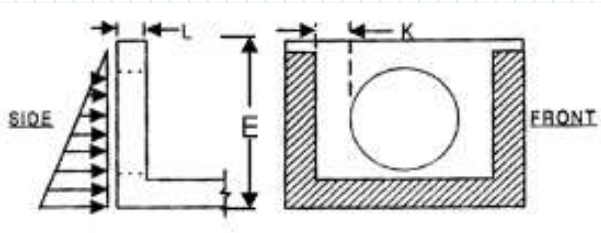
$(LC_{Vert})_{Min} := MinLC_2\left((LC_{serv})_{Vert}^{\langle 2\rangle}, (LC_{strgth})_{Vert}^{\langle 2\rangle}\right)$ *(minimum of the lateral loading, serviceability and strength)*

$LC_{Vert} := \text{augment}\left((LC_{Vert})_{Max}, (LC_{Vert})_{Min}\right) = \begin{bmatrix} 48.1 & 21.9 \\ 48.1 & 21.9 \\ & \vdots \end{bmatrix} kip$

Headwall Reinforcement Design

```
Check(X_U, X_N) :=
  for i ∈ ORIGIN .. last(X_U)
    if X_U_i > X_N_i
      ch_i ← "NO GOOD"
    else if X_U_i = X_N
      "N/A"
    else
      ch_i ← "OK"
  ch
```

Vertical Beam Between Pipe Opening and Wingwall



$b := K_{outfall} - Pipe\_Wt \cdot in$ *(strip width)*

$L_{strip} := E_{outfall}$ *(length of analysis beam)*

$Area_{wall} := \overrightarrow{\left(E_{outfall} \cdot P_{outfall}\right)} - \frac{\pi}{4} \cdot (P_{pipe.in})_{outfall}^{\ 2}$ *(stem wall surface area less pipe inlet)*

$p_{lat} :=$ *(equivalent lateral pressure to back face of stem wall)*

```
for i ∈ 1 .. ORIGIN
  if Area_wall_i = 0
    (LC_Lat)Max_i / 1 in^2
  else
    (LC_Lat)Max_i / Area_wall_i
```

$w_{lat} := \overrightarrow{p_{lat} \cdot b}$ *(average distributed force due to lateral loading)*

$W_{Lat} := \overrightarrow{w_{lat} \cdot E_{outfall} \cdot 0.5}$ *(resultant distributed force due to lateral loading)*

$M_{Lat} := 0.1283 \cdot \overrightarrow{W_{Lat} \cdot E_{outfall}}$ *(maximum moment, AISC 15th ED. Table 3-23, loading case 2)*

$Rxn_{Lat} := \frac{2}{3} \cdot W_{Lat} = \begin{bmatrix} 3.5 \\ 3.5 \\ 3.3 \\ \vdots \end{bmatrix} kip$ *(maximum resultant, AISC 15th ED. Table 3-23, loading case 2)*

Stem Wall Concrete Design

Stem Wall Layout and Design

								Resultant Loads			Vertical Rebar			
Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Pipe Size in	Pipe Wall in	Top of Outlet Wall Ft	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rsn [kips]	Vert Bar Size [size]	Vert Bar Size [in]	Vert bar Spacing [in]	Conc Cover [in]
ST-02A	LB	2+75	923.17	922.15	24	4	926.67	5.279	3.4	3.5	6	0.8	12.0	3.0
ST-03A	LB	5+80	923.9	922	24	4	927.4	5.279	3.4	3.5	6	0.8	12.0	3.0
ST-19A	LB	36+60	897.06	894.5	48	8	902.56	4.902	4.1	3.3	6	0.8	12.0	3.0
ST-22A	RB	39+25	898.68	895	24	4	902.18	5.279	3.4	3.5	6	0.8	12.0	3.0
ST-26A	RB	41+80	899.11	896.2	24	4	902.61	5.279	3.4	3.5	6	0.8	12.0	3.0
ST-43A	RB	62+50	893.12	888.2	24	4	896.62	5.279	3.4	3.5	6	0.8	12.0	3.0
ST-45A	RB	64+70	896.67	891	24	4	900.17	5.279	3.4	3.5	6	0.8	12.0	3.0
ST-45B	LB	64+85	898.87	890.5	24	4	902.37	5.279	3.4	3.5	6	0.8	12.0	3.0

$(b_s)_{stem} := (b_s)_{stem} \cdot in$ *(bar size for vertical bars in stem walls, from table above)*

$(s_{vert})_{stem} := (s_{vert})_{stem} \cdot in$ *(bar spacing for vertical bars in stem walls, from table above)*

$(A_s)_{stem} := \text{for } i \in \text{ORIGIN} .. \text{last}\left((b_s)_{stem}\right)$

$\quad \text{if } (s_{vert})_{stem_i} = 0$

$\quad\quad as_i \leftarrow \dfrac{0.25 \cdot \pi \cdot (b_s)_{stem_i}^{\ 2}}{1}$

$\quad \text{else}$

$\quad\quad as_i \leftarrow \dfrac{\overrightarrow{0.25 \cdot \pi \cdot (b_s)_{stem_i}^{\ 2} \cdot b_i}}{\left((s_{vert})_{stem_i}\right)}$

$\quad as$

$= \begin{bmatrix} 0.52 \\ 0.52 \\ 0.37 \\ 0.52 \\ 0.52 \\ 0.52 \\ 0.52 \\ 0.52 \\ \vdots \end{bmatrix} in^2$ *(area of steel for vertical bars in stem walls)*

$Cover := cover \cdot in$ *(concrete cover for reinforcing steel, from table above)*

$d := L_{outfall} - Cover - \left(0.5 \cdot (b_s)_{stem}\right)$ *(d distance for vertical reinforcement bars in stem)*

$d_{Horz} := L_{outfall} - Cover - \left(1.5 \cdot (b_s)_{stem}\right)$ *(d distance for horizontal reinforcement bars in stem)*

$a \cdot 0.85 \cdot f'c \cdot b = (A_s)_{stem} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$\beta_1 := \text{if } f'_c \geq 8000\ psi$

$\quad 0.65$

$\quad \text{else if } f'_c > 4000\ psi$

$\quad\quad 0.85 - \left(\dfrac{0.05 \cdot \left(f'_c - 4000\ psi\right)}{1000\ psi}\right)$

$\quad \text{else}$

$\quad\quad 0.85$

$= 0.8$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

$c = \left(\dfrac{\varepsilon_c}{\varepsilon_c + \varepsilon_t}\right) \cdot d$ *(neutral axis depth for beams)*

$c := \left(\dfrac{0.003}{0.003 + 0.004}\right) \cdot d$

$a := \beta_1 \cdot c$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$$\phi M_n := \overrightarrow{\phi_b \cdot (A_s)_{stem} \cdot f_y \cdot \left(d - \frac{a}{2}\right)}$$ *(nominal flexural strength, ACI 318-19)*

$$Flex_{nom} := Check\left(M_{Lat}, \phi M_n\right)$$ *(nominal flexural check)*

Flexural Check

			Resultant Loads			Vertical Rebar						
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rsn [kips]	Vert Bar Size [size]	Vert Bar Size [in]	Vert bar Spacing [in]	Conc Cover [in]	Mlat [kip-ft]	ϕMn [kip-ft]	Flexural Check
ST-02A	LB	2+75	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
ST-03A	LB	5+80	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
ST-19A	LB	36+60	4.902	4.1	3.3	6	0.8	12.0	3.0	4.1	11.8	OK
ST-22A	RB	39+25	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
ST-26A	RB	41+80	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
ST-43A	RB	62+50	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
ST-45A	RB	64+70	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
ST-45B	LB	64+85	5.279	3.4	3.5	6	0.8	12.0	3.0	3.4	16.5	OK
0	0	0+00	0.000	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	OK

minimum required steel for flexure

$K_u := $ for $i \in$ ORIGIN .. last(b) *(ratio of reinforcement stress block depth)*

if $b_i = 0$

$$b_i \leftarrow 1 - \sqrt{1 - \frac{\frac{M_{Lat_i}}{\phi_b}}{1 \cdot (kip \cdot ft)}}$$

else

$$b_i \leftarrow 1 - \sqrt{1 - \frac{\frac{M_{Lat_i}}{\phi_b}}{\overrightarrow{0.425 \cdot f'_c \cdot b_i \cdot d_i^2}}}$$

b

$$(A_{s.min})_{ten} := \frac{\overrightarrow{0.85 \cdot f'_c \cdot K_u \cdot b \cdot d}}{f_y}$$ *(minimum area of steel required for flexural strength)*

minimum required steel per ACI

$$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot \overrightarrow{b \cdot d}$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$$(A_{s.min})_b := \frac{200\ psi}{f_y} \cdot \overrightarrow{b \cdot d}$$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{\overrightarrow{L_{outfall} \cdot E_{outfall}}}{144}\right)$$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$MinA_s\left((A_{s.min})_{ten}, (A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right) :=$ for $i \in$ ORIGIN .. last$\left((A_{s.min})_{ten}\right)$

$A_{s_i} \leftarrow \max\left((A_{s.min})_{ten_i}, (A_{s.min})_{a_i}, (A_{s.min})_{b_i}, (A_{s.min})_{temp_i}\right)$

return A_s

$A_{s.min} := MinA_s\left((A_{s.min})_{ten}, (A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s\_1.3} := (A_{s.min})_{ten} \cdot \frac{4}{3}$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$A_{s.req} := \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left(A_{s\_1.3}\right)$ *(required steel to meet steel requirements)*

```
if A_s_1.3_i < (A_s)_stem_i
    as_i ← (A_s.min)_ten_i
else
    as_i ← A_s.min_i
as
```

$Steel\_Check := Check\left(A_{s.req}, (A_s)_{stem}\right)$ *(check stem reinforcement to required reinforcement)*

Minimum Steel Check

			Vertical Rebar				Minimum Steel							
Structure Name	Location	River Station	Vert Bar Size [size]	Vert Bar Size [in]	Vert bar Spacing [in]	Conc Cover [in]	A_{s} min Flex [in^2]	A_{s} min 1 [in^2]	A_{s} min 2 [in^2]	A_{s} min temp [in^2]	A_{s} min [in^2]	A_{s} req [in^2]	A_{s} stem [in^2]	Steel Check
ST-02A	LB	2+75	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK
ST-03A	LB	5+80	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK
ST-19A	LB	36+60	6	0.8	12.0	3.0	0.11	0.29	0.29	0.02	0.29	0.1	0.4	OK
ST-22A	RB	39+25	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK
ST-26A	RB	41+80	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK
ST-43A	RB	62+50	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK
ST-45A	RB	64+70	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK
ST-45B	LB	64+85	6	0.8	12.0	3.0	0.09	0.41	0.40	0.02	0.41	0.1	0.5	OK

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := \text{for } i \in 1 .. \text{last}\left((A_s)_{stem}\right)$ *(reinforcement ratio from EM 1110-2-2104 D-3)*

$\quad \rho_{max_i} \leftarrow 0.5 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y}\right) \cdot \left(\frac{87000}{87000 + \left(f_y \cdot psi^{-1}\right)}\right)$

$\quad \rho_{max}$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \text{for } i \in 1 .. \text{last}\left((A_s)_{stem}\right)$ *(actual reinforcement ratio for concrete section)*

```
if b_i = 0
    ρ_i ← (A_s)_stem_i · ft^-2 / 1
else
    ρ_i ← (A_s)_stem_i / (b_i · d_i)
ρ
```

$\rho\_check := Check\left(\rho_{actual}, \rho_{max}\right)$

Reinforcement Ratio Check									
			Vertical Rebar				Reinforcement Ratio		
Structure Name	Location	River Station	Vert Bar Size [size]	Vert Bar Size [in]	Vert bar Spacing [in]	Conc Cover [in]	ρ.. max	ρ.. act	ρ.. check
ST-02A	LB	2+75	6	0.8	12.0	3.0	0.016	0.004	OK
ST-03A	LB	5+80	6	0.8	12.0	3.0	0.016	0.004	OK
ST-19A	LB	36+60	6	0.8	12.0	3.0	0.016	0.004	OK
ST-22A	RB	39+25	6	0.8	12.0	3.0	0.016	0.004	OK
ST-26A	RB	41+80	6	0.8	12.0	3.0	0.016	0.004	OK
ST-43A	RB	62+50	6	0.8	12.0	3.0	0.016	0.004	OK
ST-45A	RB	64+70	6	0.8	12.0	3.0	0.016	0.004	OK
ST-45B	LB	64+85	6	0.8	12.0	3.0	0.016	0.004	OK

Shear Reinforcement

$\mu := 0.6\ \lambda_c$ *(coefficient of friction, ACI 318-19, Table 22.9.4.2)*

$(A_{vf})_{stem} := (A_s)_{stem} - A_{s.req}$ *(area of steel on the compression face of the stem, used for shear reinforcement)*

$V_n := \mu \cdot (A_{vf})_{stem} \cdot f_y$ *(shear friction capacity from reinforcement, ACI 318-19, Eq 22.9.4.2)*

$\phi V_n := \phi_V \cdot V_n$ *(factored shear friction capacity from reinforcement)*

$Vn\_check := Check\left(Rxn_{Lat}, \phi V_n\right)$ *(check shear capacity of reinforcement)*

Shear Friction Check								
			Resultant Loads		Vertical Rebar		Shear Reinforcement	
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Rxn [kips]	Vert Bar Size [size]	Vert bar Spacing [in]	ϕVn [kips]	Check
ST-02A	LB	2+75	5.279	3.5	6	12.0	11.5	OK
ST-03A	LB	5+80	5.279	3.5	6	12.0	11.5	OK
ST-19A	LB	36+60	4.902	3.3	6	12.0	7.1	OK
ST-22A	RB	39+25	5.279	3.5	6	12.0	11.5	OK
ST-26A	RB	41+80	5.279	3.5	6	12.0	11.5	OK
ST-43A	RB	62+50	5.279	3.5	6	12.0	11.5	OK
ST-45A	RB	64+70	5.279	3.5	6	12.0	11.5	OK
ST-45B	LB	64+85	5.279	3.5	6	12.0	11.5	OK
0	0	0+00	0.000	0.0	0	0.0	0.0	OK

Concrete Shear Capacity

$$\lambda_{s.1} := \sqrt{\frac{2}{1 + \frac{d}{10\ in}}}$$ *(size modification factor ACI 318-19, Eq. 22.5.5.1.3)*

$\lambda_s := \text{for } i \in 1 .. \text{last}\left(\lambda_{s.1}\right)$ *(size modification factor, limited to 1.0 ACI 318-19, Eq. 22.5.5.1.3)*

$\quad \lambda_{s_i} \leftarrow min\left(\lambda_{s.1_i}, 1.0\right)$

$\quad \lambda_s$

$(N_u/6A_g)_{max} := 0.05 \cdot f'_c$ *(limit of Nu/6Ag ACI 318-19, Section 22.5.5.1.2)*

$(N_u/6A_g)_{act} := \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left((W_{strt})_{stem}\right)$
$\quad$ if $b_i = 0$
$\quad\quad \omega_i \leftarrow \dfrac{(W_{strt})_{stem_i}}{1 \cdot in^2}$
$\quad$ else
$\quad\quad \omega_i \leftarrow \dfrac{(W_{strt})_{stem_i}}{6 \cdot L_{outfall_i} \cdot b_i}$
$\quad \omega$

(actual pressure for shear capacity)

$A_{v.min} := \text{for } i \in \mathbf{ORIGIN} .. \text{last}(b)$
$\quad A_i \leftarrow \max\left(0.75\ ft \cdot \sqrt{f'_c \cdot psi} \cdot \dfrac{b_i}{f_y}, \dfrac{\left(50\ \frac{lbf}{in}\right) \cdot b_i}{f_y}\right)$
$\quad A$

(minimum shear reinforcement, ACI 318-19,Table 9.6.3.1)

$\rho_w := \text{for } i \in \mathbf{ORIGIN} .. \text{last}(b)$
$\quad$ if $b_i = 0$
$\quad\quad \rho_i \leftarrow \dfrac{(A_s)_{stem_i}}{1 \cdot ft^2}$
$\quad$ else
$\quad\quad \rho_i \leftarrow \dfrac{(A_s)_{stem_i}}{b_i \cdot d_i}$
$\quad \rho$

(ratio of As to bwd, ACI 318-19, Ch. 2-Notation)

$(V_c)_a := \overrightarrow{\left(2 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} + (N_u/6A_g)_{act}\right) \cdot b \cdot d}$

(concrete shear capacity (a), ACI 318-19, Table 22.5.5.1)

$(V_c)_c := \overrightarrow{\left(8 \cdot \lambda_s \cdot \lambda_c \cdot (\rho_w)^{0.33} \cdot \sqrt{f'_c \cdot psi} + (N_u/6A_g)_{act}\right) \cdot b \cdot d}$

$(V_c)_{Max} := 5 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} \cdot \overrightarrow{b \cdot d}$

(maximum permissible value of concrete shear, ACI 318-19, Section 22.5.5.1.1)

$V_c := \text{for } i \in \mathbf{ORIGIN} .. \text{last}(b)$
$\quad$ if $A_{v.min_i} \le (A_{vf})_{stem_i}$
$\quad\quad V_i \leftarrow (V_c)_{a_i}$
$\quad$ else
$\quad\quad V_i \leftarrow (V_c)_{c_i}$
$\quad V$

(unfactored concrete shear capacity, ACI 318-19, Table 22.5.5.1)

$V_c := \text{for } i \in \mathbf{ORIGIN} .. \text{last}(V_c)$
$\quad V_i \leftarrow min\left(V_{c_i}, (V_c)_{Max_i}\right)$
$\quad V$

(unfactored concrete shear capacity, caped at maximum, ACI 318-19, Table 22.5.5.1)

$\phi V_c := \phi_V \cdot V_c$

(nominal shear strength of concrete)

$Vc\_Check := Check\left(Rxn_{Lat}, \phi V_c\right)$

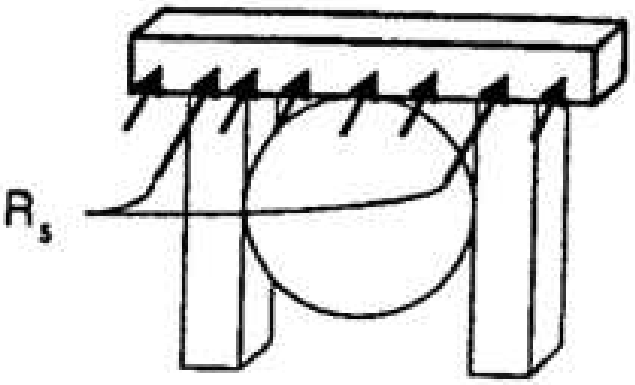
Concrete Shear Capacity										
			Resultant Loads		Vertical Rebar		Concrete Shear			
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Rxn [kips]	Vert Bar Size [size]	Vert bar Spacing [in]	Vc, calc [kips]	Vc, max [kips]	ϕVc [kips]	Check
ST-02A	LB	2+75	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
ST-03A	LB	5+80	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
ST-19A	LB	36+60	4.902	3.3	6	12.0	12.5	28.9	9.4	OK
ST-22A	RB	39+25	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
ST-26A	RB	41+80	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
ST-43A	RB	62+50	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
ST-45A	RB	64+70	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
ST-45B	LB	64+85	5.279	3.5	6	12.0	16.7	40.5	12.6	OK
0	0	0+00	0.000	0.0	0	0.0	0.0	0.0	0.0	OK

Note: Stirrups are not required to support longitudinal bars in stem wall. Wingwall bars will provide lateral containment.

Horizontal Beam Above Pipe Opening

$R_{s2} := \frac{1}{3} \cdot W_{Lat}$ *(end reaction, resolved from loading on headwall, reference vertical loading calcs)*

$EL_{hor} := FLElev \cdot ft + Pipe\_size \cdot in + Pipe\_Wt \cdot in$ *(elevation of the top of the pipe connection to headwall)*



Stem Wall Horizontal Design

$b := \text{for } j \in \text{ORIGIN} .. \text{last}(b)$
$\quad b_j \leftarrow \max\left(\left(E_{outfall_j} - Pipe\_size_j \cdot in\right) - Pipe\_Wt_j \cdot in - 1\ ft, 0\ ft\right)$
$\quad b$

(strip width above the pipe and to the top of the wall)

$w_{lat.H} := \overrightarrow{p_{lat} \cdot b}$ *(average distributed force due to lateral loading)*

$W_{Lat.H} := \overrightarrow{w_{lat.H} \cdot E_{outfall} \cdot 0.5}$ *(resultant distributed force due to lateral loading)*

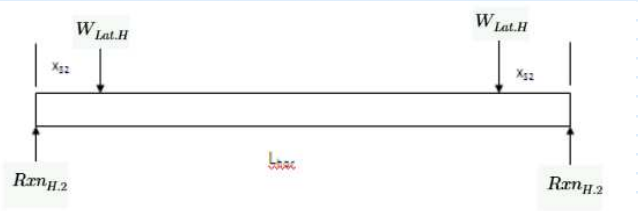
$M_{Lat} := 0.1283 \cdot \overrightarrow{W_{Lat.H} \cdot E_{outfall}}$ *(maximum moment, AISC 15th ED. Table 3-23, loading case 2)*

$Rxn_{Lat.H} := \frac{1}{3} \cdot W_{Lat} = \begin{bmatrix} 1.8 \\ 1.8 \\ \vdots \end{bmatrix} kip$ *(maximum resultant, AISC 15th ED. Table 3-23, loading case 2)*

$L_{Horz} := \frac{\left(C_{outfall} + P_{outfall}\right)}{2}$ *(effective beam width, top of stem wall)*

$Rxn_{H.2} := \left(W_{Lat.H}\right) + Rxn_{Lat.H}$ *(end reaction for horizontal strip at top of stem wall)*

$$x_{s2} := \text{for } i \in \textbf{ORIGIN} .. \text{last}\left(Rxn_{Lat.H}\right) \left\| \begin{array}{l} x \leftarrow 1\ ft \\ x \end{array} \right.$$



(lateral offset for loading to end of wall)

$$M_{lat.H} := \overrightarrow{-0.5 \cdot Rxn_{H.2} \cdot L_{Horz}} + \overrightarrow{Rxn_{Lat.H} \cdot \left(\frac{L_{Horz}}{2} - x_{s2}\right)} + \frac{\overrightarrow{w_{lat.H} \cdot \left(L_{Horz}^{\ 2}\right)}}{8}$$

$$M_{lat.H} := \overrightarrow{|M_{lat.H}|}$$

Stem Wall Layout and Desig

			Resultant Loads			Horizontal Rebar			
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rxn [kips]	Horz Bar Size [size]	Horz Bar Size [in]	Horz bar Spacing [in]	Conc Cover [in]
ST-02A	LB	2+75	5.279	11.1	1.8	6	0.8	12.0	3.0
ST-03A	LB	5+80	5.279	11.1	1.8	6	0.8	12.0	3.0
ST-19A	LB	36+60	4.902	9.4	1.6	6	0.8	12.0	3.0
ST-22A	RB	39+25	5.279	11.1	1.8	6	0.8	12.0	3.0
ST-26A	RB	41+80	5.279	11.1	1.8	6	0.8	12.0	3.0
ST-43A	RB	62+50	5.279	11.1	1.8	6	0.8	12.0	3.0
ST-45A	RB	64+70	5.279	11.1	1.8	6	0.8	12.0	3.0
ST-45B	LB	64+85	5.279	11.1	1.8	6	0.8	12.0	3.0
0	0	0+00	0.000	0.0	0.0	0	0.0	0.0	0.0

$$(b_s)_{stem.H} := (b_s)_{stem.H} \cdot in$$ *(bar size for horizontal bars in stem walls, from table above)*

$$(s_{horz})_{stem} := (s_{horz})_{stem} \cdot in$$ *(bar spacing for horizontal bars in stem walls, from table above)*

$$Cover_H := cover_H \cdot in$$ *(bar cover for horizontal bars in stem walls, from table above)*

$$(A_s)_{stem.H} := \text{for } i \in 1 .. \textbf{ORIGIN} \left\| \begin{array}{l} \text{if } (s_{horz})_{stem_i} = 0 \\ \quad \left\| \dfrac{0.25 \cdot \pi \cdot (b_s)_{stem.H}^{\ 2}}{1} \right. \\ \text{else} \\ \quad \left\| \dfrac{\overrightarrow{0.25 \cdot \pi \cdot (b_s)_{stem.H}^{\ 2} \cdot b_i}}{\overrightarrow{\left((s_{horz})_{stem_i}\right)}} \right. \end{array} \right. = \begin{bmatrix} 0.74 \\ 0.74 \\ 0.74 \\ 0.74 \\ 0.74 \\ 0.74 \\ 0.74 \\ 0.74 \\ \vdots \end{bmatrix} in^2$$ *(area of steel in horizontal area of stem wall)*

$$d := \text{for } j \in \textbf{ORIGIN} .. \text{last}\left(L_{outfall}\right) \left\| \begin{array}{l} d_j \leftarrow \max\left(L_{outfall_j} - Cover_j - 1.5\ d, 0\right) \\ d \end{array} \right.$$ *(depth of reinforcement to extreme fiber)*

$$a \cdot 0.85 \cdot f'c \cdot b = (A_s)_{stem.H} \cdot f_y$$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$$c = \left(\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_t}\right) \cdot d$$

(neutral axis depth for beams)

$$c := \left(\frac{0.003}{0.003 + 0.004}\right) \cdot d$$

$$a := \beta_1 \cdot c$$

(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)

$$\phi M_n := \overrightarrow{\phi_b \cdot (A_s)_{stem} \cdot f_y \cdot \left(d - \frac{a}{2}\right)}$$

(nominal flexural strength, ACI 318-19)

$$Flex_{nom} := Check\left(M_{Lat}, \phi M_n\right)$$

(nominal flexural check)

Flexural Check - Horizontal

			Resultant Loads			Horizontal Rebar				Flexural Check		
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rxn [kips]	Horz Bar Size [size]	Horz Bar Size [in]	Horz bar Spacing [in]	Conc Cover [in]	Mlat [kip-ft]	ϕMn [kip-ft]	Flexural Check
ST-02A	LB	2+75	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK
ST-03A	LB	5+80	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK
ST-19A	LB	36+60	4.902	9.4	1.6	6	0.8	12.0	3.0	9.4	12.3	OK
ST-22A	RB	39+25	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK
ST-26A	RB	41+80	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK
ST-43A	RB	62+50	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK
ST-45A	RB	64+70	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK
ST-45B	LB	64+85	5.279	11.1	1.8	6	0.8	12.0	3.0	11.1	17.2	OK

minimum required steel for flexure

$$K_u := \text{for } i \in \text{ORIGIN} .. \text{last}(b) \quad \begin{cases} \text{if } b_i = 0 \\ \quad k_{u_i} \leftarrow 1 - \sqrt{1 - \frac{\frac{M_{lat.H_i}}{\phi_b}}{1\ (kip \cdot ft)}} \\ \text{else} \\ \quad k_{u_i} \leftarrow 1 - \sqrt{1 - \frac{\frac{M_{lat.H_i}}{\phi_b}}{\overrightarrow{0.425 \cdot f'_c \cdot b_i \cdot d_i^2}}} \\ k_u \end{cases}$$

(ratio of reinforcement stress block depth)

$$(A_{s.min})_{ten.H} := \frac{\overrightarrow{0.85 \cdot f'_c \cdot K_u \cdot b \cdot d}}{f_y}$$

(minimum area of steel required for flexural strength)

minimum required steel per ACI

$$(A_{s.min})_{a.H} := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot \overrightarrow{b \cdot d}$$

(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))

$(A_{s.min})_{b.H} := \frac{200\ psi}{f_y} \cdot \overrightarrow{b \cdot d}$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp.H} := 0.003 \cdot \left(\frac{\overrightarrow{L_{outfall} \cdot E_{outfall}}}{144} \right)$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$$MinA_s\left((A_{s.min})_{ten}, (A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right) := \left\| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left((A_{s.min})_{ten}\right) \\ \quad \left\| A_{s_i} \leftarrow \max\left((A_{s.min})_{ten_i}, (A_{s.min})_{a_i}, (A_{s.min})_{b_i}, (A_{s.min})_{temp_i}\right) \right. \\ \text{return } A_s \end{array} \right.$$

$A_{s.min.H} := MinA_s\left((A_{s.min})_{ten.H}, (A_{s.min})_{a.H}, (A_{s.min})_{b.H}, (A_{s.min})_{temp.H}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s\_1.3.H} := (A_{s.min})_{ten.H} \cdot \frac{4}{3}$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$A_{s.req.H} := MaxLC_2\left(A_{s\_1.3.H}, A_{s.min.H}\right)$ *(required steel to meet steel requirements)*

$Steel\_Check_H := Check\left(A_{s.req.H}, (A_s)_{stem.H}\right)$ *(check stem reinforcement to required reinforcement)*

Minimum Steel Check

			Horizontal Rebar			Minimum Steel - Horizontal Beam							
Structure Name	Location	River Station	Horz Bar Size [size]	Horz bar Spacing [in]	Conc Cover [in]	As, min Flex [in2]	As, min 1 [in2]	As, min 2 [in2]	As, min temp [in2]	As, min [in2]	As, req [in2]	As, stem [in2]	Steel Check
ST-02A	LB	2+75	6	12	3	0.28	0.60	0.60	0.02	0.60	0.604	0.74	OK
ST-03A	LB	5+80	6	12	3	0.28	0.60	0.60	0.02	0.60	0.604	0.74	OK
ST-19A	LB	36+60	6	12	3	0.24	0.30	0.30	0.02	0.30	0.316	0.74	OK
ST-22A	RB	39+25	6	12	3	0.28	0.60	0.60	0.02	0.60	0.604	0.74	OK
ST-26A	RB	41+80	6	12	3	0.28	0.60	0.60	0.02	0.60	0.604	0.74	OK
ST-43A	RB	62+50	6	12	3	0.28	0.60	0.60	0.02	0.60	0.604	0.74	OK
ST-45A	RB	64+70	6	12	3	0.28	0.60	0.60	0.02	0.60	0.604	0.74	OK

Maximum Tensile Reinforcement, Minimum Spacing

$$\rho_{max.H} := \left\| \begin{array}{l} \text{for } i \in 1 .. \text{last}\left((A_s)_{stem}\right) \\ \quad \left\| \rho_{max_i} \leftarrow 0.5 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \right) \cdot \left(\frac{87000}{87000 + \left(f_y \cdot psi^{-1}\right)} \right) \right. \\ \rho_{max} \end{array} \right.$$

(reinforcement ratio from EM 1110-2-2104 D-3)

(maximum reinforcement ratio for concrete section)

$$\rho_{actual.H} := \left\| \begin{array}{l} \text{for } i \in 1 .. \text{last}\left((A_s)_{stem.H}\right) \\ \quad \left\| \begin{array}{l} \text{if } b_i = 0 \\ \quad \left\| \rho_i \leftarrow \frac{(A_s)_{stem.H_i} \cdot ft^{-2}}{1} \right. \\ \text{else} \\ \quad \left\| \rho_i \leftarrow \frac{(A_s)_{stem.H_i}}{\left(b_i \cdot d_i\right)} \right. \end{array} \right. \\ \rho \end{array} \right.$$

(actual reinforcement ratio for concrete section)

$\rho\_check_H := Check\left(\rho_{actual.H}, \rho_{max.H}\right)$

Reinforcement Ratio Check

			Horizontal Rebar				Reinforcement Ratio - Horizon		
Structure Name	Location	River Station	Horz Bar Size [size]	Horz Bar Size [in]	Horz bar Spacing [in]	Conc Cover [in]	ρs, max	ρs, act	ρs, check
ST-02A	LB	2+75	6	0.75	12	3	0.016	0.004	OK
ST-03A	LB	5+80	6	0.75	12	3	0.016	0.004	OK
ST-19A	LB	36+60	6	0.75	12	3	0.016	0.008	OK
ST-22A	RB	39+25	6	0.75	12	3	0.016	0.004	OK
ST-26A	RB	41+80	6	0.75	12	3	0.016	0.004	OK
ST-43A	RB	62+50	6	0.75	12	3	0.016	0.004	OK
ST-45A	RB	64+70	6	0.75	12	3	0.016	0.004	OK
ST-45B	LB	64+85	6	0.75	12	3	0.016	0.004	OK

Shear Reinforcement

$\mu := \lambda_c$ *(coefficient of friction, ACI 318-19, Table 22.9.4.2)*

$(A_{vf})_{stem.H} := 2 \cdot (A_s)_{stem.H} - A_{s.req.H}$ *(area of steel on the compression face of the stem, used for shear reinforcement)*

$V_{n.H} := \mu \cdot (A_{vf})_{stem.H} \cdot f_y$ *(shear friction capacity from reinforcement, ACI 318-19, Eq 22.9.4.2)*

$\phi V_{n.H} := \phi_V \cdot V_{n.H}$ *(factored shear friction capacity from reinforcement)*

$Vn\_check_H := Check\left(Rxn_{Lat.H}, \phi V_{n.H}\right)$ *(check shear capacity of reinforcement)*

Shear Friction Check

			Resultant Loads		Horizontal Rebar		Shear Reinf. - Horiz	
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Rxn [kips]	Horz Bar Size [size]	Horz bar Spacing [in]	ϕVn [kips]	Check
ST-02A	LB	2+75	5.279	1.8	6.0	12.0	39.1	OK
ST-03A	LB	5+80	5.279	1.8	6.0	12.0	39.1	OK
ST-19A	LB	36+60	4.902	1.6	6.0	12.0	52.0	OK
ST-22A	RB	39+25	5.279	1.8	6.0	12.0	39.1	OK
ST-26A	RB	41+80	5.279	1.8	6.0	12.0	39.1	OK
ST-43A	RB	62+50	5.279	1.8	6.0	12.0	39.1	OK
ST-45A	RB	64+70	5.279	1.8	6.0	12.0	39.1	OK
ST-45B	LB	64+85	5.279	1.8	6.0	12.0	39.1	OK

Concrete Shear Capacity

$$\lambda_{s.1} := \sqrt{\frac{2}{1 + \frac{d}{10\ in}}}$$ *(size modification factor ACI 318-19, Eq. 22.5.5.1.3)*

$\lambda_s := \text{for } i \in 1 .. \text{last}\left(\lambda_{s.1}\right)$ *(size modification factor, limited to 1.0 ACI 318-19, Eq. 22.5.5.1.3)*

$\quad \lambda_{s_i} \leftarrow min\left(\lambda_{s.1_i}, 1.0\right)$

$\quad \lambda_s$

$(N_u/6A_g)_{max.H} := 0.05 \cdot f'_c$ *(limit of Nu/6Ag ACI 318-19, Section 22.5.5.1.2)*

(actual pressure for shear capacity)

$$(N_u/6A_g)_{act.H} := \text{for } i \in \text{ORIGIN} .. \text{last}\left((W_{strt})_{stem}\right)$$
$$\quad \text{if } b_i = 0$$
$$\quad\quad \omega_i \leftarrow \frac{(W_{strt})_{stem_i}}{1 \cdot in^2}$$
$$\quad \text{else}$$
$$\quad\quad \omega_i \leftarrow \frac{(W_{strt})_{stem_i}}{6 \cdot L_{outfall_i} \cdot b_i}$$
$$\quad \omega$$

(minimum shear reinforcement, ACI 318-19,Table 9.6.3.1)

$$A_{v.min.H} := \text{for } i \in \text{ORIGIN} .. \text{last}(b)$$
$$\quad A_i \leftarrow \max\left(0.75\ ft \cdot \sqrt{f'_c \cdot psi} \cdot \frac{b_i}{f_y}, \frac{\left(50\ \frac{lbf}{in}\right) \cdot b_i}{f_y}\right)$$
$$\quad A$$

(ratio of As to bwd, ACI 318-19, Ch. 2-Notation)

$$\rho_{w.H} := \text{for } i \in \text{ORIGIN} .. \text{last}(b)$$
$$\quad \text{if } b_i = 0$$
$$\quad\quad \rho_i \leftarrow \frac{(A_s)_{stem.H_i}}{1 \cdot ft^2}$$
$$\quad \text{else}$$
$$\quad\quad \rho_i \leftarrow \frac{(A_s)_{stem.H_i}}{b_i \cdot d_i}$$
$$\quad \rho$$

(concrete shear capacity (a), ACI 318-19, Table 22.5.5.1)

$$(V_c)_{a.H} := \overrightarrow{\left(2 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} + (N_u/6A_g)_{act.H}\right) \cdot b \cdot d}$$

$$(V_c)_{c.H} := \overrightarrow{\left(8 \cdot \lambda_s \cdot \lambda_c \cdot (\rho_{w.H})^{0.33} \cdot \sqrt{f'_c \cdot psi} + (N_u/6A_g)_{act.H}\right) \cdot b \cdot d}$$

(maximum permissible value of concrete shear, ACI 318-19, Section 22.5.5.1.1)

$$(V_c)_{Max.H} := 5 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} \cdot \overrightarrow{b \cdot d}$$

(unfactored concrete shear capacity, ACI 318-19, Table 22.5.5.1)

$$V_{c.H} := \text{for } i \in \text{ORIGIN} .. \text{last}(b)$$
$$\quad \text{if } A_{v.min.H_i} \le (A_{vf})_{stem.H_i}$$
$$\quad\quad V_i \leftarrow (V_c)_{a.H_i}$$
$$\quad \text{else}$$
$$\quad\quad V_i \leftarrow (V_c)_{c.H_i}$$
$$\quad V$$

(unfactored concrete shear capacity, caped at maximum, ACI 318-19, Table 22.5.5.1)

$$V_{c.H} := \text{for } i \in \text{ORIGIN} .. \text{last}(V_{c.H})$$
$$\quad V_i \leftarrow min\left(V_{c.H_i}, (V_c)_{Max.H_i}\right)$$
$$\quad V$$

(nominal shear strength of concrete)

$$\phi V_{c.H} := \phi_V \cdot V_{c.H}$$

$$Vc\_Check_H := Check\left(Rxn_{Lat.H}, \phi V_{c.H}\right)$$

Concrete Shear Capacity												
			Resultant Loads		Horizontal Rebar		Concrete Shear					
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Rsn [kips]	Horz Bar Size [size]	Horz bar Spacing [in]	Va [kips]	Vc [kips]	Vc, calc [kips]	Vc, max [kips]	ϕVc [kips]	Check
ST-02A	LB	2+75	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK
ST-03A	LB	5+80	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK
ST-19A	LB	36+60	4.902	1.6	6.0	12.0	13.0	10.9	13.0	30.2	9.8	OK
ST-22A	RB	39+25	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK
ST-26A	RB	41+80	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK
ST-43A	RB	62+50	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK
ST-45A	RB	64+70	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK
ST-45B	LB	64+85	5.279	1.8	6.0	12.0	24.7	16.3	24.7	60.4	18.5	OK

Note: Stirrups are not required to support longitudinal bars in stem wall. Wingwall bars will provide lateral containment.

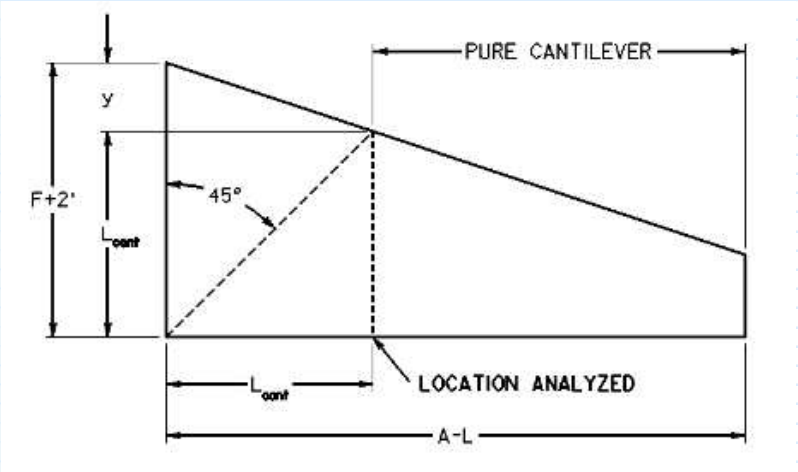
Wingwall Reinforcement Design

$$AA := \text{for } i \in \text{ORIGIN} .. \text{last}(F_{outfall})$$
$$\quad aa_i \leftarrow 8\ in$$
$$\quad aa$$

(vertical offset from top of stemwall to top of wingwall)

$$b := \text{for } i \in \text{ORIGIN} .. \text{last}(F_{outfall})$$
$$\quad b_i \leftarrow 1\ ft$$
$$\quad b$$

(lateral dimension of section being analyzed)



$$Slope := \text{for } i \in \text{ORIGIN} .. \text{last}(F_{outfall})$$
$$\quad \text{if } A_{outfall_i} = 0$$
$$\quad\quad s_i \leftarrow \frac{0\ ft}{1\ ft}$$
$$\quad \text{else}$$
$$\quad\quad s_i \leftarrow \frac{F_{outfall_i} - AA_i}{A_{outfall_i} - L_{outfall_i}}$$
$$\quad s$$

(wingwall slope, per layout)

(zero slope for unused layout)

$$(A{-}L)_{outfall} := \text{for } i \in \text{ORIGIN} .. \text{last}(A_{outfall})$$
$$\quad \text{if } A_{outfall_i} = 0$$
$$\quad\quad a_i \leftarrow 1\ ft$$
$$\quad \text{else}$$
$$\quad\quad a_i \leftarrow A_{outfall_i} - L_{outfall_i}$$
$$\quad a$$

(slope denominator adjusted to prevent zero values)

$$\frac{y}{L_{cant}} = \frac{F_{outfall} - AA}{A_{outfall} - L_{outfall}}$$

(similar triangles equation)

$y + L_{cant} = E_{outfall} - AA$

$$L_{cant} := \frac{E_{outfall} - AA}{\frac{F_{outfall}}{(A-L)_{outfall}} + 1}$$ *(similar triangles equation)*

$(\Phi_U)_{M\_V\_SLC} = 2.2$ *(load factor for serviceability loading, EM 1110-2-2104 Table 3-4)*

$(w_{lat})_{wing} := \overrightarrow{(\Phi_U)_{M\_V\_SLC} \cdot \left(\overrightarrow{\gamma_{water} \cdot L_{cant}} + \left(\overrightarrow{\gamma'_{soil} \cdot L_{cant} \cdot (K_{o\beta})_{RS}}\right)\right) \cdot b}$ *(rate of lateral loading to the side face of the wingwalls)*

$f'_{cant} := \overrightarrow{0.5 \cdot (w_{lat})_{wing} \cdot L_{cant}}$ *(lateral loading to the side face of the wingwalls)*

$dist_{cant} := \frac{1}{3} \cdot L_{cant}$ *(distance to the moment for cantilevered section)*

$M_{cant} := \overrightarrow{f'_{cant} \cdot dist_{cant}}$ *(maximum applied moment to the cantilevered section of the wingwall)*

$(M_u)_{wing} := M_{cant}$

$(V_u)_{wing} := \frac{\overrightarrow{(w_{lat})_{wing} \cdot L_{cant}}}{2}$ *(maximum applied shear to the cantilevered section of the wingwall)*

Wingwall Concrete Design

Stem Wall Layout and Design

								Resultant Loads			Vertica	
Structure Name	Location	River Station	Flowline Elevation Ft	Channel Bottom Elevation Ft	Pipe Size in	Pipe Wall in	Top of Outlet Wall Ft	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rxn [kips]	Vert Bar Size [size]	Vert Bar Size [in]
ST-02A	LB	2+75	923.17	922.15	24	4	926.67	0.621	0.5	0.6	5	0.6
ST-03A	LB	5+80	923.9	922	24	4	927.4	0.621	0.5	0.6	5	0.6
ST-19A	LB	36+60	897.06	894.5	48	8	902.56	1.278	1.5	1.3	5	0.6
ST-22A	RB	39+25	898.68	895	24	4	902.18	0.621	0.5	0.6	5	0.6
ST-26A	RB	41+80	899.11	896.2	24	4	902.61	0.621	0.5	0.6	5	0.6
ST-43A	RB	62+50	893.12	888.2	24	4	896.62	0.621	0.5	0.6	5	0.6
ST-45A	RB	64+70	896.67	891	24	4	900.17	0.621	0.5	0.6	5	0.6
ST-45B	LB	64+85	898.87	890.5	24	4	902.37	0.621	0.5	0.6	5	0.6

$Cover_{wing} := cover \cdot in$ *(cover for vertical bars in wingwalls, from table above)*

$(b_s)_{wing} := (b_s)_{wing} \cdot in$ *(bar size for vertical bars in wing walls, from table above)*

$(s_{vert})_{wing} := (s_{vert})_{wing} \cdot in$ *(bar spacing for vertical bars in wing walls, from table above)*

$(A_s)_{wing} := \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left((b_s)_{wing}\right)$ *(area of steel for vertical bars in wing walls)*

$$= \begin{bmatrix} 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \\ \vdots \end{bmatrix} in^2$$

if $(s_{vert})_{wing_i} = 0$

$as_i \leftarrow \frac{0.25 \cdot \pi \cdot (b_s)_{wing_i}^{\ 2}}{1}$

else

$as_i \leftarrow \frac{0.25 \cdot \pi \cdot (b_s)_{wing_i}^{\ 2} \cdot b_i}{\overrightarrow{\left((s_{vert})_{stem_i}\right)}}$

as

$d_{wing} := D_{outfall} - Cover_{wing} - \left(0.5 \cdot (b_s)_{wing}\right)$ *(concrete cover for reinforcing bars in wingwalls)*

$(d_{wing})_{Horz} := D_{outfall} - Cover_{wing} - \left(1.5 \cdot (b_s)_{wing}\right)$ *(concrete cover for horizontal reinforcing bars in wingwalls)*

$a \cdot 0.85 \cdot f'c \cdot b = (A_s)_{wing} \cdot f_y$ *(concrete equilibrium, ACI 318-19, section 22.2)*

$$(\beta_1)_{wing} := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.8$$

(beta value from ACI 318-19, Table 22.2.2.4.3)

$c = \left(\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_t}\right) \cdot d_{wing}$ *(neutral axis depth for beams)*

$c_{wing} := \left(\frac{0.003}{0.003 + 0.004}\right) \cdot d_{wing}$

$a_{wing} := (\beta_1)_{wing} \cdot c_{wing}$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1)*

$\overrightarrow{(M_n)_{wing} := \phi_b \cdot (A_s)_{wing} \cdot f_y \cdot \left(d_{wing} - \frac{a_{wing}}{2}\right)}$ *(nominal flexural strength, ACI 318-19)*

$Flex_{nom} := Check\left((M_u)_{wing}, (M_n)_{wing}\right)$ *(nominal flexural check)*

Flexural Check

			Resultant Loads			Vertical Rebar				Vingwall Flexural Check		
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rxn [kips]	Vert Bar Size [size]	Vert Bar Size [in]	Vert bar Spacing [in]	Conc Cover [in]	Mlat [kip-ft]	ϕMn [kip-ft]	Flexural Check
ST-02A	LB	2+75	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK
ST-03A	LB	5+80	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK
ST-19A	LB	36+60	4.902	9.4	1.6	6	0.8	12.0	3.0	1.5	5.3	OK
ST-22A	RB	39+25	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK
ST-26A	RB	41+80	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK
ST-43A	RB	62+50	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK
ST-45A	RB	64+70	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK
ST-45B	LB	64+85	5.279	11.1	1.8	6	0.8	12.0	3.0	0.5	5.3	OK

minimum required steel for flexure

$$K_u := \text{for } i \in 1 .. \text{ORIGIN} \begin{cases} 1 - \sqrt{1 - \frac{(M_u)_{wing}}{\phi_b}}{1} & \text{if } b_i = 0 \\ 1 - \sqrt{1 - \frac{(M_u)_{wing}}{\phi_b}}{\overrightarrow{0.425 \cdot f'_c \cdot b_i \cdot d_{wing_i}^2}} & \text{else} \end{cases}$$

(ratio of reinforcement stress block depth)

$\overrightarrow{(A_{s.min})_{ten} := \frac{0.85 \cdot f'_c \cdot K_u \cdot b \cdot d_{wing}}{f_y}}$ *(minimum area of steel required for flexural strength)*

minimum required steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot \overrightarrow{b \cdot d_{wing}}$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{f_y} \cdot \overrightarrow{b \cdot d_{wing}}$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.003 \cdot \left(\frac{\overrightarrow{D_{outfall} \cdot L_{cant}}}{144} \right)$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

$A_{s.min} := MinA_s\left((A_{s.min})_{ten}, (A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s\_1.3} := (A_{s.min})_{ten} \cdot \frac{4}{3}$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$A_{s.req} := MaxLC_2\left(A_{s\_1.3}, A_{s.min}\right)$ *(required steel to meet steel requirements)*

$Steel\_Check := Check\left(A_{s.req}, (A_s)_{stem}\right)$ *(check stem reinforcement to required reinforcement)*

Minimum Steel Check

			Vertical Rebar		Minimum Steel - Wingwall							
Structure Name	Location	River Station	Vert Bar Size [size]	Vert bar Spacing [in]	As, min Flex [in2]	As, min 1 [in2]	As, min 2 [in2]	As, min temp [in2]	As, min [in2]	As, req [in2]	As, wing [in2]	Steel Check
ST-02A	LB	2+75	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
ST-03A	LB	5+80	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
ST-19A	LB	36+60	5	10.0	0.07	0.19	0.19	0.01	0.19	0.2	0.3	OK
ST-22A	RB	39+25	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
ST-26A	RB	41+80	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
ST-43A	RB	62+50	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
ST-45A	RB	64+70	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
ST-45B	LB	64+85	5	10.0	0.02	0.19	0.19	0.00	0.19	0.2	0.3	OK
0	0	0+00	0	0.0	0.00	0.00	0.00	0.00	0.00	0.0	0.0	OK

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max.wing} := \text{for } i \in 1 .. \text{last}\left((A_s)_{wing}\right)$ *(reinforcement ratio from EM 1110-2-2104 D-3)*

$\quad \rho_{max_i} \leftarrow 0.5 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \right) \cdot \left(\frac{87000}{87000 + \left(f_y \cdot psi^{-1}\right)} \right)$

$\quad \rho_{max}$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual.wing} := \text{for } i \in 1 .. \text{last}\left((A_s)_{wing}\right)$ *(actual reinforcement ratio for concrete section)*

$\quad \text{if } b_i = 0$

$\quad\quad \rho_i \leftarrow \frac{(A_s)_{wing_i} \cdot ft^{-2}}{1}$

$\quad \text{else}$

$\quad\quad \rho_i \leftarrow \frac{(A_s)_{wing_i}}{\left(b_i \cdot d_{wing_i}\right)}$

$\quad \rho$

$\rho\_check := Check\left(\rho_{actual.wing}, \rho_{max.wing}\right)$

Reinforcement Ratio Check

			Vertical Rebar				Reinforcement Ratio - Wing		
Structure Name	Location	River Station	Vert Bar Size [size]	Vert Bar Size [in]	Vert bar Spacing [in]	Conc Cover [in]	ρ.. max	ρ.. act	ρ.. check
ST-02A	LB	2+75	6	0.8	12.0	3.0	0.016	0.005	OK
ST-03A	LB	5+80	6	0.8	12.0	3.0	0.016	0.005	OK
ST-19A	LB	36+60	6	0.8	12.0	3.0	0.016	0.005	OK
ST-22A	RB	39+25	6	0.8	12.0	3.0	0.016	0.005	OK
ST-26A	RB	41+80	6	0.8	12.0	3.0	0.016	0.005	OK
ST-43A	RB	62+50	6	0.8	12.0	3.0	0.016	0.005	OK
ST-45A	RB	64+70	6	0.8	12.0	3.0	0.016	0.005	OK
ST-45B	LB	64+85	6	0.8	12.0	3.0	0.016	0.005	OK

Shear Reinforcement - Wing

$\mu := 0.6\ \lambda_c$ *(coefficient of friction, ACI 318-19, Table 22.9.4.2)*

$(A_{vf})_{wing} := (A_s)_{wing} - A_{s.req}$ *(area of steel on the compression face of the stem, used for shear reinforcement)*

$V_{n.wing} := \mu \cdot (A_{vf})_{wing} \cdot f_y$ *(shear friction capacity from reinforcement, ACI 318-19, Eq 22.9.4.2)*

$\phi V_{n.wing} := \phi_V \cdot V_{n.wing}$ *(factored shear friction capacity from reinforcement)*

$Vn\_check := Check\left((V_u)_{wing}, \phi V_{n.wing}\right)$ *(check shear capacity of reinforcement)*

Shear Friction Check

			Resultant Loads			Vertical Rebar		Shear Reinf - Wingwall	
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rsn [kips]	Vert Bar Size [size]	Vert bar Spacing [in]	ϕVn.wing [kips]	Check
ST-02A	LB	2+75	0.621	0.5	0.6	5	10.0	3.2	OK
ST-03A	LB	5+80	0.621	0.5	0.6	5	10.0	3.2	OK
ST-19A	LB	36+60	1.278	1.5	1.3	5	10.0	3.2	OK
ST-22A	RB	39+25	0.621	0.5	0.6	5	10.0	3.2	OK
ST-26A	RB	41+80	0.621	0.5	0.6	5	10.0	3.2	OK
ST-43A	RB	62+50	0.621	0.5	0.6	5	10.0	3.2	OK
ST-45A	RB	64+70	0.621	0.5	0.6	5	10.0	3.2	OK

Concrete Shear Capacity - Wingwall

$$\lambda_{s.1} := \sqrt{\frac{2}{1 + \frac{d_{wing}}{10\ in}}}$$ *(size modification factor ACI 318-19, Eq. 22.5.5.1.3)*

$\lambda_s := \text{for } i \in 1 .. \text{last}\left(\lambda_{s.1}\right)$ *(size modification factor, limited to 1.0 ACI 318-19, Eq. 22.5.5.1.3)*

$\quad \lambda_{s_i} \leftarrow min\left(\lambda_{s.1_i}, 1.0\right)$

$\quad \lambda_s$

$(N_u/6A_g)_{max} := 0.05 \cdot f'_c$ *(limit of Nu/6Ag ACI 318-19, Section 22.5.5.1.2)*

$(N_u/6A_g)_{act} := \text{for } i \in \mathbf{ORIGIN} .. \text{last}\left((W_{strt})_{w.w}\right)$ *(actual pressure for shear capacity)*

$\quad \text{if } b_i = 0$

$\quad\quad \omega_i \leftarrow \frac{(W_{strt})_{w.w_i}}{1 \cdot in^2}$

$\quad \text{else}$

$\quad\quad \omega_i \leftarrow \frac{(W_{strt})_{w.w_i}}{6 \cdot L_{cant_i} \cdot b_i}$

$\quad \omega$

$A_{v.min} := \text{for } i \in \text{ORIGIN} .. \text{last}(b)$
$\quad A_i \leftarrow \max\left(0.75\ ft \cdot \sqrt{f'_c \cdot psi} \cdot \frac{b_i}{f_y}, \frac{\left(50\ \frac{lbf}{in}\right) \cdot b_i}{f_y}\right)$
$\quad A$

(minimum shear reinforcement, ACI 318-19,Table 9.6.3.1)

$\rho_w := \text{for } i \in \text{ORIGIN} .. \text{last}(b)$
$\quad \text{if } L_{outfall_i} = 0$
$\quad\quad \rho_i \leftarrow 0$
$\quad \text{else}$
$\quad\quad \rho_i \leftarrow \frac{(A_s)_{wing_i}}{b_i \cdot d_{wing_i}}$
$\quad \rho$

(ratio of As to bwd, ACI 318-19, Ch. 2-Notation)

$(V_c)_a := \overrightarrow{\left(2 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} + (N_u/6A_g)_{act}\right) \cdot b \cdot d_{wing}}$

(concrete shear capacity (a), ACI 318-19, Table 22.5.5.1)

$(V_c)_c := \overrightarrow{\left(8 \cdot \lambda_s \cdot \lambda_c \cdot (\rho_w)^{0.33} \cdot \sqrt{f'_c \cdot psi} + (N_u/6A_g)_{act}\right) \cdot b \cdot d_{wing}}$

$(V_c)_{Max} := 5 \cdot \lambda_c \cdot \sqrt{f'_c \cdot psi} \cdot \overrightarrow{b \cdot d_{wing}}$

(maximum permissible value of concrete shear, ACI 318-19, Section 22.5.5.1.1)

$V_c := \text{for } i \in \text{ORIGIN} .. \text{last}(b)$
$\quad \text{if } A_{v.min_i} \le (A_{vf})_{wing_i}$
$\quad\quad V_i \leftarrow (V_c)_{a_i}$
$\quad \text{else}$
$\quad\quad V_i \leftarrow (V_c)_{c_i}$
$\quad V$

(unfactored concrete shear capacity, ACI 318-19, Table 22.5.5.1)

$V_c := \text{for } i \in \text{ORIGIN} .. \text{last}(V_c)$
$\quad V_i \leftarrow min\left(V_{c_i}, (V_c)_{Max_i}\right)$
$\quad V$

(unfactored concrete shear capacity, caped at maximum, ACI 318-19, Table 22.5.5.1)

$\phi V_{c.wing} := \phi_V \cdot V_c$

(nominal shear strength of concrete)

$Vc\_Check := Check\left((V_u)_{wing}, \phi V_{c.wing}\right)$

Concrete Shear Capacity

			Resultant Loads			Concrete Shear - Wingwall			
Structure Name	Location	River Station	Res Lat Loading [kips]	Res Mom [kip-ft]	Res Rzn [kips]	ϕVa [kips]	ϕVc [kips]	ϕVc, calc [kips]	ϕVc, max [kips]
ST-02A	LB	2+75	0.621	0.5	0.6	7.6	5.5	5.5	18.9
ST-03A	LB	5+80	0.621	0.5	0.6	7.6	5.5	5.5	18.9
ST-19A	LB	36+60	1.278	1.5	1.3	7.6	5.5	5.5	18.9
ST-22A	RB	39+25	0.621	0.5	0.6	7.6	5.5	5.5	18.9
ST-26A	RB	41+80	0.621	0.5	0.6	7.6	5.5	5.5	18.9
ST-43A	RB	62+50	0.621	0.5	0.6	7.6	5.5	5.5	18.9
ST-45A	RB	64+70	0.621	0.5	0.6	7.6	5.5	5.5	18.9

Note: Stirrups are not required to support longitudinal bars in stem wall. Wingwall bars will provide lateral containment.

U.S.ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Triple Box Culvert Headwall at Quaker Creek

Upper Turkey Creek

MERRIAM, KS

ASSUMPTIONS

- Serviceability Loading conditions and single-load factors per EM 1110-2-2104, ASCE 7-16 where EM 2104 is silent
- Reference 'UTC Floodwall Justification Calcs' for justification on using service level single load factor analysis in lieu of multi-factored strength design for T-wall sections
- Assumption that the tallest wall section with the largest hydraulic loading will govern design, all other shorter sections of wall stem will have smaller loading, thus are ok.
- Both Drained and Undrained loading cases are used for analysis, for each of the sections of wall the largest of the loading cases is taken to produce the most conservative loading analysis.

MATERIAL PROPERTIES AND LOAD FACTORS

Soil and Water Properties

$\gamma_{water} := 62.4\ pcf$ *(unit weight of water)*

$\gamma_{soil} := 120\ pcf$ *(unit weight of saturated soil)*

$\gamma'_{soil} := \gamma_{soil} - \gamma_{water} = 57.6\ pcf$ *(buoyant unit weight of soil)*

$q_n := 2000\ psi$ *(bearing pressure, Reference DDR Table 9.2)*

Drained Soil Conditions

$\phi_{drained} := 28\ deg$ *(internal friction angle for drained soil condition)*

$C_{drained} := 0\ psf$ *(cohesion for drained soil condition)*

$k_0 := 1 - \sin\left(\phi_{drained}\right) = 0.531$ *(at-rest soil pressure coefficient for the drained condition)*

Steel Properties

$\gamma_{steel} := 490\ pcf$ *(unit weight of steel)*

$E_{steel} := 29000\ ksi$ *(young's modulus of elasticity for steel)*

$(F_y)_{30} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$(F_y)_{60} := 60\ ksi$ *(yield strength of 60 ksi steel)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.0021$ *(yield strain of 60 ksi steel)*

Concrete Properties

$f'_c := 4500\ psi$ *(concrete compressive strength for all structures)*

$f'c := f'_c \cdot psi^{-1}$ *(unitless concrete compressive strength for all structures)*

$\lambda := 1.0$ *(concrete modification factor)*

$cover_3 := 3\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$cover_4 := 4\ in$ *(concrete cover for reinforcement; EM 1110-2-2104, Table 2-1)*

$\gamma_{conc} := 150\ pcf$ *(concrete unit weight)*

$Ec := 57000 \cdot \sqrt{f'c} \cdot psi = 3823.7\ ksi$ *(young's modulus of elasticity for concrete)*

$n := \frac{E_{steel}}{Ec} = 7.6$ *(transformed section ratio)*

$\varepsilon_c := 0.003$ *(yield strain in concrete in compression)*

$\beta_1 := \begin{cases} 0.65 & \text{if } f'_c \geq 8000\ psi \\ 0.85 - \left(\frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) & \text{else if } f'_c > 4000\ psi \\ 0.85 & \text{else} \end{cases} = 0.825$ *(beta value from ACI 318-19, Table 22.2.2.4.3)*

Load Factors

Serviceability Load Factors

Note: the unusual loading condition is used for the top of the floodwalls since they are designed to the 100 year event. the usual loading condition is defined as the 10 year event.

$(\gamma_p)_{Serve.u} := 2.2$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-1)*

$(\gamma_p)_{Serve.n} := 1.6$ *(serviceability load factor for permanent loading, EM 1110-2-2104, Table 3-1)*

Strength Limit States

$\phi_b := 0.9$ *(bending load reduction factor - ACI 318-19 21.2.2, Table 21.2.2)*

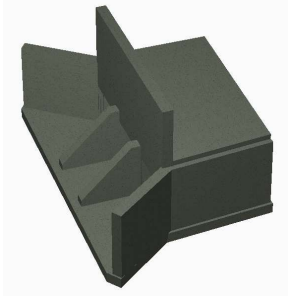
$\phi_A := 0.9$

$\phi_V := 0.75$ *(shear load reduction factor - ACI 318-19 Table 21.2.1)*

$\phi_C := 0.65$ *(compression load reduction factor - ACI 318-19 Table 21.2.2)*

$\phi_T := 0.9$ *(tension load reduction factor - ACI 318-19 Table 21.2.2)*

BOX DIMENSIONS



Headwall

$(Elev_{wall})_{top} := 920.25\ ft$ *(elevation at top of wall)*

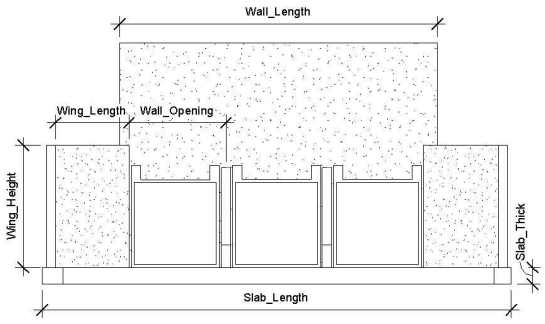
$(Elev_{wall})_{bot} := 900.25\ ft$ *(elevation at bottom of wall)*

$Wall_{height} := (Elev_{wall})_{top} - (Elev_{wall})_{bot}$ *(height of stem wall)*

$Wall_{height} = 20\ ft$

$Wall_{thick} := 18\ in$ *(thickness of wall)*

$Wall_{length} := 28.37\ ft$ *(length of monolith)*



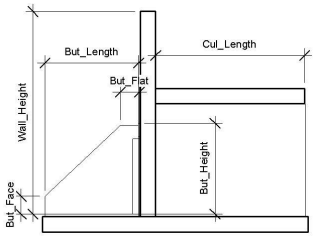
Wingwall

$Wing_{thick} := 18\ in$ *(thickness of wingwall)*

$Wing_{height} := 11\ ft$ *(total height of wingwall, from top of slab)*

$Wing_{length} := 11.5\ ft$ *(length of the wingwall)*

Buttress Wall



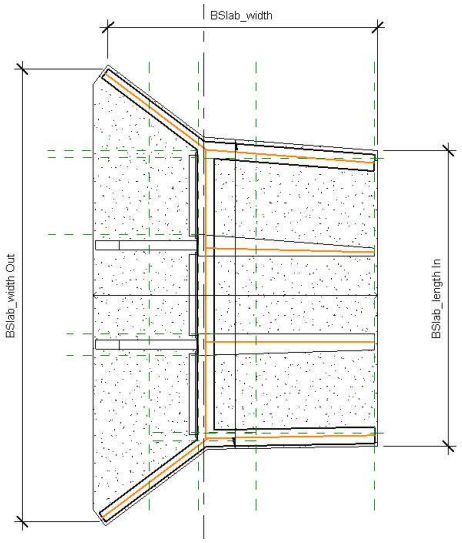
$Buttress_{thick} := 18\ in$ *(thickness of buttress wall)*

$Buttress_{height.front} := 1.5\ ft$ *(height of buttress wall at front face)*

$Buttress_{height.back} := 8.9\ ft$ *(height of buttress wall at back face)*

$Buttress_{length} := 9.33\ ft$ *(length of the wingwall)*

Culvert Walls



$CWall_{thick} := 18\ in$ *(wall thickness)*

$CWall_{height} := 7\ ft$ *(wall height)*

$CWall_{length} := 15\ ft$ *(wall length)*

Top Slab

$TSlab_{thick} := 18\ in$ *(slab thickness)*

$TSlab_{width} := Wall_{length} = 28.4\ ft$ *(wall width)*

$TSlab_{length} := CWall_{length} = 15\ ft$ *(wall length)*

Bottom Slab

$BSlab_{thick} := 18\ in$ *(slab thickness)*

$BSlab_{width} := 24.5\ ft$ *(slab width)*

$BSlab_{length.in} := 26.85\ ft$ *(slab length, measured at the back of the slab)*

$BSlab_{length.out} := 41.25\ ft$ *(slab length, measured at the front of the slab)*

LOADS

Permanent Loads

Dead Loads

Note: dead loads are calculated as the force per unit length of the member per foot of depth of the member.

$D_{stem} := \gamma_{conc} \cdot (Wall_{thick} \cdot 1\ ft) = 225\ plf$ *(self weight of concrete, per foot of depth per foot of wall length)*

$D_{wing} := \gamma_{conc} \cdot (Wing_{thick} \cdot 1\ ft) = 225\ plf$ *(self weight of concrete, per foot of depth per foot of wall length)*

$D_{but} := \gamma_{conc} \cdot (Buttress_{thick} \cdot 1\ ft) = 225\ plf$ *(self weight of concrete, per foot of depth per foot of buttress length, conservatively uses the width at bottom of wall)*

$D_{cwall} := \gamma_{conc} \cdot (CWall_{thick} \cdot 1\ ft) = 225\ plf$ *(self weight of concrete, per foot of depth per foot of wall length)*

Vertical Earth Loads

$Elev_{soil} := 912\ ft$ *(soil elevation at resisting side, on top of box)*

$Elev_{box.top} := (Elev_{wall})_{bot} + CWall_{height} + TSlab_{thick} = 908.8\ ft$ *(elevation of the box top)*

$d_{soil} := Elev_{soil} - Elev_{box.top} = 3.2\ ft$ *(soil cover at resisting side of box)*

$\sigma_{EV} := \gamma_{soil} \cdot d_{soil} = 0.39\ ksf$ *(vertical soil pressure at the top slab of box)*

Lateral Earth Loads to Back of Stem Wall

$d_{soil} = 3.2\ ft$ *(soil cover at resisting side of box)*

$k_0 = 0.5$ *(at-rest lateral earth pressure coefficient)*

$\sigma_{EH.r} := k_0 \cdot \gamma_{soil} \cdot d_{soil} = 0.207\ ksf$ *(maximum lateral earth pressure of soil on resisting side)*

Lateral Earth Loads to Wingwalls

$(d_{soil})_{wingwall} := CWall_{height} + d_{soil} + TSlab_{thick} = 11.7\ ft$ *(depth of soil on the wingwall)*

$k_0 = 0.5$ *(at-rest lateral earth pressure coefficient)*

$\sigma_{EH.wing} := k_0 \cdot \gamma_{soil} \cdot (d_{soil})_{wingwall} = 0.748\ ksf$ *(maximum lateral earth pressure of soil on wingwalls)*

Lateral Earth Loads on Exterior Box Walls

$(d_{soil})_{wall.top} := d_{soil} + TSlab_{thick} = 4.7\ ft$ *(depth of soil on the exterior walls, at top)*

$(d_{soil})_{wall.bot} := (d_{soil})_{wingwall} = 11.7\ ft$ *(depth of soil on the exterior walls, at bottom)*

$\sigma_{EH.wall.top} := k_0 \cdot \gamma_{soil} \cdot (d_{soil})_{wall.top} = 0.302\ ksf$ *(lateral soil pressure at the top of the exterior wall)*

$\sigma_{EH.wall.bot} := k_0 \cdot \gamma_{soil} \cdot (d_{soil})_{wall.bot} = 0.748\ ksf$ *(lateral soil pressure at the bottom of the exterior wall)*

Gravity Loads - Silt

Note: assume that silt is slightly more dense than the native soil, based on reduced voids from being saturated and then settling in place. Silt in loads placed on both the front face of the slab as well as the upstream end of the slab; inside the conduit of the culvert.

$\gamma_{silt} := \gamma_{soil} \cdot 1.2 = 144\ pcf$ *(unit density of silt applied to structure)*

$Silt\_in\_depth := 4\ in$ *(assumed silting depth)*

$\sigma_{Gravity} := \gamma_{silt} \cdot Silt\_in\_depth = 0.048\ ksf$ *(pressure loading to slab)*

Pavement Loading

$\gamma_{asphalt} := 145\ pcf$ *(unit density of asphalt pavement)*

$depth_{roadway} := 12\ in$ *(depth of roadway section)*

$width_{roadway} := 15\ ft$ *(width of the roadway affecting box)*

$b := 12\ in$ *(analysis width)*

$P := \gamma_{asphalt} \cdot depth_{roadway} \cdot width_{roadway} \cdot b = 2.2\ kip$ *(effective point load from pavement above box)*

$D_1 := 2 \cdot (d_{soil} \cdot 0.5) + width_{roadway} = 18.2\ ft$ *(effective loading area from roadway, AASHTO LRFD Figure 3.11.6.3-1)*

$\Delta\sigma_v := \frac{P}{D_1 \cdot b} = 0.1\ ksf$ *(effective pressure from roadway, AASHTO LRFD Figure 3.11.6.3-1)*

Note: Surcharge only applied to the top slab of the box and not applied laterally to the wall as the soil depth does not allow for the lateral surcharge force in the wall to develop. soil is at a depth of $d_{soil} = 3.2\ ft$ and the wall is approximately 10 ft from the back of the curb.

Temporary Loading

Hydrostatic Loading

$\sigma_{Hs.bot} := \gamma_{water} \cdot Wall_{height} = 1.25\ ksf$ *(unit loading to the bottom of the box wall, loading to top face of bottom slab)*

$\sigma_{Hs.top.r} := \gamma_{water} \cdot d_{soil} = 0.203\ ksf$ *(back of stem wall due to hydrostatic pressure in resisting side soil)*

$\sigma_{Hs.cond} := \gamma_{water} \cdot CWall_{height} = 0.437\ ksf$ *(hydrostatic loading to conduit and back face of stem wall due to water inside conduit)*

$\sigma_{Hs.wingwall} := \gamma_{water} \cdot Wall_{height} = 1.25\ ksf$ *(hydrostatic loading to wingwall)*

Vehicle Live Load

$LLDF := 1.15$ *(LRFD live load distribution factor, LRFD Table 3.6.1.2.6a-1)*

$s_w := 6\ ft$ *(wheel spacing, AASHTO 3.6.1.2.6b)*

$s_a := 8\ ft$ *(axel spacing, AASHTO 3.6.1.2.6b)*

$w_t := 20\ in$ *(tire patch width, AASHTO 3.6.1.2.6b)*

$l_t := 10\ in$ *(tire patch length, AASHTO 3.6.1.2.6b)*

$D_i := 7\ ft = 84\ in$ *(inside clear diameter of culvert, AASHTO 3.6.1.2.6b)*

$$H_{int} := \frac{s_w - w_t - 0.06\ D_i}{LLDF} = 3.4\ ft$$ *(load interaction depth transverse, AASHTO eq. 3.6.1.2.6b-1)*

(load interaction depth, AASHTO eq. 3.6.1.2.6b-2 and 3)

$$w_w := \begin{cases} \text{if } d_{soil} < H_{int} \\ \quad w_t + \left(LLDF \cdot d_{soil}\right) + 0.06 \cdot \left(D_i\right) \\ \text{else} \\ \quad w_t + s_w + \left(LLDF \cdot d_{soil}\right) + 0.006 \cdot \left(D_i\right) \end{cases} = 5.8\ ft$$

$$H_{int\_p} := \frac{s_a - l_t}{LLDF} = 6.2\ ft$$ *(load interaction depth, AASHTO eq. 3.6.1.2.6b-4)*

$$l_w := \begin{cases} \text{if } d_{soil} < H_{int\_p} \\ \quad l_t + LLDF \cdot d_{soil} \\ \text{else} \\ \quad l_t + s_a + LLDF \cdot d_{soil} \end{cases} = 4.6\ ft$$ *(live load patch length, AASHTO eq. 3.6.1.2.6b-5 and 6)*

$A_{LL} := l_w \cdot w_w = 26.6\ ft^2$ *(rectangular area of applied load, AASHTO eq. 3.6.1.2.6a-1)*

$D_E := d_{soil} \cdot ft^{-1} = 3.2$ *(minimum depth of earth cover above the structure, 3.6.2.2)*

$IM := \max\left(\left(33 \cdot \left(1.0 - 0.125 \cdot D_E\right)\right), 0\right) = 19.6$ *(dynamic load allowance, AASHTO eq. 3.6.2.2-1)*

$m := 1.0$ *(multiple presence factor, AASHTO table 3.6.1.1.2-1)*

$P := 32\ kip$ *(live load applied at surface, AASHTO eq. 3.6.1.2.6b-7)*

$$P_L := \frac{P \cdot \left(1 + \frac{IM}{100}\right) \cdot m}{A_{LL}} = 1.438\ ksf$$ *(applied live vehicle load at top slab surface, AASHTO eq. 3.6.1.2.6b-7)*

Dynamic Loads

Impact Loads

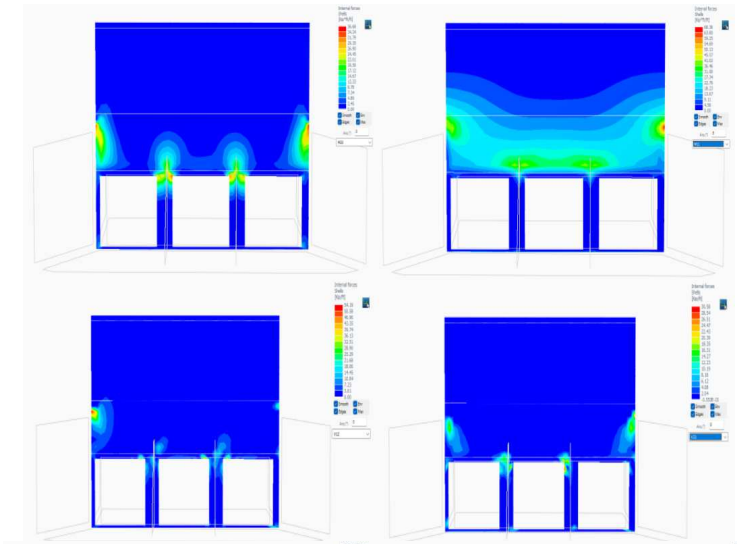
Note: Assume that impact loads are concentrated at the top 6 inches of the wall.

$IM := 0.1\ klf$ *(applied impact loading, reference DDR)*

$depth := 0.5\ ft$ *(height of applied impact loading)*

$$F_{IM} := \frac{IM}{depth} = 0.2\ ksf$$ *(impact loading in model, applied to structure)*

RAM ELEMENTS OUTPUTS



Stem Wall

$(M_{33})_{stem} := 36.68\ kip \cdot ft$

(max vertical moment in stem)

$(M_{11})_{stem} := 36.46\ kip \cdot ft$

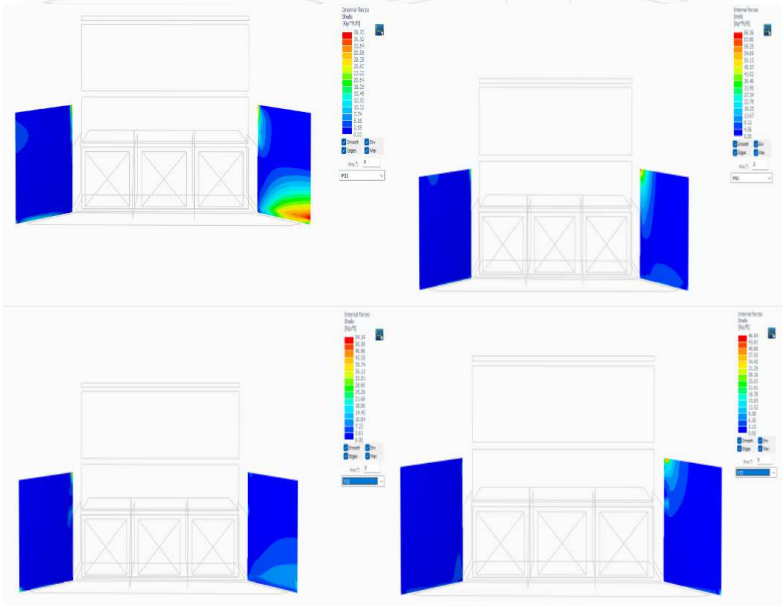
(max lateral moment in stem, taken to outside of wingwall connection)

$(V_{12})_{stem} := 54.19\ kip$

(max vertical shear in stem)

$(V_{23})_{stem} := 30.58\ kip$

(max lateral shear in stem)



Wingwalls

$(M_{33})_{wing} := 38.70\ kip \cdot ft$

(max vertical moment in wing)

$(M_{11})_{wing} := 41.02\ kip \cdot ft$

(max lateral moment in wing, taken to outside of wingwall connection)

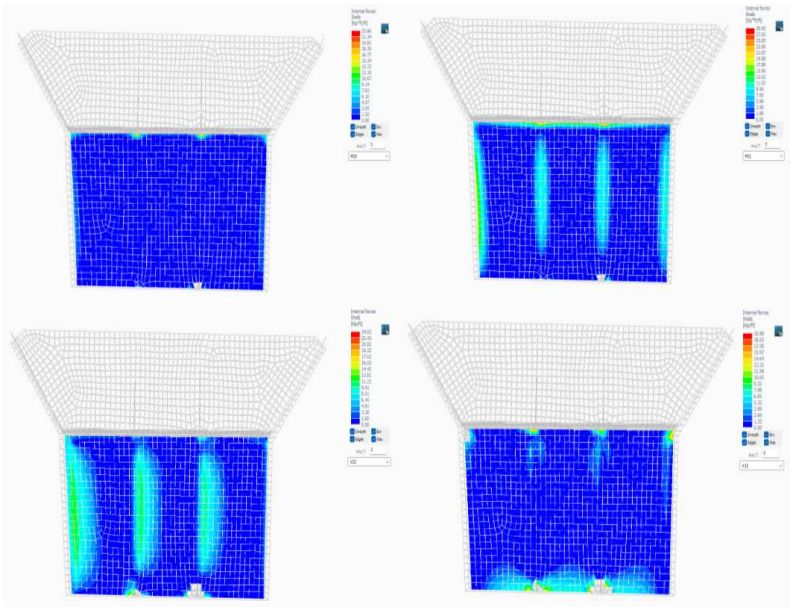
$(V_{12})_{wing} := 28.90\ kip$

(max vertical shear in wing)

$(V_{23})_{wing} := 28.16\ kip$

(max lateral shear in wing)

Note: Unusually large shear stresses located above interior walls are transferred into the wall sections, and the shear is taken a distance of approximately 12" away from the centerline of the wall.



Top Slab

$(M_{11})_{topslab} := 23.86\ kip \cdot ft$

(max longitudinal moment in slab)

$(M_{33})_{topslab} := 22.86\ kip \cdot ft$

(max lateral moment in slab)

$(V_{23})_{topslab} := 24.03\ kip$

(max longitudinal shear in slab)

$(V_{12})_{topslab} := 19.96\ kip$

(max lateral shear in slab)

Note: Unusually large shear stresses located above interior walls are transferred into the wall sections, and the shear is taken a distance of approximately 12" away from the centerline of the wall.

Box Walls

$(M_{33})_{wall} := 37.41\ kip \cdot ft$

(max vertical moment in wall)

$(M_{11})_{wall} := 23.14\ kip \cdot ft$

(max lateral moment in wall)

$(V_{12})_{wall} := 22.45\ kip$

(max compressive force in wall)

$(V_{23})_{wall} := 21.23\ kip$

(max lateral shear in wall)

Note: high shear values calculated at the connections are interpreted to the outside face of the wall element for analysis.

Bottom Slab

$(M_{11})_{botslab} := 41.27\ kip \cdot ft$

(max longitudinal moment in slab)

$(M_{33})_{botslab} := 39.84\ kip \cdot ft$

(max lateral moment in slab)

$(V_{23})_{botslab} := 69.72\ kip$

(max longitudinal shear in slab)

$(V_{12})_{botslab} := 49.36\ kip$

(max lateral shear in slab)

Buttress Walls

$(M_{33})_{but} := 12.55\ kip \cdot ft$

(max vertical moment in buttress)

$(M_{11})_{but} := 19.18\ kip \cdot ft$

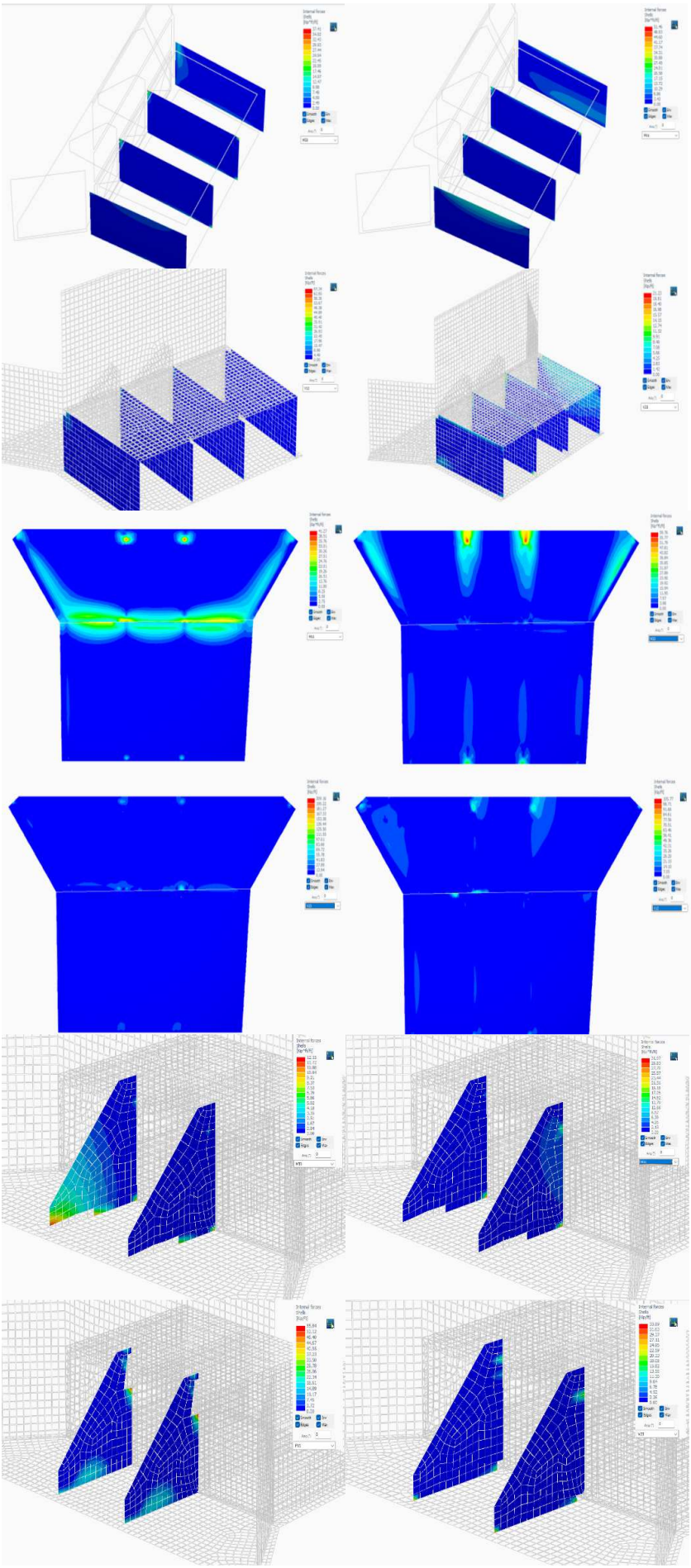
(max lateral moment in buttress)

$(F_{11})_{but} := 48.40\ kip$

(max compression in buttress)

$(V_{23})_{but} := 9.04\ kip$

(max lateral shear in buttress)



(user defined function to check the design capacity against the required capacity, function for use with comparing arrays)

$$Check_v(X_U, X_N) := \left| \begin{array}{l} \text{for } i \in \mathbf{ORIGIN} .. \text{last}(X_U) \\ \quad \left| \begin{array}{l} \text{if } X_{U_i} > X_{N_i} \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“NO GOOD”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \\ \text{else if } X_{N_i} = 0 \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“N/A”} \end{array} \right. \\ \text{else} \\ \quad \left| \begin{array}{l} ch_{i,1} \leftarrow \text{“OK”} \\ ch_{i,2} \leftarrow \dfrac{X_{U_i}}{X_{N_i}} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

(user defined function to check the design capacity against the required capacity, function for use with comparing scalar values)

$$Check(X_U, X_N) := \left| \begin{array}{l} \left| \begin{array}{l} \text{if } X_U > X_N \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“NO GOOD”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \\ \text{else if } X_U = 0 \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“N/A”} \end{array} \right. \\ \text{else} \\ \quad \left| \begin{array}{l} ch_{1,1} \leftarrow \text{“OK”} \\ ch_{1,2} \leftarrow \dfrac{X_U}{X_N} \end{array} \right. \end{array} \right. \\ ch \end{array} \right.$$

(if the required is greater than the capacity)

(if the capacity is zero, used for error handling)

(if the required is less than the capacity)

STEM WALL DESIGN

Stem Reinforcement

Vertical Tension Bars Front Face

$(d_b)_{stem.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in stem)*

$s_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c_{stem.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in stem)*

$$(A_s)_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.vert}^{\ 2} \cdot b}{s_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$$d_{stem.vert} := Wall_{thick} - c_{stem.vert} - \left(0.5 \cdot (d_b)_{stem.vert}\right) = 14.5\ in$$ *(depth to reinforcement vertical reinforcement in stem)*

Vertical Tension Bars Back Face

$(d_b)'_{stem.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in stem)*

$s'_{stem.vert} := 12\ in$ *(vertical bar spacing in stem)*

$c'_{stem.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in stem)*

$$(A_s')_{stem.vert} := \frac{0.25 \cdot \pi \cdot (d_b)'_{stem.vert}^{\ 2} \cdot b}{s'_{stem.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in stem)*

$$d'_{stem.vert} := Wall_{thick} - c'_{stem.vert} - \left(0.5 \cdot (d_b)'_{stem.vert}\right) = 14.5\ in$$ *(depth to reinforcement vertical reinforcement in stem)*

Lateral Tension Bars Front Face

$(d_b)_{stem.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in stem)*

$s_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$c_{stem.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in stem)*

$$(A_s)_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{stem.lat}^{\ 2} \cdot b}{s_{stem.lat}} = 0.785\ in^2$$ *(area of steel lateral reinforcement in stem)*

$$d_{stem.lat} := Wall_{thick} - c_{stem.vert} - \left((d_b)_{stem.vert}\right) - \left(0.5 \cdot (d_b)_{stem.lat}\right) = 13.5\ in$$ *(depth to reinforcement lateral reinforcement in stem)*

Lateral Tension Bars Back Face

$(d_b)'_{stem.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in stem)*

$s'_{stem.lat} := 12\ in$ *(lateral bar spacing in stem)*

$c'_{stem.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in stem)*

$$(A_s')_{stem.lat} := \frac{0.25 \cdot \pi \cdot (d_b)'_{stem.lat}^{\ 2} \cdot b}{s'_{stem.lat}} = 0.785\ in^2$$ *(area of steel lateral reinforcement in stem)*

$$d'_{stem.lat} := Wall_{thick} - c'_{stem.lat} - (d_b)'_{stem.vert} - \left(0.5 \cdot (d_b)'_{stem.lat}\right) = 13.5\ in$$ *(depth to reinforcement lateral reinforcement in stem)*

$\beta_1 =$

Nominal Flexural Strength

$$a_{stem.vert} := \frac{(A_s)_{stem.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$$a_{stem.lat} := \frac{(A_s)_{stem.lat} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$\varepsilon_{cu} := 0.003$ *(ultimate strain in extreme compression fiber; ACI 318-14 R21.2.2)*

$$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.002$$ *(net tensile strain limit; ACI 318-14 R21.2.2)*

$$c := \frac{a_{stem.vert}}{\beta_1} = 1.24\ in$$

Check Flexural Strength

Vertical Flexural Front Face

$(M_{33})_{stem} = 36.7\ kip \cdot ft$ *(max vertical moment in stem)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c}\right) \cdot \left(d_{stem.vert} - c\right) = 0.032$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f :=$ if $\varepsilon_t \le \varepsilon_{ty}$ $\quad = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$\quad 0.65$

else if $\varepsilon_{ty} < \varepsilon_t < 0.005$

$\quad 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}}$

else

$\quad 0.9$

$(\phi M_n)_{stem.vert} := \phi_f \cdot (A_s)_{stem.vert} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{a_{stem.vert}}{2}\right) = 49.4\ kip \cdot ft$

(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom})_{Stem} := Check\left((M_{33})_{stem}, (\phi M_n)_{stem.vert}\right) = [\text{“OK”} \quad 0.74]$

(nominal flexural check)

Lateral Flexural Front Face

$(M_{11})_{stem} = 36.5\ kip \cdot ft$ *(max vertical moment in stem)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c}\right) \cdot \left(d_{stem.lat} - c\right) = 0.0295$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f :=$ if $\varepsilon_t \le \varepsilon_{ty}$ $\quad = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$\quad 0.65$

else if $\varepsilon_{ty} < \varepsilon_t < 0.005$

$\quad 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}}$

else

$\quad 0.9$

$(\phi M_n)_{stem.lat} := \phi_f \cdot (A_s)_{stem.lat} \cdot (F_y)_{60} \cdot \left(d_{stem.lat} - \frac{a_{stem.lat}}{2}\right) = 45.9\ kip \cdot ft$

(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom.lat})_{Stem} := Check\left((M_{11})_{stem}, (\phi M_n)_{stem.lat}\right) = [\text{“OK”} \quad 0.79]$

(nominal flexural check)

Stem Wall Deflection Check

$c_{cracked} := \frac{-n \cdot (A_s)_{stem.vert} + \sqrt{\left(n \cdot (A_s)_{stem.vert}\right)^2 + \left(2 \cdot n \cdot (A_s)_{stem.vert} \cdot b \cdot d_{stem.vert}\right)}}{b} = 3.3\ in$

(location from the neutral axis to the extreme compression fiber)

$I_{cr} := \frac{1}{3} \cdot b \cdot {c_{cracked}}^3 + n \cdot (A_s)_{stem.vert} \cdot \left(d_{stem.vert} - c_{cracked}\right)^2 = 890.9\ in^4$

(cracked moment of inertia)

$M_a := (M_{33})_{stem} = 36.7\ kip \cdot ft$ *(moment applied to cracked section)*

$I_g := \frac{b \cdot {Wall_{thick}}^3}{12} = 5832\ in^4$ *(gross moment of inertia)*

$M_{cr} := \frac{\left(7.5 \cdot \sqrt{f'c} \cdot psi\right) \cdot I_g}{0.5 \cdot Wall_{thick}} = 27.2\ kip \cdot ft$ *(cracked moment of inertia, ACI 318-14, Eq 24.2.3.5)*

$$I_e := \text{if } M_a \le \left(\frac{2}{3}\cdot M_{cr}\right) \left\| I_g \right\| \text{else} \left\| \frac{I_{cr}}{1-\left(\frac{\frac{2}{3}\cdot M_{cr}}{M_a}\right)^2 \cdot \left(1-\frac{I_{cr}}{I_g}\right)} \right. = 1122.9\ in^4$$

(effective moment of inertia, ACI 318-14, Table 24.2.3.5)

Deflection

From Hydrostatic Loading

$\sigma_{Hs.bot} = 1.25\ ksf$ *(hydrostatic loading to front face of stem wall; at the bottom of the wall)*

$H := Wall_{height} - Wing_{height} = 9\ ft$ *(cantilevered portion of the wall above the wingwalls)*

$\sigma_{Hs.stem} := \frac{H}{Wall_{height}} \cdot \sigma_{Hs.bot} \cdot b = 0.56\ ft \cdot ksf$ *(hydrostatic loading to front face of stem wall; max at the cantilever section)*

$W_{Hs.stem} := 0.5 \cdot \sigma_{Hs.stem} \cdot H = 2.5\ kip$ *(resolved loading from the hydrostatic loading)*

$\Delta_{Hs} := \frac{W_{Hs.stem} \cdot \left(Wall_{height} - Wing_{height}\right)^3}{15 \cdot Ec \cdot I_e} = 0.049\ in$ *(deflections due to hydraulic loading, AISC, 15th Ed Table 3-23 #18)*

From Impact Loading

$IM = 0.1\ klf$ *(impact loading to the top of the wall)*

$W_{IM} := IM \cdot b = 0.1\ kip$ *(loading to the top of the wall section)*

$\Delta_{IM} := \frac{W_{IM} \cdot H^3}{3 \cdot Ec \cdot I_e} = 0.0098\ in$ *(deflection due to impact loading, AISC, 15th Ed, Table 3-23 #22)*

$\Delta := \Delta_{Hs} + \Delta_{IM} = 0.06\ in$ *(total deflection on stem wall)*

Minimum Required Steel for Flexure, Vertical

Given Constraints Values Solver

$As_{req} := 1.5\ in^2$

$(M_{33})_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.vert} - \frac{As_{req} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$

$As_{req.stem} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.577\ in^2$

(nominal flexural strength, EM1110-2-2104, B-49)

$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.577\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.vert} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (Wall_{thick} \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.77\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{stem.req} := \text{if } A_{s\_1.3} < (A_s)_{stem.vert} \ \| \ as \leftarrow (A_{s.min})_{stem.ten}$ $= 0.577\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

$\text{else} \ \| \ as \leftarrow A_{s.min}$

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{stem.vert} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{Vert})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.vert}\right) = [\text{“OK”} \quad 0.7]$

(check stem reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.vert}}{\left(b \cdot d_{stem.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Minimum Required Steel for Flexure, Lateral

Solve Constraints Given Values

$As_{req} := 1.5\ in^2$

$(M_{11})_{stem} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{stem.lat} - \frac{As_{req} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.stem} := \text{find}\left(As_{req}\right) = 0.619\ in^2$

$(A_{s.min})_{stem.ten} := As_{req.stem} = 0.619\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{stem.lat} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{stem.lat} = 0.54\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (Wall_{thick} \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (A_{s.min})_{stem.ten} \cdot \frac{4}{3} = 0.825\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{stem.req} := \text{if } A_{s\_1.3} < (A_s)_{stem.lat} \parallel as \leftarrow (A_{s.min})_{stem.ten}$ else $\parallel as \leftarrow A_{s.min} = 0.543\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{stem.lat} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{lat})_{Stem} := Check\left((A_s)_{stem.req}, (A_s)_{stem.lat}\right) = [\text{"OK"} \quad 0.7]$

(check stem reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{stem.lat}}{\left(b \cdot d_{stem.lat}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{Stem} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Shear Check

$V_u := (V_{12})_{stem} = 54.2\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{stem.vert} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{stem.vert} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{stem.vert} + (A_s')_{stem.vert} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot Wall_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{stem.vert}} = 0.009$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{stem.vert}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \parallel \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{stem.vert}$ else $\parallel \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{stem.vert} = 23.3\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_c := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{stem.vert}\right) = 23.3\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs} := \phi_V \cdot V_c = 17.5\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} :=$ if $V_u > \phi V_{cs}$ "Shear strength from steel required" else "Shear strength from steel not required" = "Shear strength from steel required"

$V_s := \dfrac{A_V \cdot (F_y)_{60} \cdot d_{stem.vert}}{s_{stem.vert}} = 113.9\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu := \phi_V \cdot \left(V_c + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{stem.vert}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_u, Vu\right) = [\text{"OK"} \quad 0.6]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn := \phi_V \cdot \left(V_c + V_s\right) = 102.9\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_u = 54.2\ kip$

$(Check_v)_{Stem} := Check\left(V_u, \phi_V Vn\right) = [\text{"OK"} \quad 0.5]$

Lateral Shear Check

$V_{u.lat} := (V_{23})_{stem} = 30.6\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{stem.lat} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \dfrac{b}{(F_y)_{60}}\right), s_{stem.lat} \cdot \left(50 \cdot psi \cdot \dfrac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{stem.lat} + (A_s')_{stem.lat} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot Wall_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \dfrac{A_V}{b \cdot d_{stem.lat}} = 0.0097$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \dfrac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\dfrac{2}{1 + \dfrac{d_{stem.lat}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{stem.lat} \right.$ else $\left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{stem.lat} \right. = 21.7\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_{c.lat} := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{stem.lat}\right) = 21.7\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs.lat} := \phi_V \cdot V_{c.lat} = 16.3\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_{u.lat} > \phi V_{cs.lat} \left\| \text{“Shear strength from steel required”} \right.$ else $\left\| \text{“Shear strength from steel not required”} \right. = \text{“Shear strength from steel required”}$

$V_{s.lat} := \dfrac{A_V \cdot (F_y)_{60} \cdot d_{stem.lat}}{s_{stem.lat}} = 106\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu_{lat} := \phi_V \cdot \left(V_{c.lat} + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{stem.lat}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_{u.lat}, Vu_{lat}\right) = [\text{“OK”} \quad 0.4]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn_{lat} := \phi_V \cdot \left(V_{c.lat} + V_{s.lat}\right) = 95.8\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_{u.lat} = 30.6\ kip$

$(Check_v)_{Stem.lat} := Check\left(V_{u.lat}, \phi_V Vn_{lat}\right) = [\text{“OK”} \quad 0.3]$

STEM WALL	Bar Size	Spacing (in)
Vertical Bars		
Front Face	#8	12
Back Face	#8	12
Lateral Bars		
Front Face	#8	12
Back Face	#8	12

	Ma	Mn	Check	IC
Vertical Flexural Check	36.46	49.43	OK	0.74
Lateral Flexural Check	36.68	45.90	OK	0.79

	Va	Vn	Check	IC
Vertical Shear Check	54.19	102.92	OK	0.53
Lateral Shear Check	30.58	95.82	OK	0.32

WING WALL DESIGN

Wing Wall Reinforcement

Vertical Tension Bars Front Face

$(d_b)_{wing.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in wing)*

$s_{wing.vert} := 12\ in$ *(vertical bar spacing in wing)*

$c_{wing.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in wing)*

$(A_s)_{wing.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{wing.vert}^{\ 2} \cdot b}{s_{wing.vert}} = 0.785\ in^2$ *(area of steel vertical reinforcement in wing)*

$d_{wing.vert} := Wing_{thick} - c_{wing.vert} - \left(0.5 \cdot (d_b)_{wing.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in wing)*

Vertical Tension Bars Back Face

$(d_b)'_{wing.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in wing)*

$s'_{wing.vert} := 12\ in$ *(vertical bar spacing in wing)*

$c'_{wing.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in wing)*

$(A_s')_{wing.vert} := \frac{0.25 \cdot \pi \cdot (d_b)'_{wing.vert}^{\ 2} \cdot b}{s'_{wing.vert}} = 0.785\ in^2$ *(area of steel vertical reinforcement in wing)*

$d'_{wing.vert} := Wing_{thick} - c'_{wing.vert} - \left(0.5 \cdot (d_b)'_{wing.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in wing)*

Lateral Tension Bars Front Face

$(d_b)_{wing.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in wing)*

$s_{wing.lat} := 12\ in$ *(lateral bar spacing in wing)*

$c_{wing.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in wing)*

$(A_s)_{wing.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{wing.lat}^{\ 2} \cdot b}{s_{wing.lat}} = 0.785\ in^2$ *(area of steel lateral reinforcement in wing)*

$d_{wing.lat} := Wing_{thick} - c_{wing.vert} - \left((d_b)_{wing.vert}\right) - \left(0.5 \cdot (d_b)_{wing.lat}\right) = 13.5\ in$ *(depth to reinforcement lateral reinforcement in wing)*

Lateral Tension Bars Back Face

$(d_b)'_{wing.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in wing)*

$s'_{wing.lat} := 12\ in$ *(lateral bar spacing in wing)*

$c'_{wing.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in wing)*

$(A_s')_{wing.lat} := \frac{0.25 \cdot \pi \cdot (d_b)'_{wing.lat}^{\ 2} \cdot b}{s'_{wing.lat}} = 0.785\ in^2$ *(area of steel lateral reinforcement in wing)*

$d'_{wing.lat} := Wing_{thick} - c'_{wing.lat} - (d_b)'_{wing.vert} - \left(0.5 \cdot (d_b)'_{wing.lat}\right) = 13.5\ in$ *(depth to reinforcement lateral reinforcement in wing)*

Nominal Flexural Strength

$a_{wing.vert} := \frac{(A_s)_{wing.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$a_{wing.lat} := \frac{(A_s)_{wing.lat} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$\varepsilon_{cu} := 0.003$ *(ultimate strain in extreme compression fiber; ACI 318-14 R21.2.2)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.002$ *(net tensile strain limit; ACI 318-14 R21.2.2)*

$c_{wing} := \frac{a_{wing.vert}}{0.85} = 1.2\ in$ *(distance to the neutral axis for wingwall section)*

Check Flexural Strength

Vertical Flexural Front Face

$(M_{33})_{wing} = 38.7\ kip \cdot ft$ *(max vertical moment in wing)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{wing}}\right) \cdot \left(d_{wing.vert} - c_{wing}\right) = 0.033$ *(net tensile strain; ACI 318-14 R21.2.2)*

$$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \dfrac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{else if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{else} \end{cases} = 0.9$$

(strength reduction factor for moment; ACI 318-14 Table 21.2.2)

$(\phi M_n)_{wing.vert} := \phi_f \cdot (A_s)_{wing.vert} \cdot (F_y)_{60} \cdot \left(d_{wing.vert} - \frac{a_{wing.vert}}{2}\right) = 49.4\ kip \cdot ft$

(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom})_{Wing} := Check\left((M_{33})_{wing}, (\phi M_n)_{wing.vert}\right) = [\text{“OK”} \quad 0.78]$

(nominal flexural check)

Lateral Flexural Front Face

$(M_{11})_{wing} = 41\ kip \cdot ft$ *(max lateral moment in wing)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{wing}}\right) \cdot \left(d_{wing.lat} - c_{wing}\right) = 0.031$ *(net tensile strain; ACI 318-14 R21.2.2)*

$$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \dfrac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{else if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{else} \end{cases} = 0.9$$

(strength reduction factor for moment; ACI 318-14 Table 21.2.2)

$(\phi M_n)_{wing.lat} := \phi_f \cdot (A_s)_{wing.lat} \cdot (F_y)_{60} \cdot \left(d_{wing.lat} - \frac{a_{wing.lat}}{2}\right) = 45.9\ kip \cdot ft$

(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom.lat})_{Wing} := Check\left((M_{11})_{wing}, (\phi M_n)_{wing.lat}\right) = [\text{“OK”} \quad 0.89]$

(nominal flexural check)

Wing Wall Deflection Check

$c_{cracked} := \frac{-n \cdot (A_s)_{wing.vert} + \sqrt{\left(n \cdot (A_s)_{wing.vert}\right)^2 + \left(2 \cdot n \cdot (A_s)_{wing.vert} \cdot b \cdot d_{wing.vert}\right)}}{b} = 3.3\ in$

(location from the neutral axis to the extreme compression fiber)

$I_{cr} := \frac{1}{3} \cdot b \cdot {c_{cracked}}^3 + n \cdot (A_s)_{wing.vert} \cdot \left(d_{wing.vert} - c_{cracked}\right)^2 = 890.9\ in^4$

(cracked moment of inertia)

$M_a := (M_{33})_{wing} = 38.7\ kip \cdot ft$ *(moment applied to cracked section)*

$I_g := \frac{b \cdot {Wing_{thick}}^3}{12} = 5832\ in^4$ *(gross moment of inertia)*

$M_{cr} := \frac{\left(7.5 \cdot \sqrt{f'c} \cdot psi\right) \cdot I_g}{0.5 \cdot Wing_{thick}} = 27.2\ kip \cdot ft$ *(cracked moment of inertia, ACI 318-14, Eq 24.2.3.5)*

$$I_e := \text{if } M_a \le \left(\frac{2}{3}\cdot M_{cr}\right) \left\| I_g \right\| \text{else} \left\| \frac{I_{cr}}{1-\left(\frac{\frac{2}{3}\cdot M_{cr}}{M_a}\right)^2 \cdot \left(1-\frac{I_{cr}}{I_g}\right)} \right. = 1093.9\ in^4$$ *(effective moment of inertia, ACI 318-14, Table 24.2.3.5)*

Deflection

From Hydrostatic Loading

$\sigma_{Hs.bot} = 1.25\ ksf$ *(hydrostatic loading to front face of wing wall; at the bottom of the wall)*

$H := Wing_{height} = 11\ ft$ *(cantilevered portion of the wing wall)*

$\sigma_{Hs.wing} := \frac{H}{Wing_{height}} \cdot \sigma_{Hs.bot} \cdot b = 1.25\ ft \cdot ksf$ *(hydrostatic loading to the front face of the wing wall)*

$W_{Hs.wing} := 0.5 \cdot \sigma_{Hs.wing} \cdot H = 6.9\ kip$ *(resolved loading from the hydrostatic loading)*

$\Delta_{Hs} := \frac{W_{Hs.wing} \cdot (Wing_{height})^3}{15 \cdot Ec \cdot I_e} = 0.252\ in$ *(deflections due to hydraulic loading, AISC, 15th Ed Table 3-23 #18)*

$\Delta := \Delta_{Hs} = 0.25\ in$ *(total deflection on stem wall)*

Minimum Required Steel for Flexure, Vertical

Solve Constraints Guess Values

$As_{req} := 1.5\ in^2$

$(M_{33})_{wing} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{wing.vert} - \frac{(A_s)_{wing.vert} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.wing} := \text{find}\left(As_{req}\right) = 0.615\ in^2$

$(A_{s.min})_{wing.ten} := As_{req.wing} = 0.615\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{wing.vert} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{wing.vert} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot \left(Wing_{thick} \cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{wing.ten} \cdot \frac{4}{3} = 0.82\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{wing.req} := \text{if } A_{s\_1.3} < (A_s)_{wing.vert} \; as \leftarrow (A_{s.min})_{wing.ten} \text{ else } as \leftarrow A_{s.min} = 0.584\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{wing.vert} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the wing wall)*

$(Steel\_Check_{Vert})_{Wing} := Check\left((A_s)_{wing.req}, (A_s)_{wing.vert}\right) = [\text{“OK”} \quad 0.7]$

(check wing reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{wing.vert}}{\left(b \cdot d_{wing.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Wing} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Minimum Required Steel for Flexure, Lateral

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$(M_{11})_{wing} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{wing.lat} - \frac{(A_s)_{wing.vert} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.wing} := \text{find}\left(\mathbf{As_{req}}\right) = 0.702\ in^2$

$(A_{s.min})_{wing.ten} := As_{req.wing} = 0.702\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{wing.lat} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{wing.lat} = 0.54\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot \left(Wing_{thick} \cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (A_{s.min})_{wing.ten} \cdot \frac{4}{3} = 0.936\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{wing.req} := \text{if } A_{s\_1.3} < (A_s)_{wing.lat} \; as \leftarrow (A_{s.min})_{wing.ten} \text{ else } as \leftarrow A_{s.min} = 0.543\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{wing.lat} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the wing wall)*

$(Steel\_Check_{lat})_{Wing} := Check\left((A_s)_{wing.req}, (A_s)_{wing.lat}\right) = [\text{"OK"} \quad 0.7]$ *(check wing wall reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{wing.lat}}{\left(b \cdot d_{wing.lat}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{Wing} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Shear Check

$V_u := (V_{12})_{wing} = 28.9\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{wing.vert} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{wing.vert} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{wing.vert} + (A_s')_{wing.vert} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot Wall_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{wing.vert}} = 0.009$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{wing.vert}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{wing.vert} \right.$
$\text{else} \left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{wing.vert} \right. = 23.3\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_c := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{wing.vert}\right) = 23.3\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs} := \phi_V \cdot V_c = 17.5\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_u > \phi V_{cs} \left\| \text{"Shear strength from steel required"} \right.$
$\text{else} \left\| \text{"Shear strength from steel not required"} \right. = \text{"Shear strength from steel required"}$

$V_s := \frac{A_V \cdot (F_y)_{60} \cdot d_{wing.vert}}{s_{wing.vert}} = 113.9\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu := \phi_V \cdot \left(V_c + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{wing.vert}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_u, Vu\right) = [\text{"OK"} \quad 0.3]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn := \phi_V \cdot \left(V_c + V_s\right) = 102.9\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_u = 28.9\ kip$

$(Check_v)_{Wing} := Check\left(V_u, \phi_V Vn\right) = [\text{"OK"} \quad 0.3]$

Lateral Shear Check

$V_{u.lat} := (V_{23})_{wing} = 28.2\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{wing.lat} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{wing.lat} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{wing.lat} + (A_s')_{wing.lat} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot Wall_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot Wall_{thick}} = 0.0073$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{wing.lat}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{wing.lat} \right.$
$\text{else} \left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{wing.lat} \right. = 21.7\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_{c.lat} := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{wing.lat}\right) = 21.7\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs.lat} := \phi_V \cdot V_{c.lat} = 16.3\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_{u.lat} > \phi V_{cs.lat} \left\| \text{"Shear strength from steel required"} \right.$
$\text{else} \left\| \text{"Shear strength from steel not required"} \right. = \text{"Shear strength from steel required"}$

$V_{s.lat} := \frac{A_V \cdot (F_y)_{60} \cdot d_{wing.lat}}{s_{wing.lat}} = 106\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu_{lat} := \phi_V \cdot \left(V_{c.lat} + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{wing.lat}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_{u.lat}, Vu_{lat}\right) = [\text{"OK"} \quad 0.3]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn_{lat} := \phi_V \cdot \left(V_{c.lat} + V_{s.lat}\right) = 95.8\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_{u.lat} = 28.2\ kip$

$(Check_v)_{Wing.lat} := Check\left(V_{u.lat}, \phi_V Vn_{lat}\right) = [\text{"OK"} \quad 0.3]$

WING WALL	Bar Size	Spacing (in)
Vertical Bars		
Front Face	#8	12
Back Face	#8	12
Lateral Bars		
Front Face	#8	12
Back Face	#8	12

	Ma	Mn	Check	IC
Vertical Flexural Check	38.7	49.43	OK	0.78
Lateral Flexural Check	41.02	45.90	OK	0.89

	Ma	Mn	Check	IC
Vertical Shear Check	28.9	102.92	OK	0.28
Lateral Shear Check	28.16	95.82	OK	0.29

BOX WALL DESIGN

Box Wall Reinforcement

Note: design for the walls of the new box are designed for the loading conditions to the exterior walls as they experience higher loads. Interior box walls are loaded with hydrostatic pressure from the flow inside of the structure, exterior walls are loaded from hydrostatic and lateral earth pressures

Vertical Tension Bars Front Face

$(d_b)_{box.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in box)*

$s_{box.vert} := 12\ in$ *(vertical bar spacing in box)*

$c_{box.vert} := 4''\ \text{cover}$ *(cover for vertical reinforcement in box)*

$$(A_s)_{box.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{box.vert}^{\ 2} \cdot b}{s_{box.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in box)*

$$d_{box.vert} := CWall_{thick} - c_{box.vert} - \left(0.5 \cdot (d_b)_{box.vert}\right) = 13.5\ in$$ *(depth to reinforcement vertical reinforcement in box)*

Vertical Tension Bars Back Face

$(d_b)'_{box.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in box)*

$s'_{box.vert} := 12\ in$ *(vertical bar spacing in box)*

$c'_{box.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in box)*

$$(A_s')_{box.vert} := \frac{0.25 \cdot \pi \cdot (d_b)'_{box.vert}^{\ 2} \cdot b}{s'_{box.vert}} = 0.785\ in^2$$ *(area of steel vertical reinforcement in box)*

$$d'_{box.vert} := CWall_{thick} - c'_{box.vert} - \left(0.5 \cdot (d_b)'_{box.vert}\right) = 14.5\ in$$ *(depth to reinforcement vertical reinforcement in box)*

Lateral Tension Bars Front Face

$(d_b)_{box.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in box)*

$s_{box.lat} := 12\ in$ *(lateral bar spacing in box)*

$c_{box.lat} := 4''\ \text{cover}$ *(cover for lateral reinforcement in box)*

$$(A_s)_{box.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{box.lat}^{\ 2} \cdot b}{s_{box.lat}} = 0.785\ in^2$$ *(area of steel lateral reinforcement in box)*

$$d_{box.lat} := CWall_{thick} - c_{box.vert} - \left((d_b)_{box.vert}\right) - \left(0.5 \cdot (d_b)_{box.lat}\right) = 12.5\ in$$ *(depth to reinforcement lateral reinforcement in box)*

Lateral Tension Bars Back Face

$(d_b)'_{box.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in box)*

$s'_{box.lat} := 12\ in$ *(lateral bar spacing in box)*

$c'_{box.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in box)*

$$(A_s')_{box.lat} := \frac{0.25 \cdot \pi \cdot (d_b)'_{box.lat}^{\ 2} \cdot b}{s'_{box.lat}} = 0.785\ in^2$$ *(area of steel lateral reinforcement in box)*

$$d'_{box.lat} := CWall_{thick} - c'_{box.lat} - (d_b)'_{box.vert} - \left(0.5 \cdot (d_b)'_{box.lat}\right) = 13.5\ in$$ *(depth to reinforcement lateral reinforcement in box)*

Nominal Flexural Strength

$$a_{box.vert} := \frac{(A_s)_{box.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$$a_{box.lat} := \frac{(A_s)_{box.lat} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$\varepsilon_{cu} := 0.003$ *(ultimate strain in extreme compression fiber; ACI 318-14 R21.2.2)*

$$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.002$$ *(net tensile strain limit; ACI 318-14 R21.2.2)*

$$c_{box} := \frac{a_{box.vert}}{0.85} = 1.2\ in$$ *(depth to neutral axis)*

Check Flexural Strength

Vertical Flexural Front Face

$(M_{33})_{wall} = 37.4\ kip \cdot ft$ *(max vertical moment in box)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{box}}\right) \cdot \left(d_{box.vert} - c_{box}\right) = 0.031$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{box.vert} := \phi_f \cdot (A_s)_{box.vert} \cdot (F_y)_{60} \cdot \left(d_{box.vert} - \frac{a_{box.vert}}{2}\right) = 45.9\ kip \cdot ft$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$(Flex_{nom})_{Box} := Check\left((M_{33})_{wall}, (\phi M_n)_{box.vert}\right) = [\text{"OK"} \quad 0.82]$ *(nominal flexural check)*

Lateral Flexural Front Face

$(M_{11})_{wall} = 23.1\ kip \cdot ft$ *(max lateral moment in box)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{box}}\right) \cdot \left(d_{box.lat} - c_{box}\right) = 0.028$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{box.lat} := \phi_f \cdot (A_s)_{box.lat} \cdot (F_y)_{60} \cdot \left(d_{box.lat} - \frac{a_{box.lat}}{2}\right) = 42.4\ kip \cdot ft$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$(Flex_{nom.lat})_{Box} := Check\left((M_{11})_{wall}, (\phi M_n)_{box.lat}\right) = [\text{"OK"} \quad 0.55]$ *(nominal flexural check)*

Box Culvert Wall Deflection Check

$c_{cracked} := \frac{-n \cdot (A_s)_{box.vert} + \sqrt{\left(n \cdot (A_s)_{box.vert}\right)^2 + \left(2 \cdot n \cdot (A_s)_{box.vert} \cdot b \cdot d_{box.vert}\right)}}{b} = 3.2\ in$ *(location from the neutral axis to the extreme compression fiber)*

$I_{cr} := \frac{1}{3} \cdot b \cdot {c_{cracked}}^3 + n \cdot (A_s)_{box.vert} \cdot \left(d_{box.vert} - c_{cracked}\right)^2 = 763\ in^4$ *(cracked moment of inertia)*

$M_a := (M_{11})_{wall} = 23.1\ kip \cdot ft$ *(moment applied to cracked section)*

$I_g := \frac{b \cdot {CWall_{thick}}^3}{12} = 5832\ in^4$ *(gross moment of inertia)*

$M_{cr} := \frac{\left(7.5 \cdot \sqrt{f'c} \cdot psi\right) \cdot I_g}{0.5 \cdot CWall_{thick}} = 27.2\ kip \cdot ft$ *(cracked moment of inertia, ACI 318-14, Eq 24.2.3.5)*

$$I_e := \text{if } M_a \le \left(\frac{2}{3}\cdot M_{cr}\right) \;\|\; I_g \;\text{else}\; \left\| \frac{I_{cr}}{1-\left(\frac{\frac{2}{3}\cdot M_{cr}}{M_a}\right)^2 \cdot \left(1-\frac{I_{cr}}{I_g}\right)} \right. = 1632.1\ in^4$$ *(effective moment of inertia, ACI 318-14, Table 24.2.3.5)*

Minimum Required Steel for Flexure, Vertical

Solve Constraints Given Values

$As_{req} := 1.5\ in^2$

$(M_{33})_{wall} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{box.vert} - \frac{(A_s)_{box.vert}\cdot (F_y)_{60}}{2\cdot 0.85\cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.box} := \text{find}\left(As_{req}\right) = 0.64\ in^2$

$(A_{s.min})_{box.ten} := As_{req.box} = 0.64\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3\cdot\sqrt{f'_c\cdot psi}}{(F_y)_{60}}\cdot b\cdot d_{box.vert} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}}\cdot b\cdot d_{box.vert} = 0.54\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5\cdot 0.003\cdot\left(CWall_{thick}\cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (A_{s.min})_{box.ten}\cdot\frac{4}{3} = 0.854\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{box.req} := \text{if } A_{s\_1.3} < (A_s)_{box.vert} \;\|\; as \leftarrow (A_{s.min})_{box.ten} \;\text{else}\; \| as \leftarrow A_{s.min} = 0.543\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{box.vert} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the box wall)*

$(Steel\_Check_{Vert})_{Box} := Check\left((A_s)_{box.req}, (A_s)_{box.vert}\right) = [\text{“OK”}\quad 0.7]$ *(check box wall reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25\cdot\left(\frac{0.85\cdot\beta_1\cdot f'_c}{(F_y)_{60}}\right)\cdot\left(\frac{87000}{87000+\left((F_y)_{60}\cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{box.vert}}{\left(b\cdot d_{box.vert}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{Box} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \;\; 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Minimum Required Steel for Flexure, Lateral

Solver Constraints Guess Values

$As_{req} := 1.5\ in^2$

$(M_{11})_{wall} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{box.lat} - \dfrac{(A_s)_{box.vert} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.box} := \mathbf{find}\left(As_{req}\right) = 0.429\ in^2$

$(A_{s.min})_{box.ten} := As_{req.box} = 0.429\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{box.lat} = 0.503\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \dfrac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{box.lat} = 0.5\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot \left(CWall_{thick} \cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.503\ in^2$

$A_{s\_1.3} := (A_{s.min})_{box.ten} \cdot \dfrac{4}{3} = 0.572\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{box.req} := \text{if } A_{s\_1.3} < (A_s)_{box.lat}$ *(required steel to meet steel requirements)*

$\quad as \leftarrow (A_{s.min})_{box.ten}$ *(includes ACI 318-19 9.6.1.2(a) and (b))*

else

$\quad as \leftarrow A_{s.min}$ *(excludes ACI 318-19 9.6.1.2(a) and (b))*

$(A_s)_{box.lat} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the box wall)*

$(Steel\_Check_{lat})_{Box} := Check\left((A_s)_{box.req}, (A_s)_{box.lat}\right) = [\text{“OK”} \;\; 0.5]$

(check stem reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\dfrac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\dfrac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \dfrac{(A_s)_{box.lat}}{\left(b \cdot d_{box.lat}\right)} = 0.0052$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{Box} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{“OK”} \;\; 0.7]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

<u>Box Wall Compression Check</u>

$(A_g)_{comp.box} := b \cdot CWall_{thick} = 216\ in^2$ *(gross area for compression analysis, ACI 318-19 Eq 22.4.2.2)*

$(A_{st})_{comp.box} := (A_s)_{box.vert} + (A_s')_{box.vert} = 1.6\ in^2$ *(total area of longitudinal reinforcement, ACI 318-19 Eq 22.4.2.2)*

$P_0 := 0.85 \cdot f'_c \cdot \left((A_g)_{comp.box} - (A_{st})_{comp.box}\right) + (F_y)_{60} \cdot (A_{st})_{comp.box} = 914.4\ kip$ *(nominal axial strength in concrete section)*

$P_{n.max} := 0.8 \cdot P_0 = 731.6\ kip$ *(maximum axial strength, ACI 318-19 Table 22.4.2.1)*

$\phi_C P_{n.max} := \phi_C \cdot P_{n.max} = 475.5\ kip$ *(factored axial strength)*

$(Comp_{nom})_{Box} := Check\left((V_{12})_{wall}, \phi_C P_{n.max}\right) = [\text{"OK"} \quad 0]$ *(axial strength check for member)*

Lateral Shear Check

$V_{u.lat} := (V_{23})_{wall} = 21.2\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{box.lat} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{box.lat} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{box.lat} + (A_s')_{box.lat} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot CWall_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{box.lat}} = 0.0105$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{box.lat}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \parallel \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{box.lat}$ else $\left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{box.lat} = 20.1\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_{c.lat} := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{box.lat}\right) = 20.1\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs.lat} := \phi_V \cdot V_{c.lat} = 15.1\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_{u.lat} > \phi V_{cs.lat} \parallel \text{"Shear strength from steel required"}$ else $\parallel \text{"Shear strength from steel not required"} = \text{"Shear strength from steel required"}$

$V_{s.lat} := \frac{A_V \cdot (F_y)_{60} \cdot d_{box.lat}}{s_{box.lat}} = 98.2\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu_{lat} := \phi_V \cdot \left(V_{c.lat} + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{box.lat}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_{u.lat}, Vu_{lat}\right) = [\text{"OK"} \quad 0.3]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn_{lat} := \phi_V \cdot \left(V_{c.lat} + V_{s.lat}\right) = 88.7\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_{u.lat} = 21.2\ kip$

$(Check_v)_{Box.lat} := Check\left(V_{u.lat}, \phi_V Vn_{lat}\right) = [\text{"OK"} \quad 0.2]$

BOX CULVERTWALL	Bar Size	Spacing (in)
Vertical Bars		
Front Face	#8	12
Back Face	#8	12
Lateral Bars		
Front Face	#8	12
Back Face	#8	12

	Ma	Mn	Check	IC
Vertical Flexural Check	23.14	45.90	OK	0.82
Lateral Flexural Check	37.41	42.36	OK	0.55

	Pa	Pn	Check	IC
Compression Check	22.45	475.51	OK	0.05
	Va	Vn	Check	IC
Lateral Shear Check	21.23	88.72	OK	0.24

BUTTRESS WALL DESIGN

Buttress Wall Reinforcement

Vertical Tension Bars Front Face

$(d_b)_{but.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in buttress)*

$s_{but.vert} := 12\ in$ *(vertical bar spacing in buttress)*

$c_{but.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in buttress)*

$(A_s)_{but.vert} := \frac{0.25 \cdot \pi \cdot (d_b)_{but.vert}^{\ 2} \cdot b}{s_{but.vert}} = 0.785\ in^2$ *(area of steel vertical reinforcement in buttress)*

$d_{but.vert} := Buttress_{thick} - c_{but.vert} - \left(0.5 \cdot (d_b)_{but.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in buttress)*

Vertical Tension Bars Back Face

$(d_b)'_{but.vert} := \text{Bar Size: \#8} = 1\ in$ *(vertical bar size in buttress)*

$s'_{but.vert} := 12\ in$ *(vertical bar spacing in buttress)*

$c'_{but.vert} := 3''\ \text{cover}$ *(cover for vertical reinforcement in buttress)*

$(A_s')_{but.vert} := \frac{0.25 \cdot \pi \cdot (d_b)'_{but.vert}^{\ 2} \cdot b}{s'_{but.vert}} = 0.785\ in^2$ *(area of steel vertical reinforcement in buttress)*

$d'_{but.vert} := Buttress_{thick} - c'_{but.vert} - \left(0.5 \cdot (d_b)'_{but.vert}\right) = 14.5\ in$ *(depth to reinforcement vertical reinforcement in buttress)*

Lateral Tension Bars Front Face

$(d_b)_{but.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in buttress)*

$s_{but.lat} := 12\ in$ *(lateral bar spacing in buttress)*

$c_{but.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in buttress)*

$(A_s)_{but.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{but.lat}^{\ 2} \cdot b}{s_{but.lat}} = 0.785\ in^2$ *(area of steel lateral reinforcement in buttress)*

$d_{but.lat} := Buttress_{thick} - c_{but.vert} - \left((d_b)_{but.vert}\right) - \left(0.5 \cdot (d_b)_{but.lat}\right) = 13.5\ in$ *(depth to reinforcement lateral reinforcement in buttress)*

Lateral Tension Bars Back Face

$(d_b)'_{but.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in buttress)*

$s'_{but.lat} := 12\ in$ *(lateral bar spacing in buttress)*

$c'_{but.lat} := 3''\ \text{cover}$ *(cover for lateral reinforcement in buttress)*

$(A_s')_{but.lat} := \frac{0.25 \cdot \pi \cdot (d_b)'_{but.lat}^{\ 2} \cdot b}{s'_{but.lat}} = 0.785\ in^2$ *(area of steel lateral reinforcement in buttress)*

$d'_{but.lat} := Buttress_{thick} - c'_{but.lat} - (d_b)'_{but.vert} - \left(0.5 \cdot (d_b)'_{but.lat}\right) = 13.5\ in$ *(depth to reinforcement lateral reinforcement in buttress)*

Nominal Flexural Strength

$a_{but.vert} := \frac{(A_s)_{but.vert} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$a_{but.lat} := \frac{(A_s)_{but.lat} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$\varepsilon_{cu} := 0.003$ *(ultimate strain in extreme compression fiber; ACI 318-14 R21.2.2)*

$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.002$ *(net tensile strain limit; ACI 318-14 R21.2.2)*

$c_{butt} := \frac{a_{but.vert}}{0.85} = 1.2\ in$ *(neutral axis depth for buttress walls)*

Check Flexural Strength

Vertical Flexural Front Face

$(M_{33})_{but} = 12.6\ kip \cdot ft$ *(max vertical moment in buttress wall)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{butt}}\right) \cdot \left(d_{but.vert} - c_{butt}\right) = 0.033$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{but.vert} := \phi_f \cdot (A_s)_{but.vert} \cdot (F_y)_{60} \cdot \left(d_{but.vert} - \frac{a_{but.vert}}{2}\right) = 49.4\ kip \cdot ft$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$(Flex_{nom})_{But} := Check\left((M_{33})_{but}, (\phi M_n)_{but.vert}\right) = [\text{"OK"} \quad 0.25]$ *(nominal flexural check)*

Lateral Flexural Front Face

$(M_{11})_{but} = 19.2\ kip \cdot ft$ *(max lateral moment in buttress wall)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{butt}}\right) \cdot \left(d_{but.lat} - c_{butt}\right) = 0.031$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{but.lat} := \phi_f \cdot (A_s)_{but.lat} \cdot (F_y)_{60} \cdot \left(d_{but.lat} - \frac{a_{but.lat}}{2}\right) = 45.9\ kip \cdot ft$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$(Flex_{nom.lat})_{But} := Check\left((M_{11})_{but}, (\phi M_n)_{but.lat}\right) = [\text{"OK"} \quad 0.42]$ *(nominal flexural check)*

Minimum Required Steel for Flexure, Vertical

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$(M_{33})_{but} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{but.vert} - \frac{(A_s)_{but.vert} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.but} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.199\ in^2$

$(A_{s.min})_{but.ten} := As_{req.but} = 0.199\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{but.vert} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{but.vert} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (Buttress_{thick} \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{but.ten} \cdot \frac{4}{3} = 0.266\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{but.req} := \text{if } A_{s\_1.3} < (A_s)_{but.vert} \parallel as \leftarrow (A_{s.min})_{but.ten}$ else $\parallel as \leftarrow A_{s.min} = 0.199\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{but.vert} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the buttress wall)*

$(Steel\_Check_{Vert})_{But} := Check\left((A_s)_{but.req}, (A_s)_{but.vert}\right) = [\text{"OK"} \quad 0.3]$

(check box wall reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{but.vert}}{\left(b \cdot d_{but.vert}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Vert})_{But} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Minimum Required Steel for Flexure, Lateral

Solve Constraints Values

$As_{req} := 1.5\ in^2$

$(M_{11})_{but} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{but.lat} - \frac{(A_s)_{but.lat} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.but} := \text{find}\left(\mathbf{As_{req}}\right) = 0.328\ in^2$

$(A_{s.min})_{but.ten} := As_{req.but} = 0.328\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{but.lat} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{but.lat} = 0.54\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (Buttress_{thick} \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (A_{s.min})_{but.ten} \cdot \frac{4}{3} = 0.438\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{box.req} := \text{if } A_{s\_1.3} < (A_s)_{but.lat} \parallel as \leftarrow (A_{s.min})_{but.ten} \text{ else } \parallel as \leftarrow A_{s.min} = 0.328\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{but.lat} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the box wall)*

$(Steel\_Check_{lat})_{But} := Check\left((A_s)_{but.req}, (A_s)_{but.lat}\right) = [\text{"OK"} \quad 0.3]$ *(check buttress reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{but.lat}}{(b \cdot d_{but.lat})} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{But} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Tensile Loading

$(A_{st})_{ten.but} := (A_s)_{but.vert} + (A_s')_{but.vert} = 1.6\ in^2$ *(total area of longitudinal reinforcement, ACI 318-19 Eq 22.4.2.2)*

$f_{py} := 0\ ksi$ *(specified yield strength of prestressing reinforcement, ACI 318-19)*

$f_{se} := 0\ ksi$ *(effective stress in prestressed reinforcement, ACI 318-19)*

$\Delta f_p := 0\ ksi$ *(increase in stress in prestressed reinforcement due to factored loads, ACI 318-19)*

$A_{pt} := 0\ in^2$ *(area of pretension reinforcement, ACI 318-19)*

$P_{nt.max} := (F_y)_{60} \cdot (A_{st})_{ten.but} + \left(\left(f_{se} + \Delta f_p\right) \cdot A_{pt}\right) = 94.2\ kip$ $(F_1$

$\phi_T P_{n.max} := \phi_T \cdot P_{nt.max} = 84.8\ kip$ *(maximum axial strength, ACI 318-19 Table 22.4.2.1)*

$(Tens_{nom})_{But} := Check\left((F_{11})_{but}, \phi_T P_{n.max}\right) = [\text{"OK"} \quad 0.6]$ *(axial strength check for member)*

Lateral Shear Check

$V_{u.lat} := (V_{23})_{but} = 9\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$$Av,min := \max\left(s_{but.lat} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{but.lat} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{but.lat} + (A_s')_{but.lat} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot Buttress_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{but.vert}} = 0.009$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{but.lat}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c :=$ if $A_V \geq Av,min$ $\left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{but.lat}\right.$ else $\left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{but.lat}\right.$ $= 21.7\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_{c.lat} := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{but.lat}\right) = 21.7\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs.lat} := \phi_V \cdot V_{c.lat} = 16.3\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} :=$ if $V_{u.lat} > \phi V_{cs.lat}$ $\left\|\right.$ "Shear strength from steel required" else $\left\|\right.$ "Shear strength from steel not required" $=$ "Shear strength from steel not required"

$V_{s.lat} := \frac{A_V \cdot (F_y)_{60} \cdot d_{but.lat}}{s_{but.lat}} = 106\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu_{lat} := \phi_V \cdot \left(V_{c.lat} + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{but.lat}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_{u.lat}, Vu_{lat}\right) = [\text{"OK"} \quad 0.1]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn_{lat} := \phi_V \cdot \left(V_{c.lat} + V_{s.lat}\right) = 95.8\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_{u.lat} = 9\ kip$

$(Check_v)_{But.lat} := Check\left(V_{u.lat}, \phi_V Vn_{lat}\right) = [\text{"OK"} \quad 0.1]$

BUTTRESS WALL	Bar Size	Spacing (in)
Vertical Bars		
Front Face	#8	12
Back Face	#8	12
Lateral Bars		
Front Face	#8	12
Back Face	#8	12

	Ma	Mn	Check	IC
Vertical Flexural Check	12.55	49.43	OK	0.25
Lateral Flexural Check	19.18	45.90	OK	0.42

	Ta	Tn	Check	IC
Axial Tensile Check	48.4	84.82	OK	0.57
	Va	Vn	Check	IC
Lateral Shear Check	9.04	95.82	OK	0.09

TOP SLAB DESIGN

Top Slab Reinforcement

Longitudinal Tension Bars, Top Face

$(d_b)_{tslab.long} := \text{Bar Size: \#8} = 1\ in$ *(longitudinal top bar size in slab)*

$s_{tslab.long} := 12\ in$ *(longitudinal top bar spacing in slab)*

$c_{tslab.long} := \text{3″ cover}$ *(cover for longitudinal top reinforcement)*

$$(A_s)_{tslab.long} := \frac{0.25 \cdot \pi \cdot (d_b)_{tslab.long}^{\ 2} \cdot b}{s_{tslab.long}} = 0.785\ in^2$$ *(area of steel longitudinal top reinforcement)*

$$d_{tslab.long} := TSlab_{thick} - c_{tslab.long} - \left(0.5 \cdot (d_b)_{tslab.long}\right) = 14.5\ in$$ *(depth to reinforcement longitudinal reinforcement)*

Longitudinal Tension Bars, Bottom Face

$(d_b)'_{tslab.long} := \text{Bar Size: \#8} = 1\ in$ *(longitudinal bottom bar size)*

$s'_{tslab.long} := 12\ in$ *(longitudinal bottom bar spacing)*

$c'_{tslab.long} := \text{3″ cover}$ *(cover for longitudinal bottom reinforcement)*

$$(A_s')_{tslab.long} := \frac{0.25 \cdot \pi \cdot (d_b)'_{tslab.long}^{\ 2} \cdot b}{s'_{tslab.long}} = 0.785\ in^2$$ *(area of steel longitudinal bottom reinforcement)*

$$d'_{tslab.long} := TSlab_{thick} - c'_{tslab.long} - \left(0.5 \cdot (d_b)'_{tslab.long}\right) = 14.5\ in$$ *(depth to longitudinal reinforcement)*

Lateral Tension Bars Top Face

$(d_b)_{tslab.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in top slab)*

$s_{tslab.lat} := 12\ in$ *(lateral bar spacing in top slab)*

$c_{tslab.lat} := \text{3″ cover}$ *(cover for lateral reinforcement in top slab)*

$$(A_s)_{tslab.lat} := \frac{0.25 \cdot \pi \cdot (d_b)_{tslab.lat}^{\ 2} \cdot b}{s_{tslab.lat}} = 0.785\ in^2$$ *(area of steel lateral reinforcement in top slab)*

$$d_{tslab.lat} := TSlab_{thick} - c_{tslab.long} - \left((d_b)_{tslab.long}\right) - \left(0.5 \cdot (d_b)_{tslab.lat}\right) = 13.5\ in$$ *(depth to reinforcement lateral reinforcement in top slab)*

Lateral Tension Bars Bottom Face

$(d_b)'_{tslab.lat} := \text{Bar Size: \#8} = 1\ in$ *(lateral bar size in top slab)*

$s'_{tslab.lat} := 12\ in$ *(lateral bar spacing in top slab)*

$c'_{tslab.lat} := \text{3″ cover}$ *(cover for lateral reinforcement in top slab)*

$$(A_s')_{tslab.lat} := \frac{0.25 \cdot \pi \cdot (d_b)'_{tslab.lat}^{\ 2} \cdot b}{s'_{tslab.lat}} = 0.785\ in^2$$ *(area of steel lateral reinforcement in top slab)*

$$d'_{tslab.lat} := TSlab_{thick} - c'_{tslab.lat} - (d_b)'_{tslab.long} - \left(0.5 \cdot (d_b)'_{tslab.lat}\right) = 13.5\ in$$ *(depth to reinforcement lateral reinforcement in top slab)*

Nominal Flexural Strength

$$a_{tslab.long} := \frac{(A_s)_{tslab.long} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$$a_{tslab.lat} := \frac{(A_s)_{tslab.lat} \cdot (F_y)_{60}}{0.85 \cdot f'_c \cdot b} = 1.03\ in$$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$\varepsilon_{cu} := 0.003$ *(ultimate strain in extreme compression fiber; ACI 318-14 R21.2.2)*

$$\varepsilon_{ty} := \frac{(F_y)_{60}}{E_{steel}} = 0.002$$ *(net tensile strain limit; ACI 318-14 R21.2.2)*

$$c_{tslab} := \frac{a_{tslab.long}}{0.85} = 1.2\ in$$ *(depth to neutral axis for top slab)*

Check Flexural Strength

Longitudinal Flexural Top Face

$(M_{11})_{topslab} = 23.9\ kip \cdot ft$ *(max longitudinal moment in top slab)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{tslab}}\right) \cdot \left(d_{tslab.long} - c_{tslab}\right) = 0.033$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{tslab.long} := \phi_f \cdot (A_s)_{tslab.long} \cdot (F_y)_{60} \cdot \left(d_{tslab.long} - \frac{a_{tslab.long}}{2}\right) = 49.4\ kip \cdot ft$
(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom})_{TSlab} := Check\left((M_{11})_{topslab}, (\phi M_n)_{tslab.long}\right) = [\text{“OK”} \quad 0.48]$
(nominal flexural check)

Lateral Flexural Front Face

$(M_{33})_{topslab} = 22.9\ kip \cdot ft$ *(max lateral moment in top slab)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{tslab}}\right) \cdot \left(d_{tslab.lat} - c_{tslab}\right) = 0.031$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{tslab.lat} := \phi_f \cdot (A_s)_{tslab.lat} \cdot (F_y)_{60} \cdot \left(d_{tslab.lat} - \frac{a_{tslab.lat}}{2}\right) = 45.9\ kip \cdot ft$
(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom.lat})_{TSlab} := Check\left((M_{33})_{topslab}, (\phi M_n)_{tslab.lat}\right) = [\text{“OK”} \quad 0.5]$

(nominal flexural check)

Minimum Required Steel for Flexure, Longitudinal

Solve Constraints Guess Values

$As_{req} := 1.5\ in^2$

$(M_{11})_{topslab} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{tslab.long} - \frac{(A_s)_{tslab.long} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.tslab} := \mathbf{find}\left(\mathbf{As_{req}}\right) = 0.379\ in^2$

$(A_{s.min})_{tslab.ten} := As_{req.tslab} = 0.379\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{tslab.long} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{tslab.long} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot \left(TSlab_{thick} \cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{tslab.ten} \cdot \frac{4}{3} = 0.505\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{tslab.req} := \text{if } A_{s\_1.3} < (A_s)_{tslab.long} \parallel as \leftarrow (A_{s.min})_{tslab.ten}$ $\text{else} \parallel as \leftarrow A_{s.min} = 0.379\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{tslab.long} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{Long})_{TSlab} := Check\left((A_s)_{tslab.req}, (A_s)_{tslab.long}\right) = [\text{"OK"} \quad 0.5]$

(check stem reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{tslab.long}}{\left(b \cdot d_{tslab.long}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Long})_{TSlab} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Minimum Required Steel for Flexure, Lateral

Given

$As_{req} := 1.5\ in^2$

$(M_{33})_{topslab} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{tslab.lat} - \frac{(A_s)_{tslab.lat} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.tslab} := \text{find}\left(As_{req}\right) = 0.391\ in^2$

SolveConstraints Guess Values

$(A_{s.min})_{tslab.ten} := As_{req.tslab} = 0.391\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{tslab.lat} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{tslab.lat} = 0.54\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot \left(TSlab_{thick} \cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (A_{s.min})_{tslab.ten} \cdot \frac{4}{3} = 0.522\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{tslab.req} := \text{if } A_{s\_1.3} < (A_s)_{tslab.lat} \parallel as \leftarrow (A_{s.min})_{tslab.ten}$ $\text{else} \parallel as \leftarrow A_{s.min} = 0.391\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{tslab.lat} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{lat})_{TSlab} := Check\left((A_s)_{tslab.req}, (A_s)_{tslab.lat}\right) = [\text{"OK"} \quad 0.5]$ *(check stem reinforcement to required reinforcement)*

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{tslab.lat}}{\left(b \cdot d_{tslab.lat}\right)} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{TSlab} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Shear Check

$V_u := (V_{23})_{topslab} = 24\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{tslab.long} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{tslab.long} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{tslab.long} + (A_s')_{tslab.long} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot TSlab_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{tslab.long}} = 0.009$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{tslab.long}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{tslab.long} \right. \text{else} \left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{tslab.long} \right. = 23.3\ kip$ *(concrete shear strength per ACI 318-19 Table 22.5.5.1)*

$V_c := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{tslab.long}\right) = 23.3\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs} := \phi_V \cdot V_c = 17.5\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_u > \phi V_{cs} \left\| \text{“Shear strength from steel required”} \right. \text{else} \left\| \text{“Shear strength from steel not required”} \right. = \text{“Shear strength from steel required”}$

$V_s := \frac{A_V \cdot (F_y)_{60} \cdot d_{tslab.long}}{s_{tslab.long}} = 113.9\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu := \phi_V \cdot \left(V_c + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{tslab.long}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_u, Vu\right) = [\text{“OK”} \quad 0.3]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn := \phi_V \cdot \left(V_c + V_s\right) = 102.9\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_u = 24\ kip$

$(Check_v)_{TSlab} := Check\left(V_u, \phi_V Vn\right) = [\text{“OK”} \quad 0.2]$

Lateral Shear Check

$V_{u.lat} := (V_{12})_{topslab} = 20\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{tslab.lat} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{tslab.lat} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{tslab.lat} + (A_s')_{tslab.lat} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot TSlab_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{tslab.lat}} = 0.0097$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{tslab.lat}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{tslab.lat} \right.$ else $\left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{tslab.lat} \right. = 21.7\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_{c.lat} := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{tslab.lat}\right) = 21.7\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs.lat} := \phi_V \cdot V_{c.lat} = 16.3\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_{u.lat} > \phi V_{cs.lat} \left\| \text{“Shear strength from steel required”} \right.$ else $\left\| \text{“Shear strength from steel not required”} \right. = \text{“Shear strength from steel required”}$

$V_{s.lat} := \dfrac{A_V \cdot (F_y)_{60} \cdot d_{tslab.lat}}{s_{tslab.lat}} = 106\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu_{lat} := \phi_V \cdot \left(V_{c.lat} + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{tslab.lat}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_{u.lat}, Vu_{lat}\right) = [\text{“OK”} \quad 0.2]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn_{lat} := \phi_V \cdot \left(V_{c.lat} + V_{s.lat}\right) = 95.8\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_{u.lat} = 20\ kip$

$(Check_v)_{TSlab.lat} := Check\left(V_{u.lat}, \phi_V Vn_{lat}\right) = [\text{“OK”} \quad 0.2]$

TOP SLAB	Bar Size	Spacing (in)
Vertical Bars		
Front Face	#8	12
Back Face	#8	12
Lateral Bars		
Front Face	#8	12
Back Face	#8	12

	Ma	Mn	Check	IC
Vertical Flexural Check	23.86	49.43	OK	0.48
Lateral Flexural Check	22.86	45.90	OK	0.50

	Va	Vn	Check	IC
Vertical Shear Check	24.03	102.92	OK	0.23
Lateral Shear Check	19.96	95.82	OK	0.21

BOTTOM SLAB DESIGN

Bottom Slab Reinforcement

Longitudinal Tension Bars, Top Face

$(d_b)_{bslab.long} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(longitudinal top bar size in slab)*

$s_{bslab.long} := 12\ in$ *(longitudinal top bar spacing in slab)*

$c_{bslab.long} := \boxed{3''\ \text{cover}}$ *(cover for longitudinal top reinforcement)*

$(A_s)_{bslab.long} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{bslab.long}^{\ 2} \cdot b}{s_{bslab.long}} = 0.785\ in^2$ *(area of steel longitudinal top reinforcement)*

$d_{bslab.long} := BSlab_{thick} - c_{bslab.long} - \left(0.5 \cdot (d_b)_{bslab.long}\right) = 14.5\ in$ *(depth to reinforcement longitudinal reinforcement)*

Longitudinal Tension Bars, Bottom Face

$(d_b)'_{bslab.long} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(longitudinal bottom bar size)*

$s'_{bslab.long} := 12\ in$ *(longitudinal bottom bar spacing)*

$c'_{bslab.long} := \boxed{3''\ \text{cover}}$ *(cover for longitudinal bottom reinforcement)*

$(A_s')_{bslab.long} := \dfrac{0.25 \cdot \pi \cdot (d_b)'_{bslab.long}^{\ 2} \cdot b}{s'_{bslab.long}} = 0.785\ in^2$ *(area of steel longitudinal bottom reinforcement)*

$d'_{bslab.long} := BSlab_{thick} - c'_{bslab.long} - \left(0.5 \cdot (d_b)'_{bslab.long}\right) = 14.5\ in$ *(depth to longitudinal reinforcement)*

Lateral Tension Bars Top Face

$(d_b)_{bslab.lat} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(lateral bar size in top slab)*

$s_{bslab.lat} := 12\ in$ *(lateral bar spacing in top slab)*

$c_{bslab.lat} := \boxed{3''\ \text{cover}}$ *(cover for lateral reinforcement in top slab)*

$(A_s)_{bslab.lat} := \dfrac{0.25 \cdot \pi \cdot (d_b)_{bslab.lat}^{\ 2} \cdot b}{s_{bslab.lat}} = 0.785\ in^2$ *(area of steel lateral reinforcement in top slab)*

$d_{bslab.lat} := BSlab_{thick} - c_{bslab.long} - \left((d_b)_{bslab.long}\right) - \left(0.5 \cdot (d_b)_{bslab.lat}\right) = 13.5\ in$ *(depth to reinforcement lateral reinforcement in top slab)*

Lateral Tension Bars Bottom Face

$(d_b)'_{bslab.lat} := \boxed{\text{Bar Size: \#8}} = 1\ in$ *(lateral bar size in top slab)*

$s'_{bslab.lat} := 12\ in$ *(lateral bar spacing in top slab)*

$c'_{bslab.lat} := \boxed{3''\ \text{cover}}$ *(cover for lateral reinforcement in top slab)*

$(A_s')_{bslab.lat} := \dfrac{0.25 \cdot \pi \cdot (d_b)'_{bslab.lat}^{\ 2} \cdot b}{s'_{bslab.lat}} = 0.785\ in^2$ *(area of steel lateral reinforcement in top slab)*

$d'_{bslab.lat} := BSlab_{thick} - c'_{bslab.lat} - (d_b)'_{bslab.long} - \left(0.5 \cdot (d_b)'_{bslab.lat}\right) = 13.5\ in$ *(depth to reinforcement lateral reinforcement in top slab)*

Nominal Flexural Strength

$a_{bslab.long} := \dfrac{(A_s)_{bslab.long} \cdot (F_y)_{60}}{\beta_1 \cdot f'_c \cdot b} = 1.06\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$a_{bslab.lat} := \dfrac{(A_s)_{bslab.lat} \cdot (F_y)_{60}}{\beta_1 \cdot f'_c \cdot b} = 1.06\ in$ *(depth of equivalent stress block, ACI 318-19 Eq. 22.2.2.4.1, EM 1110-2-2104 Eq B-50)*

$\varepsilon_{cu} := 0.003$ *(ultimate strain in extreme compression fiber; ACI 318-14 R21.2.2)*

$\varepsilon_{ty} := \dfrac{(F_y)_{60}}{E_{steel}} = 0.002$ *(net tensile strain limit; ACI 318-14 R21.2.2)*

$c_{bslab} := a_{bslab.long} \div 0.85 = 1.2\ in$ *(depth to the neutral axis for bottom slab)*

Check Flexural Strength

Longitudinal Flexural Top Face

$(M_{11})_{botslab} = 41.3\ kip \cdot ft$ *(max longitudinal moment in top slab)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{bslab}}\right) \cdot \left(d_{bslab.long} - c_{bslab}\right) = 0.032$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{bslab.long} := \phi_f \cdot (A_s)_{bslab.long} \cdot (F_y)_{60} \cdot \left(d_{bslab.long} - \frac{a_{bslab.long}}{2}\right) = 49.4\ kip \cdot ft$
(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom})_{BSlab} := Check\left((M_{11})_{botslab}, (\phi M_n)_{bslab.long}\right) = [\text{“OK”} \quad 0.84]$
(nominal flexural check)

Lateral Flexural Front Face

$(M_{33})_{botslab} = 39.8\ kip \cdot ft$ *(max lateral moment in top slab)*

$\varepsilon_t := \left(\frac{\varepsilon_{cu}}{c_{bslab}}\right) \cdot \left(d_{bslab.lat} - c_{bslab}\right) = 0.03$ *(net tensile strain; ACI 318-14 R21.2.2)*

$\phi_f := \begin{cases} 0.65 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.65 + (0.9 - 0.65) \cdot \frac{\varepsilon_t - \varepsilon_{ty}}{0.005 - \varepsilon_{ty}} & \text{if } \varepsilon_{ty} < \varepsilon_t < 0.005 \\ 0.9 & \text{otherwise} \end{cases} = 0.9$ *(strength reduction factor for moment; ACI 318-14 Table 21.2.2)*

$(\phi M_n)_{bslab.lat} := \phi_f \cdot (A_s)_{bslab.lat} \cdot (F_y)_{60} \cdot \left(d_{bslab.lat} - \frac{a_{bslab.lat}}{2}\right) = 45.8\ kip \cdot ft$
(nominal flexural strength, EM1110-2-2104, B-49)

$(Flex_{nom.lat})_{BSlab} := Check\left((M_{33})_{botslab}, (\phi M_n)_{bslab.lat}\right) = [\text{“OK”} \quad 0.87]$
(nominal flexural check)

Minimum Required Steel for Flexure, Longitudinal

Solve Constraints Guess Values

$As_{req} := 1.5\ in^2$

$(M_{11})_{botslab} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{bslab.long} - \frac{(A_s)_{bslab.long} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.bslab} := \text{find}\left(As_{req}\right) = 0.656\ in^2$

$(A_{s.min})_{bslab.ten} := As_{req.bslab} = 0.656\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{bslab.long} = 0.584\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{bslab.long} = 0.58\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

temperature and shrinkage reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot \left(BSlab_{thick} \cdot b\right) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.584\ in^2$

$A_{s\_1.3} := (A_{s.min})_{bslab.ten} \cdot \frac{4}{3} = 0.874\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{bslab.req} := \text{if } A_{s\_1.3} < (A_s)_{bslab.long} \parallel as \leftarrow (A_{s.min})_{bslab.ten}$ $\text{else} \parallel as \leftarrow A_{s.min} = 0.584\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{bslab.long} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{Long})_{BSlab} := Check\left((A_s)_{bslab.req}, (A_s)_{bslab.long}\right) = [\text{"OK"} \quad 0.7]$

(check stem reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{bslab.long}}{\left(b \cdot d_{bslab.long}\right)} = 0.0045$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{Long})_{BSlab} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Minimum Required Steel for Flexure, Lateral

Solve Constraints Given Values

$As_{req} := -151.5\ in^2$

$(M_{33})_{botslab} = \phi_b \cdot As_{req} \cdot (F_y)_{60} \cdot \left(d_{bslab.lat} - \frac{(A_s)_{bslab.lat} \cdot (F_y)_{60}}{2 \cdot 0.85 \cdot f'_c \cdot b}\right)$ *(nominal flexural strength, EM1110-2-2104, B-49)*

$As_{req.bslab} := \text{find}\left(As_{req}\right) = 0.682\ in^2$

$(A_{s.min})_{bslab.ten} := As_{req.bslab} = 0.682\ in^2$

Minimum Required Steel per ACI

$(A_{s.min})_a := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{(F_y)_{60}} \cdot b \cdot d_{bslab.lat} = 0.543\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(a))*

$(A_{s.min})_b := \frac{200\ psi}{(F_y)_{60}} \cdot b \cdot d_{bslab.lat} = 0.54\ in^2$ *(minimum area of steel required ACI 318-19, Section 9.6.1.2(b))*

Temperature and Shrinkage Reinforcement

$(A_{s.min})_{temp} := 0.5 \cdot 0.003 \cdot (BSlab_{thick} \cdot b) = 0.324\ in^2$ *(minimum temperature and shrinkage steel, EM 1110-2-2104, Section 2-9(a))*

Note: Per ACI 318-19Section 9.6.1.3, if the area of steel provided is 1.333 times the area of steel required by analysis than 9.6.1.1 and 9.6.1.2 of ACI need not apply

$A_{s.min} := \max\left((A_{s.min})_a, (A_{s.min})_b, (A_{s.min})_{temp}\right)$ *(minimum area of steel required for flexural capacity)*

$A_{s.min} = 0.543\ in^2$

$A_{s\_1.3} := (A_{s.min})_{bslab.ten} \cdot \frac{4}{3} = 0.909\ in^2$ *(1/3 greater than steel required, ACI 318-19, Section 9.6.1.3)*

$(A_s)_{bslab.req} := \text{if } A_{s\_1.3} < (A_s)_{bslab.lat} \parallel as \leftarrow (A_{s.min})_{bslab.ten} \text{ else } \parallel as \leftarrow A_{s.min} = 0.543\ in^2$ *(required steel to meet steel requirements)*

(includes ACI 318-19 9.6.1.2(a) and (b))

(excludes ACI 318-19 9.6.1.2(a) and (b))

$(A_s)_{bslab.lat} = 0.8\ in^2$ *(area of steel provided in the vertical direction for the stem)*

$(Steel\_Check_{lat})_{BSlab} := Check\left((A_s)_{bslab.req}, (A_s)_{bslab.lat}\right) = [\text{"OK"} \quad 0.7]$

(check slab reinforcement to required reinforcement)

Maximum Tensile Reinforcement, Minimum Spacing

$\rho_{max} := 0.25 \cdot \left(\frac{0.85 \cdot \beta_1 \cdot f'_c}{(F_y)_{60}}\right) \cdot \left(\frac{87000}{87000 + \left((F_y)_{60} \cdot psi^{-1}\right)}\right) = 0.008$ *(reinforcement ratio from EM 1110-2-2104)*

$\rho_{max} = 0.0078$ *(maximum reinforcement ratio for concrete section)*

$\rho_{actual} := \frac{(A_s)_{bslab.lat}}{(b \cdot d_{bslab.lat})} = 0.0048$ *(actual reinforcement ratio for concrete section)*

$(\rho\_check_{lat})_{BSlab} := Check\left(\rho_{actual}, \rho_{max}\right) = [\text{"OK"} \quad 0.6]$ *(check that actual reinforcement ratios are lower than the maximum reinforcement ratio)*

Shear Check

$V_u := (V_{23})_{botslab} = 69.7\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{bslab.long} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{bslab.long} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$ *(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)*

$A_V := (A_s)_{bslab.long} + (A_s')_{bslab.long} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot BSlab_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \frac{A_V}{b \cdot d_{bslab.long}} = 0.009$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \frac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\frac{2}{1 + \frac{d_{bslab.long}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \quad \left\| \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{bslab.long} \right. \quad \text{else} \quad \left\| \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{bslab.long} \right. = 23.3\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_c := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{bslab.long}\right) = 23.3\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs} := \phi_V \cdot V_c = 17.5\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_u > \phi V_{cs} \quad \left\| \text{“Shear strength from steel required”} \right. \quad \text{else} \quad \left\| \text{“Shear strength from steel not required”} \right. = \text{“Shear strength from steel required”}$

$V_s := \dfrac{A_V \cdot (F_y)_{60} \cdot d_{bslab.long}}{s_{bslab.long}} = 113.9\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu := \phi_V \cdot \left(V_c + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{bslab.long}\right)\right) = 87.5\ kip$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_u, Vu\right) = [\text{“OK”} \quad 0.8]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn := \phi_V \cdot \left(V_c + V_s\right) = 102.9\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_u = 69.7\ kip$

$(Check_v)_{BSlab} := Check\left(V_u, \phi_V Vn\right) = [\text{“OK”} \quad 0.7]$

Lateral Shear Check

$V_{u.lat} := (V_{12})_{botslab} = 49.4\ kip$ *(shear in member, from Ram Elements output)*

$N_u := 0\ kip$ *(axial load in member, from Ram Elements output)*

$Av,min := \max\left(s_{bslab.lat} \cdot \left(0.75 \cdot \sqrt{f'c} \cdot psi \cdot \frac{b}{(F_y)_{60}}\right), s_{bslab.lat} \cdot \left(50 \cdot psi \cdot \frac{b}{(F_y)_{60}}\right)\right) = 0.1\ in^2$

(minimum shear reinforcement for section, ACI 318-19, Table 9.6.3.4)

$A_V := (A_s)_{bslab.lat} + (A_s')_{bslab.lat} = 1.6\ in^2$ *(effective area of steel in section, ACI 318-19, sec 22.5.8.5.5)*

$A_g := b \cdot BSlab_{thick} = 216\ in^2$ *(gross area of concrete)*

$\rho_w := \dfrac{A_V}{b \cdot d_{bslab.lat}} = 0.0097$ *(ratio of steel area to gross concrete area)*

Shear Strength in Concrete

$(N_u/6A_g) := min\left(0.05 \cdot f'_c, \dfrac{N_u}{6 \cdot A_g}\right) = 0\ psi$ *(limit on axial factor per ACI 318-19 22.5.5.1.2)*

$\lambda_s := min\left(\sqrt{\dfrac{2}{1 + \dfrac{d_{bslab.lat}}{10\ (in)}}}, 1.0\right) = 0.9$ *(size modification factor per ACI 318-19 22.5.5.1.3)*

$V_c := \text{if } A_V \geq Av,min \;\|\; \left(2 \cdot \lambda \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{bslab.lat}$ else $\|\; \left(8 \cdot \lambda_s \cdot \lambda \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'c} \cdot psi + (N_u/6A_g)\right) \cdot b \cdot d_{bslab.lat} = 21.7\ kip$

(concrete shear strength per ACI 318-19 Table 22.5.5.1)

$V_{c.lat} := min\left(V_c, 5 \cdot \lambda \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{bslab.lat}\right) = 21.7\ kip$ *(limiting shear strength in concrete per ACI 318-19 22.5.5.1.1)*

$\phi V_{cs.lat} := \phi_V \cdot V_{c.lat} = 16.3\ kip$ *(factored shear strength in concrete per ACI 318-19 sec 22.5)*

Shear Strength in Steel

Note: where the concrete shear strength is not enough to meet the required strength, the shear strength of the steel is considered.

$V_{s\_check} := \text{if } V_{u.lat} > \phi V_{cs.lat} \;\|\; \text{“Shear strength from steel required”}$ else $\|\; \text{“Shear strength from steel not required”} = \text{“Shear strength from steel required”}$

$V_{s.lat} := \dfrac{A_V \cdot (F_y)_{60} \cdot d_{bslab.lat}}{s_{bslab.lat}} = 106\ kip$ *(steel shear strength per ACI 318-19, Eq 22.5.8.5.3)*

Dimensional Cross Section Check

$Vu_{lat} := \phi_V \cdot \left(V_{c.lat} + \left(8 \cdot \sqrt{f'c} \cdot psi \cdot b \cdot d_{bslab.lat}\right)\right)$ *(limit of cross section per ACI 318-19, Eq 22.5.1.2)*

$Check_{Cross\_sec} := Check\left(V_{u.lat}, Vu_{lat}\right) = [\text{“OK”} \quad 0.6]$ *(dimensional cross section check per ACI 318-19, Eq 22.5.1.2)*

Shear Capacity Check

$\phi_V Vn_{lat} := \phi_V \cdot \left(V_{c.lat} + V_{s.lat}\right) = 95.8\ kip$ *(section combined shear strength, ACI 318-19, Eq 22.5.1.1)*

$V_{u.lat} = 49.4\ kip$

$(Check_v)_{BSlab.lat} := Check\left(V_{u.lat}, \phi_V Vn_{lat}\right) = [\text{“OK”} \quad 0.5]$

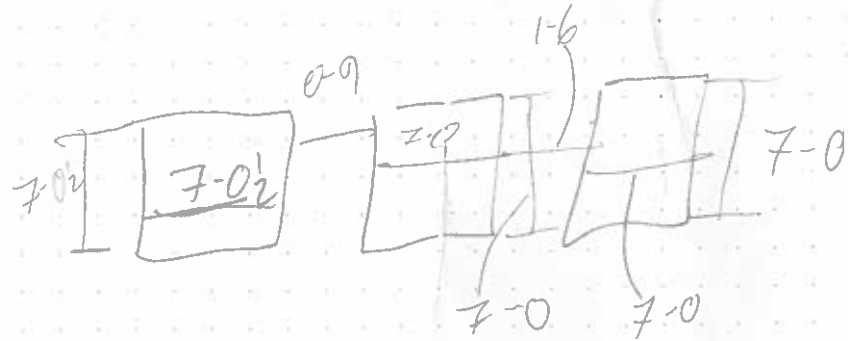
BOTTOM SLAB	Bar Size	Spacing (in)
Vertical Bars		
Front Face	#8	12
Back Face	#8	12
Lateral Bars		
Front Face	#8	12
Back Face	#8	12

	Ma	Mn	Check	IC
Vertical Flexural Check	41.27	49.38	OK	0.84
Lateral Flexural Check	39.84	45.84	OK	0.87

	Va	Vn	Check	IC
Vertical Shear Check	69.72	102.92	OK	0.68
Lateral Shear Check	49.36	95.82	OK	0.52

Quaker Creek Box Design Summary

				M_{maj},a		V_{maj}		T		Min Steel Check		Max Steel Check	
Component	Bar Size	Spacing in	Cover in	Check	IC	Check	IC	Check	IC	Check	IC	Check	IC
Stem Wall Vert	#8	12.0	3.0	OK	0.7	OK	0.5			OK	0.7	OK	0.6
Stem Wall Lateral	#8	12.0	3.0	OK	0.8	OK	0.3			OK	0.7	OK	0.6
Wing Wall Vert	#8	12.0	3.0	OK	0.8	OK	0.3			OK	0.7	OK	0.6
Wing Wall Lateral	#8	12.0	3.0	OK	0.9	OK	0.3			OK	0.7	OK	0.6
Box Wall Vert	#8	12.0	4.0	OK	0.8			OK	0.0	OK	0.7	OK	0.6
Box Wall Lateral	#8	12.0	4.0	OK	0.5	OK	0.2			OK	0.5	OK	0.7
Buttress Wall Vert	#8	12.0	3.0	OK	0.3			OK	0.6	OK	0.3	OK	0.6
Buttress Wall Lateral	#8	12.0	3.0	OK	0.4	OK	0.1			OK	0.3	OK	0.6
Top Slab Longitudinal	#8	12.0	3.0	OK	0.5	OK	0.2			OK	0.5	OK	0.6
Top Slab Lateral	#8	12.0	3.0	OK	0.5	OK	0.2			OK	0.5	OK	0.6
Bottom Slab Longitudinal	#8	12.0	3.0	OK	0.8	OK	0.7			OK	0.7	OK	0.6
Bottom Slab Lateral	#8	12.0	3.0	OK	0.9	OK	0.5			OK	0.7	OK	0.6



7-0½
7-0½
0-9
7-0
1-6
7-0
7-0
7-0

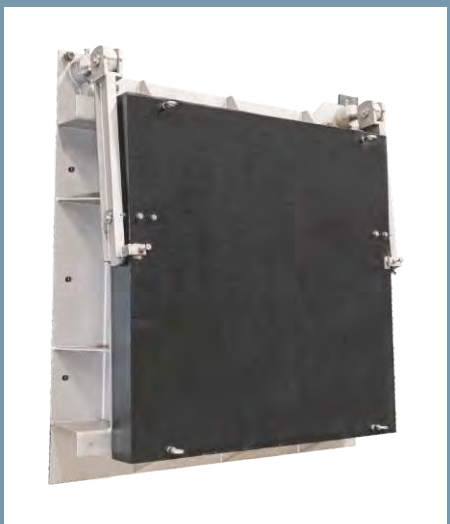
RODNEY HUNT
SINCE 1840

Rodney Hunt

Water & Sewage Control Equipment








FLAP GATES

Backflow Prevention in Free Discharge Application

THE BIGGEST CAST IRON GATES MANUFACTURERS OF THE WORLD

ENGINEERING EXCELLENCE AND EXPERIENCE

Since 1840, the engineering team at Rodney Hunt has pioneered safe and reliable flow control systems in thousands of applications around the world. In 2016, Rodney Hunt joined forces with Jash Engineering Ltd., a company with 68 years experience in manufacturing of flow control products and having a leadership position in Asia. Together, we bring more than 250 years of experience designing customized solutions to meet the needs of the most challenging applications worldwide.

COMPREHENSIVE PRODUCT OFFERING

Our product offering is unrivalled in the flow control marketplace. From all types and sizes of cast and fabricated gates, in a wide variety of material options encompassing metals and plastics, to custom valve and actuation options, we bring a total solution to every project. The breadth of our product offering enables us to bring unequaled expertise to your planning and decision making process.

SUPERIOR QUALITY

We bring exceptional quality to every project using application based solutions, proven designs, manufacturing flexibility and rigorous testing procedures. We offer one of the most flexible and comprehensive cast, metal fabrication, machining, assembly and testing operations at our in-house facilities in the USA and India. This allows us to monitor and ensure quality in all aspects of production and to provide consistent reliable and superior products.

RESPONSIVE SERVICE

We pride ourselves on providing professional responsiveness to your needs throughout the design, manufacturing and installation processes. Our engineering team is available for consultation during all phases of your project. Dedicated project managers serve as a single point of contact once the order is in-house and our knowledgeable field service team is always ready to provide on-site support.



Two 60" (1500 mm) Series 60 Flap Gates help protect a North Dakota watershed from forward and reverse surges.

FLAP GATES & FEATURES

RODNEY HUNT
SINCE 1840

BACKFLOW PREVENTION IN FREE DISCHARGE APPLICATIONS:

Rodney Hunt flap gates (or flap valves) are designed as an efficient backflow prevention solution in most free discharge application. These are often used for automatic drainage from outfall structures to rivers, estuaries and sea & for preventing reverse flow during flood / tide from rivers, estuaries and sea to the plant.

These are gravity operated and need minimal water pressure for operation. The flap opens when the head in the pipe or conduit exceeds 0.2 feet above the head in the receiving body of water. The flap closes when the head in the receiving body of water exceeds the head in the pipe or conduit. When fully open the flap gate/valve offers a very low head-loss.

Rodney Hunt offers flap gates / valves with many different options and features as shown hereunder to suit specific applications:

OPENING AND SIZE OPTIONS:

Round, square and rectangular shaped opening as per client's requirement in a wide range of sizes starting from 4 inch to 144 inch and greater.

MATERIAL OPTIONS:

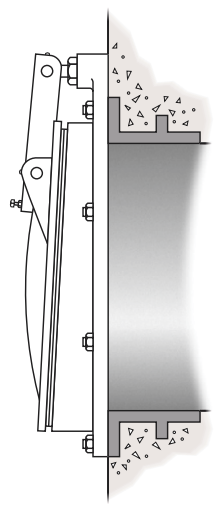
Cast Iron / Cast Ductile Iron / Alloy Cast Iron / Fabricated Stainless Steel / Aluminum, HDPE or Composite construction depending upon application, head and fluid condition.

MULTI-FLAP / DOOR OPTION:

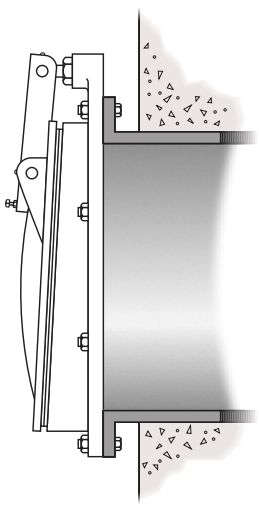
Multiple doors to cover the opening in large sized flap gates, when specified.

MOUNTING OPTIONS:

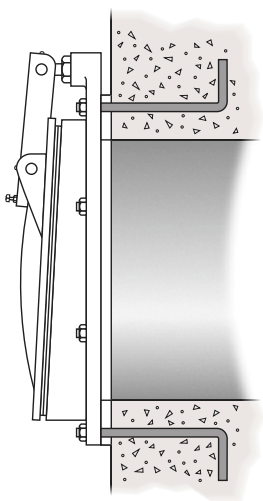
Various mounting options available such as Wall thimble mounted / Pipe flange mounted / Wall mounted as shown below.



Wall Thimble Mounted



Pipe Flange Mounted



Wall Mounted

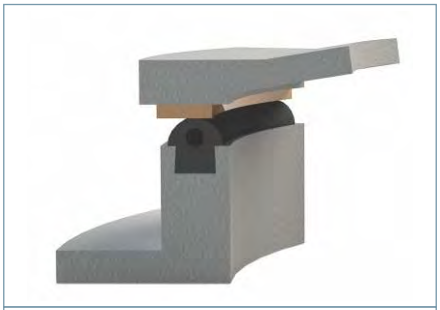
FLAP GATES & FEATURES

SEALING OPTIONS:

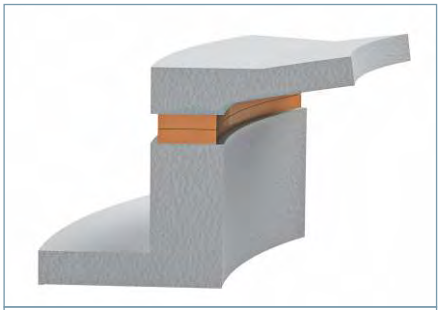
Depending upon material of construction of flap gate, varied sealing options as shown below are offered for achieving low leakage in valve closed condition.



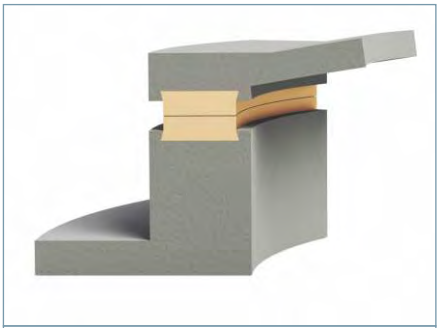
Cast Iron to Rubber for
Cast Iron Flap Gate



Bronze to Rubber for
Cast Iron Flap Gate



Metal to Metal Fitted on Plain Face
for Cast Iron Flap Gate



Metal to Metal in Dovetail Groove
for Cast Iron Flap Gate



Metal to Rubber for
Fabricated Flap Gate



HDPE to Rubber for
HDPE Flap Gate

HINGE OPTIONS:

Depending upon requirement and size single or double hinged design is offered. Both single hinged or double hinged designs can be provided with double pivot to increase sensitivity.



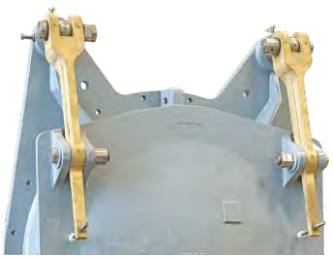
Single Hinge Double Pivot



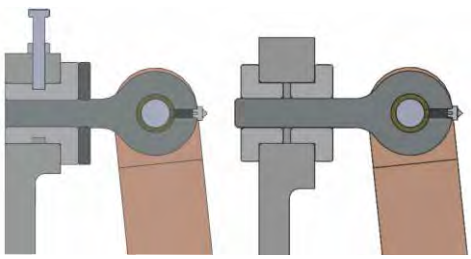
Double Hinge Double Pivot

ADJUSTABLE UPPER AND LOWER PIVOT OPTIONS:

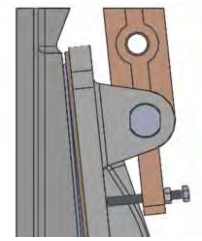
Adjustable upper pivot is provided to adjust the flap gates sensitivity to unseating head. The lower pivot is provided to allow for a controlled amount of cover rotation to ensure that cover seat and body seat lie in the same plane. Adjustable upper pivot & lower pivot are provided generally in large sized flap gates.



Adjustable Upper & Lower Pivot



Upper Pivot Options



Lower Pivot

ANTI LOCKING BAR & BUMPER SPRING OPTION:

- Anti-locking bar and bumper spring options are used in pump discharges application.
- Anti-locking bar is provided to prevent flap / cover overturn damage, when specified.
- Bumper is given in case of pump discharge application, to prevent flap from traveling over 90° and hitting the wall.



Anti Locking Bar Option



Bumper Spring Option

Rodney Hunt has the capability to design and manufacture flap gates for varied applications as requested by client and incorporate custom features to suit the application. Rodney Hunt offers the widest range of flap gates in the industry as show cased in the subsequent pages.

CAST IRON FLAP GATES

SERIES: A - 601

SINGLE HINGED - DOUBLE PIVOT



DOUBLE HINGED - DOUBLE PIVOT



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

These flap gates are suitable for back flow prevention applications and suitable for seating head up to 20 feet.

SIZE RANGE:

- From 4 inch to 30 inch.

FEATURES:

- Frame / body suitable for directly mounting on face of the wall using anchor fasteners and secondary grout between wall and frame / body or on wall thimble or on pipe flange.
- Single piece flap cover to withstand applicable head.
- Resilient seal to absorb closing shocks.
- Resilient seal mounted in machined groove on frame / body and in contact with mating face of flap cover.
- Double pivot design for all sizes of flap gate.
- Single hinge design upto flap gate size 10" & double hinge design above 10".
- Tamper proof locking arrangement for hinge pin.

OPTIONAL FEATURES:

- Wall thimble mounting.
- Mounting flange drilling pattern to match 25 lb. or 125 lb. as required.
- Mating face of flap cover provided with non corrosive seat to match resilient seal on frame / body.
- Metallic sealing arrangement comprising of non ferrous seats mounted on frame / body in contact with matching non ferrous seat mounted on flap cover.

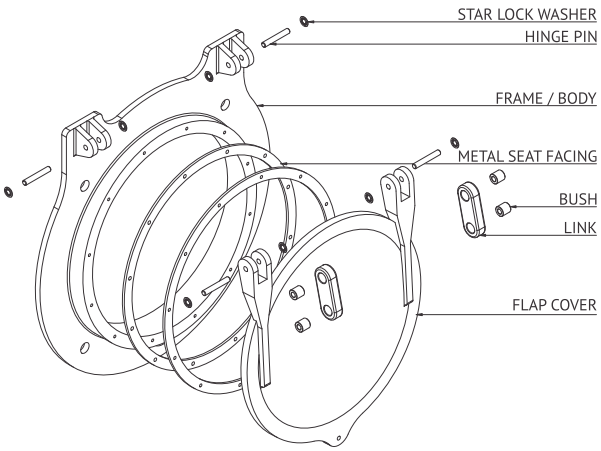
MATERIAL OF CONSTRUCTION:

Frame / Body	Cast Iron ASTM A126 Class B
Flap Cover	Cast Iron ASTM A126 Class B
Frame / Body Seat Facing	Neoprene ASTM D-2000 or Bronze B21 – C464
Flap Cover Seat Facing	Cast Iron ASTM A126 Class B or Bronze B21 C464
Hinges	Stainless Steel 304
Fasteners	Stainless Steel 304 / 316

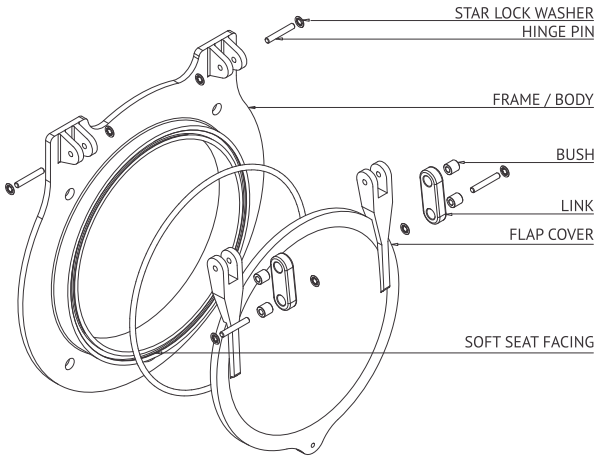
SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

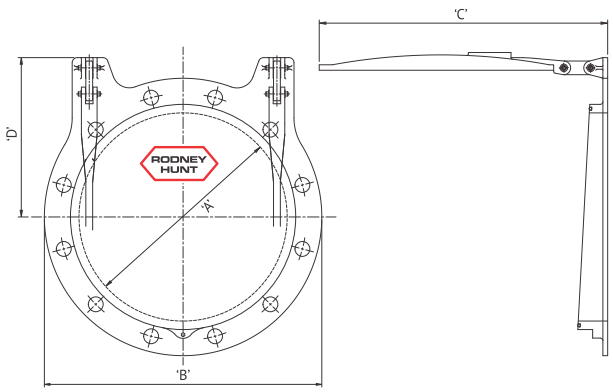
DRAWING AND DATASHEET



EXPLODED VIEW - METAL SEATED



EXPLODED VIEW - SOFT SEATED



FLAP GATE DIMENSIONS (IN INCH)			
Flap Gate Size - A	**B**	**C**	**D**
4	9	10 - 3/8	6
6	11	12 - 3/8	7 - 1/4
8	13 - 1/2	14 - 3/8	8 - 1/4
10	16	16 - 3/8	9 - 7/8
12	19	18 - 3/8	11
14	21	20 - 3/8	12 - 1/4
16	23 - 1/2	22 - 3/8	13
18	25	24 - 3/8	13 - 7/8
20	27 - 1/2	26 - 3/8	14 - 1/2
24	32	30 - 3/8	18 - 1/4
30	38 3/4	36 - 1/2	23 - 1/2

CAST IRON FLAP GATES

SERIES: A - 602 (FV - AC & FV - ACP)

DOUBLE HINGED - DOUBLE PIVOT FLAP GATE



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

These flap gates are suitable for back flow prevention applications and suitable for high seating head condition as per requirement.

SIZE RANGE:

- From 12 inch to 108 inch for FV-AC design.
- From 12 inch to 60 inch for FV-ACP design.
- Higher sizes can be offered on request.

FEATURES:

- Frame / body suitable for directly mounting on face of the wall using anchor fasteners and secondary grout between wall and frame / body or on wall thimble or on pipe flange.
- Single piece flap cover to withstand high seating head.
- Sensitive to low unseating head – opens at 0.2 feet.
- Resilient seal to absorb closing shocks.
- Resilient sealing arrangement comprising of resilient seal mounted in machined groove on frame/body and in contact with mating face of flap cover.
- Double pivot design for all sizes of flap gate.
- Adjustable upper and lower pivot to adjust sensitivity to unseating head.
- Rigidly designed hinge arms/brackets.
- Adjustable hinge bolts of toughened steel to reduce / increase sensitivity of flap cover operation.
- Replaceable hinge pins and hinge bushes.

ADDITIONAL FEATURES FOR FV-ACP DESIGN TO BE USED FOR PUMP DISCHARGE APPLICATION:

- Anti-locking bar to prevent flap overturn damage.
- Spring bumper to limit disc travel to 90 degrees.

OPTIONAL FEATURES:

- Wall thimble mounting.
- Mounting flange drilling pattern to match 25 lb. or 125 lb. as required.
- Mating face of flap cover provided with non corrosive seat to match resilient seal on frame / body.
- Metallic sealing arrangement comprising of non ferrous seats mounted on frame / body in contact with matching non ferrous seat mounted on flap cover.

MATERIAL OF CONSTRUCTION:

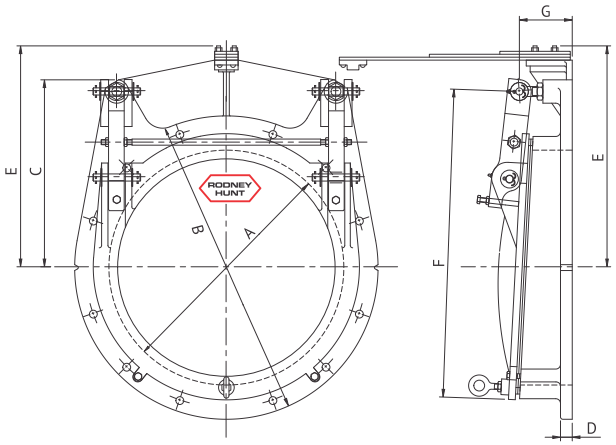
Frame / Body	Cast Iron ASTM A126 Class B
Flap Cover	Cast Iron ASTM A126 Class B
Frame / Body Seat Facing	Neoprene ASTM D-2000 or Bronze B21 – C464
Flap Cover Seat Facing	Neoprene ASTM D-2000 or Cast Iron ASTM A126 Class B or Bronze B21 – C464
Hinge Arm	Ductile Iron ASTM A536 65-45-12 / Bronze B98 – C655
Hinge Bolt	Toughened Steel
Anti-Locking Bar	Steel
Bumper Spring	Heat Treated Spring Steel A 689
Fasteners	Stainless Steel 304 / 316

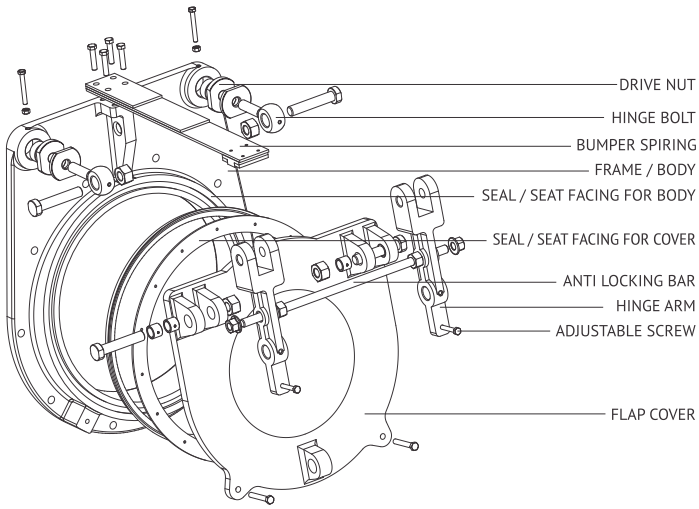
SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

DRAWING AND DATASHEET

RODNEY HUNT
SINCE 1840





FLAP GATE DIMENSIONS (IN INCH)						
Flap Gate Size - A	**B**	**C**	**D**	**E**	**F**	**G**
12	19	9 - 3/4	1	13 - 1/2	16 - 1/4	3
14	21	12	1 - 1/8	15 - 1/2	19 - 1/2	3 - 1/4
16	23 - 1/2	12 - 3/4	1 - 1/8	17 - 1/8	21 - 1/4	3 - 1/4
18	25	14 - 1/2	1 - 1/4	18 - 5/8	24	3 - 1/2
20	27 - 1/2	16 - 1/8	1 - 1/4	17 - 7/8	16 - 1/2	5
24	32	19 - 1/2	1 - 3/8	22 - 3/4	31 - 3/4	5 - 1/2
30	38 - 3/4	24	1 - 1/2	28 - 5/8	39 - 1/2	6
36	46	28 - 1/2	1 - 5/8	32 - 3/4	47	6 - 1/2
42	53	33	1- 3/4	36 - 1/2	54 - 1/2	7
48	59 - 1/2	38	2	41 - 1/2	62	7 - 1/2
54	66 - 1/4	42 - 1/2	2 - 1/4	46 - 3/4	69 - 1/2	8 - 1/2
60	73	47	2 - 1/4	52	77 - 1/4	8 - 1/2
66	79	51 - 1/2	2 - 1/4	55 - 1/2	85 - 1/4	8 - 1/2
72	86 - 1/2	54 - 1/8	2 - 1/2	--	92 - 3/4	8 - 1/2
78	93 - 1/8	60 - 1/2	2 - 1/2	--	100 - 1/2	8 - 1/2
84	99 - 3/4	65 - 1/4	2 - 5/8	--	108	9 - 1/2
96	113 - 1/4	74 - 1/2	2 - 3/4	--	123	9 - 1/2
108	128 - 1/2	83 - 1/2	3	--	138	10 - 7/8

CAST IRON FLAP GATES

SERIES: A - 603 (FV - AR)

DOUBLE HINGED - DOUBLE PIVOT FLAP GATE



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

These flap gates are suitable for back flow prevention applications and suitable for high seating head condition as per requirement.

SIZE RANGE:

- From 12 inch to 108 inch.
- Higher sizes can be offered on request.

FEATURES:

- Square / Rectangular flange back frame / body for directly mounting on face of wall using anchor fasteners and secondary grout between wall & frame / body or to wall thimble.
- Single piece flap cover to withstand high seating head.
- Sensitive to low unseating head – opens at 0.2 feet.
- Resilient seal to absorb closing shocks.
- Resilient sealing arrangement comprise of resilient seal mounted on frame / body and in contact with mating face of flap cover.
- Double pivot design for all sizes of flap gate.
- Adjustable upper and lower pivot to adjust sensitivity to unseating head.
- Rigidly designed hinge arms/brackets.
- Adjustable hinge bolts of toughened steel to reduce / increase sensitivity of flap cover operation.
- Replaceable hinge pins and hinge bushes.

OPTIONAL FEATURES:

- Wall thimble mounting.
- Mating face of flap cover provided with non corrosive seat to match resilient seal on frame / body.
- Metallic sealing arrangement comprising of non ferrous seats mounted on frame / body in contact with matching non ferrous seat mounted on flap cover.
- Multiple flap covers to cover the opening in large sized flap gates, when specified.

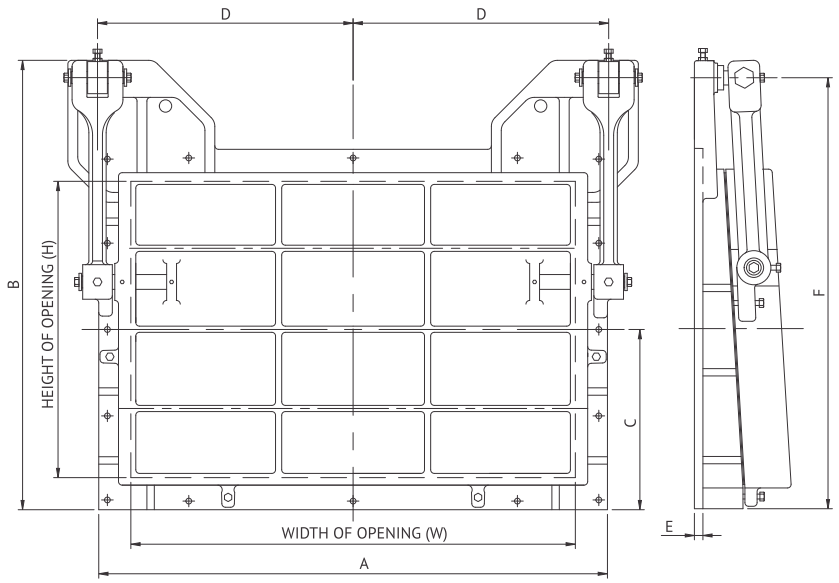
MATERIAL OF CONSTRUCTION:

Frame / Body	Cast Iron ASTM A126 Class B
Flap Cover	Cast Iron ASTM A126 Class B
Frame / Body Seat Facing	Neoprene ASTM D-2000 or Bronze B21 – C464
Flap Cover Seat Facing	Neoprene ASTM D-2000 or Cast Iron ASTM A126 Class B or Bronze B21 – C464
Hinge Arm	Bronze B98 – C655
Hinge Bolt	Toughened Steel
Fasteners	Stainless Steel 304 / 316

SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

RODNEY HUNT
SINCE 1840



FLAP GATE DIMENSIONS (IN INCH)							
WIDTH (W)	HEIGHT (H)	A	B	C	D	E	F
12	12	19	19 - 1/4	9 - 1/2	7 - 3/4	1	18 - 3/4
18	18	25	27	12 - 1/2	11 - 1/16	1 - 1/4	24 - 1/2
24	24	32	35 - 1/2	16	14 - 9/16	1 - 3/8	32 - 3/4
30	30	36	43	19	18 - 1/16	1 - 5/8	40 - 1/2
36	24	45	36	18 - 1/2	21	1 - 1/2	32 - 3/4
36	36	45	51	22 - 1/2	21 - 1/4	1 - 5/8	47
36	48	45	66 - 3/4	28 - 3/4	21 - 1/4	2	62
36	54	46	72 - 1/2	32	21 - 1/2	2 - 1/4	69 - 1/2
42	42	51 - 1/2	58 - 3/4	25 - 3/4	24 - 1/4	1 - 3/4	54 - 1/2
48	18	56	28	13	26 - 13/16	1 - 5/8	24 - 1/2
48	24	56	35 - 1/2	16	26 - 13/16	1 - 5/8	32 - 3/4
48	30	56	43	19	27 - 1/16	1 - 5/8	40 - 1/2
48	36	56	51	22 - 1/2	27 - 1/16	1 - 5/8	47
48	48	57 - 1/2	66 - 3/4	28 - 3/4	27 - 1/2	2	62
48	60	57 - 1/2	81 - 3/4	34 - 3/4	27 - 1/2	2 - 1/4	77 - 3/4
54	54	64	74 - 1/2	32	30 - 1/2	2 - 1/4	69 - 1/2
60	36	69 - 1/2	51 - 1/2	22 - 3/4	33 - 1/4	1 - 3/4	47
60	48	69 - 1/2	66 - 3/4	28 - 3/4	33 - 1/4	2 - 1/4	62
60	60	69 - 1/2	81 - 3/4	34 - 3/4	33 - 1/2	2 - 1/4	77 - 3/4
60	72	69 - 1/2	95 - 1/2	41 - 1/2	33 - 1/2	2 - 1/4	93 - 3/4
72	48	81 - 1/2	64 - 3/4	28 - 3/4	39 - 1/2	2 - 1/4	62
72	60	83	82 - 1/2	35 - 1/2	39 - 1/2	2 - 1/4	77 - 3/4
72	72	83	95 - 1/2	41 - 1/2	40 - 1/4	2 - 1/2	93 - 3/4
84	84	95	112 - 1/2	47 - 1/2	46 - 1/2	2 - 3/4	107 - 3/4
96	96	107	124 - 1/2	53 - 1/2	52 - 1/2	2 - 3/4	122 - 3/4
108	108	120	138	60	58 - 1/2	2 - 3/4	135

FABRICATED FLAP GATES

SERIES: A - 611 & A - 612

SERIES: A - 611



SERIES: A - 612



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

These flap gates are suitable for back flow prevention applications & suitable for high seating head condition as per requirement.

SIZE RANGE:

- From 12 inch to 108 inch.
- Higher sizes can be offered on request.

SERIES:

- **A-611:** Round flange back frame/body for directly mounting on face of wall using anchor fasteners and secondary grout between wall and frame/body or on the wall thimble or on the pipe flange.
- **A-612:** Square / Rectangular flange back frame / body for mounting on face of wall using anchor fasteners and secondary grout between wall and frame/body or to wall thimble.

FEATURES:

- Single piece flap cover to withstand applicable head.
- Resilient seal to absorb closing shocks.
- Resilient sealing arrangement comprise of resilient seal mounted on outer peripheral edge of frame/body and in contact with mating face of flap cover.
- Single or double pivot design depending upon size, sensitivity require per the application.
- Single hinge design upto flap gate size 10" & double hinge design above 10".
- Sizes 30" and above provided with rigidly designed hinge arms / brackets having provision for adjustment of the flap gate sensibility to unseating head.
- Replaceable hinge pins and self lubricating hinge bushings.

OPTIONAL FEATURES:

- Wall thimble mounting.
- Mounting flange drilling pattern to match 25 lb. or 125 lb. as required.
- Material can be offer in Aluminum.

MATERIAL OF CONSTRUCTION:

Frame / Body	Stainless Steel ASTM A240-304L/316L
Flap Cover	Stainless Steel ASTM A276-304L / 316L
Hinge Arm	Stainless Steel ASTM A276-304L / 316L
Seals	Neoprene ASTM D-2000
Hinge Bushing	UHMWPE ASTM D-4020
Hinge Bolt	Toughened Steel
Fasteners	Stainless Steel 304 / 316

Higher grade of stainless steel can be offered on request.

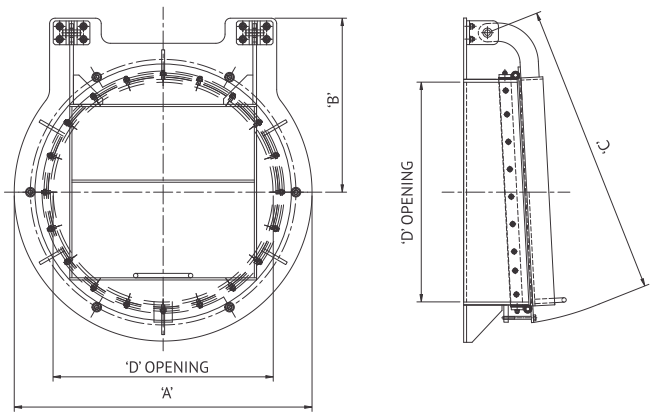
SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

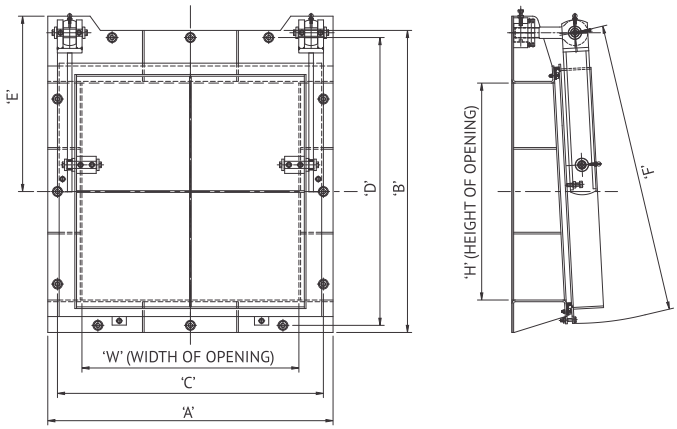
DRAWING AND DATASHEET

RODNEY HUNT
SINCE 1840

SERIES: A - 611



SERIES: A - 612



FLAP GATE DIMENSIONS (IN INCH)					
D	A			B	C
	CW, WT	CWX-RCP	CWX		
6	11	-	17	7 - 1/2	8
8	13 - 1/2	24 - 3/4	19 - 1/8	8 - 1/2	11 - 15/32
10	16	26 - 3/4	21 - 1/8	9 - 1/2	13 - 7/16
12	20	26 - 1/2	23 - 1/4	11 - 1/2	15
15	23	29 - 1/2	26 - 3/8	13	18
18	26	33	29 - 1/2	14 - 1/2	22 - 3/4
21	29	36 - 1/2	35 - 5/8	16	24 - 1/2
24	32	39 - 1/2	36	18 - 1/2	27 - 1/8
30	38 - 3/4	47 - 1/2	42	25	40 - 1/2
36	46	55 - 1/2	48 - 1/2	30	46 - 1/4
42	53	62 - 1/2	54 - 1/2	34	52 - 1/4
48	59 - 1/2	72 - 1/2	62 - 7/8	37	58 - 1/2
54	66 - 1/4	78 - 1/2	69 - 1/8	41	64 - 1/2
60	73	85 - 1/2	-	53	67 - 1/2
72	86 - 1/2	98	-	60	78 - 1/2
84	99 - 3/4	113 - 1/2	-	66	94 - 1/2
96	106	129	-	73	107 - 1/16

FLAP GATE DIMENSIONS (IN INCH)							
W	H	A	B	C	D	E	F
12	12	20	20	18	18	-	16
16	16	24	24	22	22	-	20
18	18	26	26	24	24	-	22
21	21	29	29	27	27	-	25
24	24	32	32	30	30	-	28
30	30	38	38	36	36	21 - 1/8	36 - 3/4
32	32	40	40	38	38	22 - 1/8	38 - 3/4
36	36	44	44	42	42	24 - 1/8	42 - 3/4
40	40	48	48	46	46	26 - 1/8	46 - 3/4
42	42	50	50	48	48	27 - 1/8	48 - 3/4
48	48	56	56	54	54	30 - 1/8	54 - 3/4
54	54	65	65	62	62	33 - 1/8	60 - 3/4
60	60	71	71	68	68	36 - 3/4	67 - 3/8
72	72	83	83	80	80	42 - 3/4	79 - 3/8
84	84	95	95	92	92	48 - 3/4	91 - 3/8
96	96	107	107	104	104	56 - 3/4	107 - 1/16

CW: Concrete Wall Mounted

WT: Wall Thimble Mounted

CWX RCP: Concrete Wall Mounted Termination of Concrete Pipe

CWX: Concrete Wall Mounted Termination of Pipe

HDPE FLAP GATES

SERIES: A - 641 & A - 642

SERIES: A - 641



SERIES: A - 642



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

For low cost back flow prevention applications in low head conditions. HDPE material enable light weight construction thereby offering ease in installation and also high corrosion resistance, longevity and minimum maintenance.

SIZE RANGE:

- From 12 inch to 60 inch.
- Higher sizes can be offered on request.

SERIES:

- **A-641:** Round flange back frame / body for directly mounting on face of wall using anchor fasteners and secondary grout between wall and frame / body or on the wall thimble or on the pipe flange.
- **A-642:** Square / Rectangular flange back frame / body for mounting on face of wall using anchor fasteners and secondary grout between wall & frame/body or to wall thimble.

FEATURES:

- Single piece flap cover to withstand applicable head. Door may also have stainless steel stiffening depending upon size.
- Resilient seal to absorb closing shocks.
- Resilient sealing arrangement comprise of resilient seal mounted on outer peripheral edge of frame/body and in contact with mating face of flap cover.
- Single or double pivot design depending upon size, sensitivity required and application.
- Single or double hinged with hinge sized to prevent formation of oval hole due to higher applying forces.
- Replaceable hinge pins and self lubricating hinge bushings.

OPTIONAL FEATURES:

- Wall thimble mounting.
- Mounting flange drilling pattern to match 25 lb. or 125 lb. as required.
- Stainless Steel frame can be offered on request.

MATERIAL OF CONSTRUCTION:

Frame / Body	HDPE ASTM D-4020 / Stainless Steel ASTM A240-304L / 316L
Flap Cover	HDPE ASTM D-4020
Stiffners	HDPE / Stainless Steel
Hinge arm	Stainless Steel ASTM A240-304L / 316L
Seals	Neoprene ASTM D-2000
Hinge Bushing	UHMWPE ASTM D-4020
Hinge Pin & Washers	Stainless Steel 304
Fasteners	Stainless Steel 304 / 316

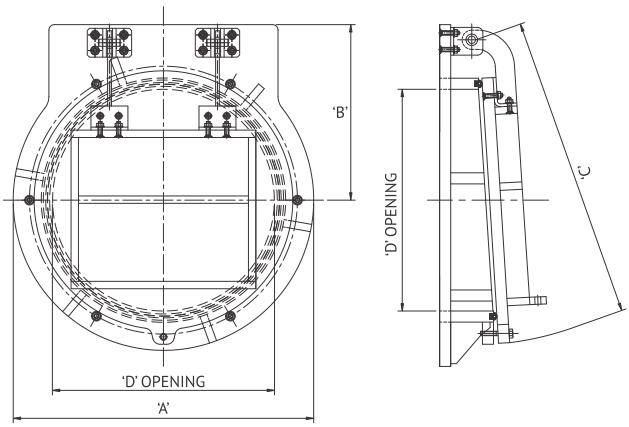
SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

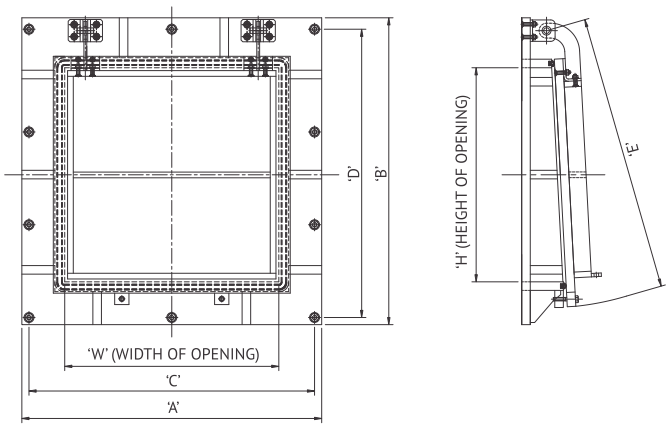
DRAWING AND DATASHEET

RODNEY HUNT
SINCE 1840

SERIES: A - 641



SERIES: A - 642



FLAP GATE DIMENSIONS (IN INCH)			
D	A	B	C
6	16	8-1/2	13
8	18	9-1/2	15
10	20	10-1/2	17
12	22	12-1/2	19
15	25	13	24
18	28	15-1/2	27
21	31	16-1/2	30
24	34	19	33
30	40	25	39
36	46	30	45
42	52	34	51
48	58	37	57
54	64	41	63
60	70	53	69

FLAP GATE DIMENSIONS (IN INCH)						
W	H	A	B	C	D	E
12	12	22	25 - 5/8	20	23 - 5/8	20 - 1/2
16	16	26	29 - 5/8	24	27 - 5/8	24 - 1/2
18	18	28	31 - 5/8	26	29 - 5/8	26 - 1/2
21	21	31	36 - 1/8	29	33 - 5/8	31 - 1/2
24	24	34	38 - 5/8	32	37 - 5/8	34 - 1/2
30	30	40	44 - 5/8	38	42 - 5/8	36 - 1/2
32	32	42	47 - 5/8	40	45 - 5/8	40 - 1/2
36	36	46	51 - 5/8	44	49 - 5/8	44 - 1/2
40	40	50	55 - 5/8	48	53 - 5/8	48 - 1/2
42	42	52	58	50	56 - 5/8	52 - 1/2
48	48	58	64	56	62	56 - 1/2
54	54	64	70	62	68	64 - 1/2
60	60	70	76	68	74	70 - 1/2

COMPOSITE FLAP GATES

SERIES: A - 652

SERIES: A - 652



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

Back flow prevention applications in low head conditions. Composite material enable high corrosion resistance, longevity and minimum maintenance.

SIZE RANGE:

- From 12 inch to 108 inch.
- Higher sizes can be offered on request.

FEATURES:

- Square / Rectangular fabricated flange back frame / body for mounting on face of wall using anchor fasteners and secondary grout between wall and frame/body or to wall thimble.
- Single piece flap cover manufactured as a composite sandwich construction comprising of light weight rigid cellular core with a fully welded steel matrix covered by rigid compressed composite plastic to withstand applicable head.
- Flap cover is internally chemical bonded, sealed and free of exposed reinforcing material.
- Resilient seal to absorb closing shocks.
- Resilient sealing arrangement comprise of resilient seal mounted on outer peripheral edge of frame / body and in contact with mating face of flap cover.
- Double pivot design for all sizes of flap gate.
- Double hinged with hinge sized to prevent formation of oval hole due to higher applying forces.
- Replaceable hinge pins and self lubricating hinge bushings.

OPTIONAL FEATURES:

- Wall thimble mounting.

MATERIAL OF CONSTRUCTION:

Frame / Body	Stainless Steel ASTM A240-304L / 316L OR FRP
Flap Cover	Composite FRP & Steel OR FRP
Hinge Arm	Stainless Steel ASTM A240-304 / 316
Seals	Neoprene ASTM D-2000
Hinge Bushing	UHMWPE ASTM D-4020
Hinge Bolt	Toughened Steel
Fasteners	Stainless Steel 304 / 316

Higher grade of stainless steel can be offered on request.

SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

FLEXIBLE FLAP GATES
SERIES: A - 672

SERIES: A - 672



SPECIFICATION:

These flap gates are made as per Rodney Hunt design.

APPLICATION:

For low cost back flow prevention applications in low head conditions. A stainless steel frame with neoprene material enable light weight construction thereby offering ease in installation and also high corrosion resistance, longevity and minimum maintenance.

SIZE RANGE:

- From 12 inch to 48 inch.
- For wider openings, two or more can be bonded and clamped.

FEATURES:

- Square / Rectangular flange back frame / body for mounting on face of wall using anchor fasteners and secondary grout between wall and frame/body or to wall thimble.
- Opens under very low unseating head.
- Resilient seal to withstand slamming action of pump discharge and wave action.
- Single piece flap cover to withstand applicable head. Door may also have stainless steel stiffening depending upon size.
- Stainless steel frame with flexible reinforced neoprene flap.

OPTIONAL FEATURES:

- Wall thimble mounting.
- Neoprene to neoprene seating for low leakage.

MATERIAL OF CONSTRUCTION:

Frame / Body	Stainless Steel ASTM 240-304L / 316L
Flap Cover	Neoprene / EPDM ASTM D-2000
Stiffeners	Stainless Steel ASTM A240-304L / 316L
Fasteners	Stainless Steel 304 / 316

SHOP TESTING:

Seat clearance check of each flap gate for checking clearance between mating sealing face.

PROJECT PHOTOGRAPHS



96" x 72" Flap Gate, Garney Washington CSO, Nashville, TN



30" Diameter Flap Gates, 59th Ave Pump Station in Wickenburg, AZ



120" x 90" Flap Gates, Alley Creek- East River, Queens, NY



48" x 36" Flap Gate, Metro Biosolids Center, San Diego, CA



60" x 48" HDPE Flap Gates, Kepala Batas, Malaysia



108" Diameter Flap Gates, Sugarland, County of Fort Bend.



72" Diameter Flap Gates, Russell Allison Ambraw Levee, Lawrenceville, IL

Rodney Hunt also brings to you a wide range of products in cast iron, stainless steel, aluminum, HDPE, composite and steel construction so as to meet most of the flow control applications.

For more information about our products or to contact a sales representative, visit the Rodney Hunt website **www.rodneyhunt.com** or call at any of our offices.



CAST IRON SLIDE GATE



STAINLESS STEEL SLIDE GATE



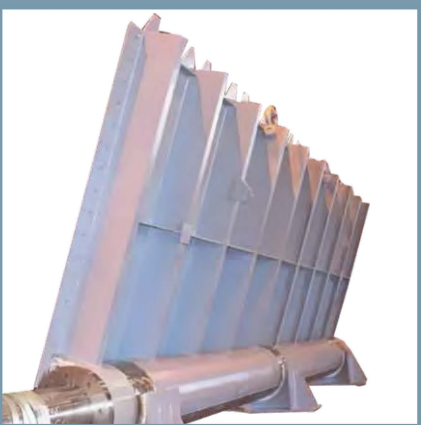
ROLLER GATE



COMPOSITE SLIDE GATE



BUTTERFLY GATE



CREST GATE

818

RODNEY HUNT INC.

6200 Savoy Dr Ste 750
Houston, TX 77036, USA
Phone: (281) 962 6369
Email: houston@rodneyhunt.com

RODNEY HUNT

www.rodneyhunt.com

RODNEY HUNT INC.

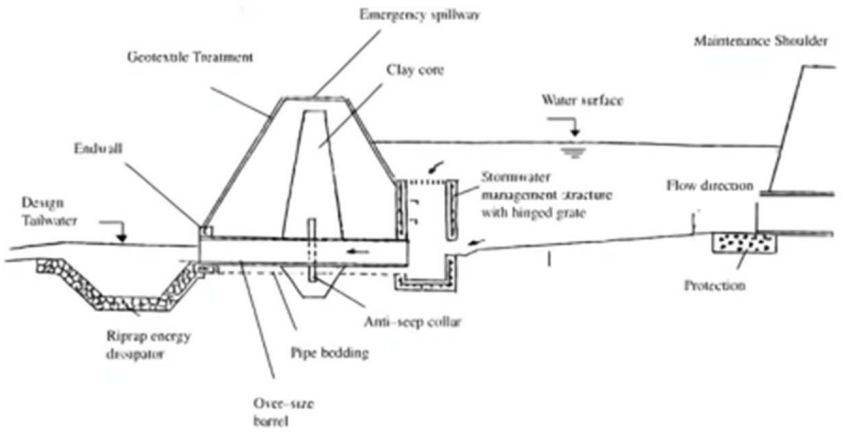
46 Mill Street
Orange, MA 01364, USA
Phone: (978) 633 4362
Email: orange@rodneyhunt.com

US Army Corps of Engineers

Upper Turkey Creek
Detention Pond Drop Inlet Structure
Sta. 57+60 (Approx.)

Comp by: JWW 01/29/2026
Chkd by:

Notes:
1. Inlet structure 1will drain through a 2 48" RCP with inlet at top and low flow near the bottom of the box.
2. There is a flapgate at Turkey Creek. Water is assumed to the top of structure with full hydrostatic head and at-rest soil pressures applied for loading and assumed empty for uplift.



Elevations:

$EL_1 := 897.58\ ft + 2\ in = 897.747\ ft$ — Top of structure + assumed 2" settlement per DDR

$EL_{inv} := 890\ ft$ — Invert elevation per Civil Plans

$EL_S := EL_1$ — Elev. of soil

$H_{inv} := EL_1 - EL_{inv} = 7.747\ ft$ — Top of structure to top of base slab

Dimensions:

$D := 10 \cdot in$ — Wall thickness (all walls)

$C_{c1} := 2.5 \cdot in$ — Wall clear cover

$L_{short} := 3.5 \cdot ft$ — Clear distance Short Span

$L_{long} := 12.0 \cdot ft$ — Clear distance Long Span

$D_f \equiv 12 \cdot in$ — Base slab thickness

$C_{cf} := 2.5 \cdot in$ — Base slab clear cover

$t_{tslab} := 0 \cdot in$ — Top slab thickness (No Top Slab/Grate Only):

$L_{ext} := 12 \cdot in$ — Base slab overhang (length of that extends beyond walls)

$D_{p1} := 1 \cdot ft$ — opening height entering structure

$D_{p2} := 4 \cdot ft$ RCP diameter exiting structure

$Grate := 2.67\ ft \cdot 2.67\ ft = 7.129\ ft^2$ Grate at top of structure

Design Parameters:

$F_y := 60\ ksi$ Steel Yield Strength

$f_c := 4000 \cdot psi$ Concrete Strength

$\gamma_c := 150 \cdot pcf$ Concrete Unit Weight

$\gamma_w := 62.4 \cdot pcf$ Water Unit Weight

Soil Properties:

$\gamma := 115 \cdot pcf$ Blanket Unit Weight (Assumed from DDR)

$\phi := 28 \cdot deg$ Internal Friction Angle (Assumed from DDR)

$surcharge := 0\ psf$ Surcharge - none assumed for controlling flood case, also negligible at controlling locations analyzed (just above or below top of pipe)

$HEAD := 0 \cdot ft$ Height of water in gatewell; conservatively assumed empty

$K_o := 1 - \sin(\phi) = 0.531$ At-rest soil coefficient

Strength Reduction Factors:

$\phi_V := 0.75$ Shear Strength

$\phi_B := 0.90$ Flexural Strength

Load Factors: EM 1110-2-2104, 2023 Chapter 3 (Table 3-2)

$\gamma_N := 1.4$ Unusual Load Factor 10 to 750 year return period

Load Cases:

Unusual Case (100 year return period): reference the Justification Calculations, single load case for serviceability governs design over strength cases.

Flotation Check:

Weight:

Note: The base slab extension is included for submerged soil weight only. It is ignored for uplift since water acts on top and bottom.

$W_{long\_walls1} := 1 \cdot \left(\left(H_{inv} - t_{tslab}\right) \cdot \left(L_{long} + 2 \cdot D\right) - \left(D_{p1} \cdot L_{long}\right)\right) \cdot D \cdot \gamma_c = 11.734$ **kip**

$W_{long\_walls2} := 1 \cdot \left(\left(H_{inv} - t_{tslab}\right) \cdot \left(L_{long} + 2 \cdot D\right) - \left(\frac{D_{p2}^{\,2} \cdot \pi}{4}\right)\right) \cdot D \cdot \gamma_c = 11.663$ **kip**

$W_{short\_walls} := 2 \cdot \left(\left(H_{inv} - t_{tslab}\right) \cdot \left(L_{short}\right)\right) \cdot D \cdot \gamma_c = 6.778$ **kip**

$W_{top\_slab} := \left(\left(L_{short} + 2 \cdot D\right) \cdot \left(L_{long} + 2 \cdot D\right) - Grate\right) \cdot t_{tslab} \cdot \gamma_c = 0$ **kip**

$Base_{SlabArea} := \left(L_{short} + \left(2 \cdot D\right)\right) \cdot \left(L_{long} + \left(2 \cdot D\right)\right) = 70.611$ **ft²**

$W_{base\_slab} := Base_{SlabArea} \cdot D_f \cdot \gamma_c = 10.592$ **kip**

Total Weight of Structure:

$W_S := W_{long\_walls1} + W_{short\_walls} + W_{top\_slab} + W_{base\_slab} + W_{long\_walls2}$ $W_S = 41$ **kip**

Weight of soil (submerged weight):

$soil_{area} := \left(L_{long} + 2 \cdot D + L_{ext}\right) \cdot L_{ext} \cdot 2 = 29.333$ **ft²**

$H_{soil} := H_{inv} - \left(EL_1 - EL_S\right) = 7.747$ **ft**

$W_G := soil_{area} \cdot H_{soil} \cdot \left(\gamma - \gamma_w\right) = 11.953$ **kip**

Water In Structure:

$W_C := \gamma_w \cdot HEAD \cdot \left(\left(L_{short}\right) \cdot \left(L_{long}\right)\right) = 0$ **lbf** $W_C = 0$ **kip**

Structure Factor of Safety for Flotation

$H_u := H_{inv} + D_f$ Depth to bottom of structure

Uplift under Base:

$p_u := H_u \cdot \gamma_w$ $p_u = 545.792$ **psf** Pore water pressure om structure

$Uplift := p_u \cdot Base_{SlabArea}$ $Uplift = 38.539$ **kip**

Stability of structure: **Note:** Reference EM 1110-1-2100, Table 3-4. The required factor of safety for flotation of a structure with unusual load conditions (as defined in Table 3-1) is 1.2.

$Stability := \frac{W_S + W_C + W_G}{Uplift}$ $Stability = 1.37$

$Check_{Stability} := \text{if}(Stability \geq 1.2, \text{“Okay”}, \text{“Not Okay”})$ $Check_{Stability} = \text{“Okay”}$

Strength Analysis

Strength Check #1 - 6" above pipe (full box section)

Wall Loading:

Height from invert to location of analysis: $z := 4.5 \cdot ft$

Height from top of structure to location of analysis: $H_z := H_{inv} - z = 3.247\ ft$

Unusual Case: Soil & Flood Pressures:
Top of structure head on the outside with the structure empty.

$p_u := \gamma_N \cdot (K_o \cdot (\gamma - \gamma_w) \cdot H_z + \gamma_w \cdot H_z + surcharge) = 410\ psf$

For 1 ft strip of the Wall: $b := 12 \cdot in$

$w_u := p_u \cdot b$ $w_u = 410\ plf$

Moment Demand:

Use moment distribution to determine corner moment:

Define lengths to mid-point of intersecting wall for analysis:

$L_1 := L_{long} + D = 12.833\ ft$ $L_2 := L_{short} + D = 4.333\ ft$

Moment at center of intersection of walls:

$M_{u\_center} := \frac{w_u}{12} \cdot \left(L_1^2 - (L_1^2 - L_2^2) \cdot \frac{L_2}{L_1 + L_2} \right) = 4.374\ kip \cdot ft$

Negative moment at face of long wall (note: the short wall has the largest negative moment):

$$M_{u\_neg} := M_{u\_center} - w_u \cdot \left(L_{short} + D\right) \cdot \frac{D}{4} + \frac{w_u \cdot D^2}{8} = 4.039 \ \textit{kip} \cdot \textit{ft}$$

Max Positive Moment for Long Span

$$M_{u\_pos} := \frac{w_u \cdot L_1^{\,2}}{8} - M_{u\_center} = 4.077 \ \textit{kip} \cdot \textit{ft}$$

Max moment:

$$M_u := \max\left(M_{u\_neg}, M_{u\_pos}\right) \qquad M_u = 4.08 \ \textit{kip} \cdot \textit{ft}$$

Moment Capacity:

Axial Load: $P_u := 0 \ \textit{kip}$ $b = 1 \ \textit{ft}$

Diameter of bar: $d_b := \frac{6}{8} \cdot \textit{in}$ #6's @ 12" EW/EF

Area of bar: $A_s := \frac{\pi \cdot d_b^{\,2}}{4} = 0.442 \ \textit{in}^2$

$$d := D - C_{cl} - \frac{d_b}{2} = 7.13 \ \textit{in}$$

$$a := \frac{A_s \cdot F_y + P_u}{0.85 \cdot f_c \cdot b} = 0.65 \ \textit{in}$$ EM 2104 Eq (D-2) rearranged to define "a"

$$\phi M_n := \phi_B \cdot \left(0.85 \cdot f_c \cdot a \cdot b \cdot \left(d - \frac{a}{2}\right) - P_u \cdot \left(d - \frac{D}{2}\right)\right) = 13.519 \ \textit{kip} \cdot \textit{ft}$$ EM 2104 Eq. (D-1)

$$Ratio := \frac{M_u}{\phi M_n} \qquad Ratio = 0.30$$

$$Check_{Moment} := \text{if}\left(\phi M_n > M_u, \text{“OKAY”}, \text{“NO GOOD”}\right) \qquad Check_{Moment} = \text{“OKAY”}$$

Minimum Steel Check:

Solve for minimum required for strength:

Solver Constraints Values

$$A_{sreq} := 0.1 \cdot \textit{in}^2$$

$$M_u = \phi_B \cdot \left(0.85 \cdot f_c \cdot \frac{A_{sreq} \cdot F_y + P_u}{0.85 \cdot f_c \cdot b} \cdot b \cdot \left(d - \frac{\frac{A_{sreq} \cdot F_y + P_u}{0.85 \cdot f_c \cdot b}}{2}\right) - P_u \cdot \left(d - \frac{D}{2}\right)\right)$$ EM 1110-1-2104 Eq. (D-1) with "a" defined from (D-2) substituted

$$A_{sreq} := \text{Find}\left(A_{sreq}\right) = 0.129 \ \textit{in}^2$$

Minimum per EM 2104 3.5.1.4 (2016 Ed.)

$$A_{smin'} := \max\left(\left(\frac{200 \cdot b \cdot d}{F_y} \cdot psi\right), \left(\frac{3 \cdot \sqrt{f_c \cdot psi}}{F_y} \cdot b \cdot d\right)\right) = 0.285\ in^2$$

$$A_{smin} := min\left(A_{smin'}, \left(\frac{4}{3} \cdot A_{sreq}\right)\right) = 0.17\ in^2$$

$Check_{min} := \text{if}\left(A_s > A_{smin}, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $Check_{min} = \text{“OKAY”}$

Minimum Reinforcement Required for Temperature and Shrinkage:

$$A_{s\_TS} := \frac{0.003 \cdot D \cdot b}{2} = 0.18\ in^2$$ EM 1110-2-2104 (Rev 2016), Divided by 2 layers

$Overall_{min} := \max\left(A_{smin}, A_{s\_TS}\right)$ $Overall_{min} = 0.18\ in^2$

$Check_{min} := \text{if}\left(A_s > Overall_{min}, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $Check_{min} = \text{“OKAY”}$

Maximum Steel Check:

$\beta_1 := 0.85$ Normal weight concrete

$$\rho := \frac{A_s}{b \cdot d} = 0.0052$$

$$\rho_b := \frac{0.85 \cdot f_c}{F_y} \cdot \beta_1 \cdot \frac{87000}{(87000 + 60000)} = 0.029$$

$\rho_{max} := 0.25 \cdot \rho_b = 0.0071$ Limit to 25% rho balanced per EM 1110-2-2104 3.5.1.1

$A_{smax} := \rho_{max} \cdot D \cdot b = 0.855\ in^2$ $A_{smax} = 0.86\ in^2$

$Check_{max} := \text{if}\left(A_{smax} > A_s, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $Check_{max} = \text{“OKAY”}$

Shear Check:

Shear Demand (at a distance "d" from the wall):

$$V_u := w_u \cdot \left(\frac{L_{long}}{2} - d\right)$$ $V_u = 2.22\ kip$

Shear Capacity:

$\phi V_n := \phi_V \cdot \left(2 \cdot \sqrt{f_c \cdot psi}\right) \cdot b \cdot d$ $\phi V_n = 8.111\ kip$

$Ratio := \frac{V_u}{\phi V_n} = 0.274$

$Check_{Shear} := \text{if}\left(\phi V_n > V_u, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $Check_{Shear} = \text{“OKAY”}$

Check Shear Capacity Using ACI 318-14 Sect. 22.5.6.1:

$A_g := b \cdot d = 85.5\ in^2$ $\lambda := 1.0$

$\phi V_n := \phi_V \cdot 2 \cdot \left(1 + \left(\frac{P_u}{2000 \cdot A_g \cdot psi}\right)\right) \cdot \lambda \cdot \sqrt{f_c \cdot psi} \cdot b \cdot d = 8.111\ kip$ Includes axial load

$Check_{Shear} := \text{if}\left(\phi V_n > V_u, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $Check_{Shear} = \text{“OKAY”}$

Strength Check #2 - Short wall below top of pipe

Wall Loading:

Location of analysis assumes a 45 degree line of action to reach the height of pure horizontal bending (above the zone of two-way bending/support of the base slab)

Height from invert to location of analysis: $z := 0.5\ L_{short}$

Height from top of structure to location of analysis: $H_z := H_{inv} - z = 5.997\ ft$

Load on 1 ft strip: $w_u := \gamma_N \cdot \left(K_o \cdot \left(\gamma - \gamma_w\right) \cdot H_z + \gamma_w \cdot H_z + surcharge\right) \cdot b$ $w_u = 758.147\ plf$

Moment Demand:

$M_{u\_below\_pipe} := \frac{w_u \cdot L_{long}^{\ 2}}{12} = 9.098\ kip \cdot ft$

Moment Capacity:

Diameter of bar: $d_b := \frac{5}{8} \cdot in$

Area of bar: $A_s := \frac{\pi \cdot d_b^{\ 2}}{4} = 0.307\ in^2$

$d := D - C_{cl} - \frac{d_b}{2} = 7.19\ in$

$$a := \frac{A_s \cdot F_y}{0.85 \cdot f_c \cdot b} = 0.45 \ in$$

$$\phi M_n := \phi_B \cdot \left(A_s \cdot F_y \cdot \left(d - \frac{a}{2} \right) \right) = 9.611 \ kip \cdot ft$$

$$Ratio := \frac{M_{u\_below\_pipe}}{\phi M_n}$$

$Ratio = 0.95$

$Check_{Moment} := \text{if}\left(\phi M_n > M_{u\_below\_pipe}, \text{“OKAY”}, \text{“NO GOOD”}\right)$

$Check_{Moment} = \text{“OKAY”}$

Minimum Steel Check:

Solve Constraints/values

$$A_{sreq} := 0.1 \cdot in^2$$

$$M_{u\_below\_pipe} = \phi_B \cdot A_{sreq} \cdot F_y \cdot \left(d - \left(\frac{A_{sreq} \cdot F_y}{2 \cdot .85 \cdot f_c \cdot b} \right) \right)$$

$$A_{sreq} := \text{Find}\left(A_{sreq}\right) = 0.29 \ in^2$$

$$A_{smin'} := \max\left(\left(\frac{200 \cdot b \cdot d}{F_y} \cdot psi \right), \left(\frac{3 \cdot \sqrt{f_c \cdot psi}}{F_y} \cdot b \cdot d \right) \right) = 0.288 \ in^2$$

$$A_{smin} := min\left(A_{smin'}, \left(\frac{4}{3} \cdot A_{sreq} \right) \right) = 0.288 \ in^2$$

$Check_{min} := \text{if}\left(A_s > A_{smin}, \text{“OKAY”}, \text{“NO GOOD”}\right)$

$Check_{min} = \text{“OKAY”}$

Shear Check ("d" from wall):

$$V_u := w_u \cdot \left(\frac{L_{short}}{2} - d \right)$$

$V_u = 0.873 \ kip$

Nominal Shear Capacity:

$$\phi V_n := \phi_V \cdot \left(2 \cdot \sqrt{f_c \cdot psi} \right) \cdot b \cdot d$$

$\phi V_n = 8.18 \ kip$

$$Ratio := \frac{V_u}{\phi V_n} = 0.107$$

$Check_{Shear} := \text{if}\left(\phi V_n > V_u, \text{“OKAY”}, \text{“NO GOOD”}\right)$

$Check_{Shear} = \text{“OKAY”}$

Strength Check #3 - Base Slab

Factored pressure below base:

$$p_{ub} := \gamma_N \cdot \left(\frac{W_S}{Base_{SlabArea}} - D_f \cdot \gamma_c \right) = 598 \ psf$$

Factored load on1 ft strip:

$w_u := p_{ub} \cdot b$ $\quad w_u = 598 \ plf$

Moment Demand (Top Steel):

$$M_u := \frac{w_u \cdot L_{short}^{\ 2}}{8} = 0.916 \ kip \cdot ft$$ Check one-way conservatively

Diameter of bar: $d_b := \frac{5}{8} \cdot in$

Area of bar: $A_s := \frac{\pi \cdot d_b^{\ 2}}{4} = 0.307 \ in^2$

$$d := D_f - C_{cf} - \frac{d_b}{2} = 9.19 \ in$$

$$a := \frac{A_s \cdot F_y}{0.85 \cdot f_c \cdot b} = 0.451 \ in \qquad b = 1 \ ft$$

$$\phi M_n := \phi_B \cdot A_s \cdot F_y \cdot \left(d - \frac{a}{2} \right) = 12.373 \ kip \cdot ft$$ $\quad \phi M_n = 12.373 \ kip \cdot ft$

$$Ratio := \frac{M_u}{\phi M_n}$$ $\quad Ratio = 0.074$

$Check_{Moment} := \text{if}\left(\phi M_n > M_u, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $\quad Check_{Moment} = \text{“OKAY”}$

Minimum Steel Check:

Solve Constraints Values

$A_{sreq} := 1 \cdot in^2$

$M_u = \phi_B \cdot A_{sreq} \cdot F_y \cdot \left(d - \left(\frac{A_{sreq} \cdot F_y}{2 \cdot .85 \cdot f_c \cdot b} \right) \right)$

$A_{sreq} := \text{Find}\left(A_{sreq}\right) = 0.022 \ in^2$

$$A_{smin'} := \max\left(\left(\frac{200 \cdot b \cdot d}{F_y} \cdot psi \right), \left(\frac{3 \cdot \sqrt{f_c \cdot psi}}{F_y} \cdot b \cdot d \right) \right) = 0.368 \ in^2$$

$$A_{smin} := min\left(A_{smin'}, \left(\frac{4}{3} \cdot A_{sreq} \right) \right) = 0.03 \ in^2$$

$Check_{min} := \text{if}\left(A_s > A_{smin}, \text{“OKAY”}, \text{“NO GOOD”}\right)$ $\quad Check_{min} = \text{“OKAY”}$

Shear Check ("d" away from wall):

$$V_u := w_u \cdot \left(\frac{L_{short}}{2} - d \right)$$

$V_u = 0.589\ kip$

Nominal Shear Capacity:

$$\phi V_n := \phi_V \cdot \left(2 \cdot \sqrt{f_c \cdot psi} \right) \cdot b \cdot d$$

$\phi V_n = 10.459\ kip$

$$Ratio := \frac{V_u}{\phi V_n} = 0.056$$

$$Check_{Shear} := \text{if}\left(\phi V_n > V_u, \text{“OKAY”}, \text{“NO GOOD”} \right)$$

$Check_{Shear} = \text{“OKAY”}$

US Army Corps of Engineers®
Kansas City District

UPPER TURKEY CREEK

Johnson Drive Bridge Report

P2 #505229

Design Documentation Report

September 2025

PREPARED BY:

HNTB

Table of Contents

1. Executive Summary **1**

2. Background Information, Procedures, and Design Criteria **2**

2.1 Background Information 2

2.1.1 Project Location 2

2.1.2 Project Purpose 3

2.1.3 Drawing Backgrounds 3

2.2 Existing Features 4

2.2.1 Existing Johnson Drive Bridge 4

2.2.2 Existing Johnson Drive Roadway 4

2.2.3 Existing Utilities 7

2.2.4 Existing Right-of-Way (ROW) 8

2.3 Design Criteria 8

3. Project Approach **9**

3.1 Approach to Alternative Development 9

3.1.1 Water Surface Elevation 9

3.1.2 Bridge Profile Elevation 10

3.2 Evaluation Matrix 10

3.2.1 Evaluation Categories 10

3.2.2 Evaluation Category Weighting 12

3.2.3 Evaluation Matrix Scoring 13

3.3 Alternatives Considered 13

3.3.1 Project Approach 13

3.3.2 Bridge Type 15

3.4 Recommended Alternative Selection 17

4. Design Approach **18**

4.1 USACE Hydraulic Analysis 18

4.2 Typical Sections 18

4.3 Vertical Design 19
4.3.1 Profile Grade Criteria 19
4.3.2 Existing Profile Grade Elevations 19
4.3.3 Profile Grade Constraints 19
4.3.4 Optimal Profile Grade 20
4.3.5 USACE-Provided Conceptual Profile 20
4.3.6 Allowable Profile Grade 21
5. Recommended Civil Design 22
5.1 Bridge Profile Over Turkey Creek 22
5.2 Potential Impacts East of Turkey Creek 22
5.3 Potential Impacts West of Turkey Creek 23
5.4 Potential Utility Impacts 23
5.4.1 Evergy 19
5.4.2 Johnson County Wastewater 19
5.4.3 Kansas Gas / WaterOne 19
5.4.4 AT&T / Everfast / Google / Spectrum / Verizon 20
5.5 Quantities 24

List of Appendices

- Appendix A – Design Criteria
- Appendix B – USACE Provided Drawings
- Appendix C – Evaluation Matrix
- Appendix D – Hydraulic Analysis
- Appendix E – Proposed Typical Sections
- Appendix F – Concept Exhibits
- Appendix G – Quantities

1. Executive Summary

In March 2025, the United States Army Corps of Engineers (USACE) appointed HNTB to perform an analysis of the Johnson Drive bridge over Turkey Creek to develop a conceptual design that may reduce flood risk to downtown Merriam. In the scope of work, HNTB was given the following tasks:

- Analyze Johnson Drive bridge concepts developed by USACE
- Provide insight on up to two (2) additional viable alternatives
- Estimate impacts to surrounding infrastructure
- Develop a decision matrix
- Provide a recommended alternative
- Estimate quantities for the recommended alternative
- Facilitate discussions with government, city, and HNTB to drive alternative selection

Existing conditions at the Johnson Drive crossing indicate the bridge overtops in major storm events. The existing bridge has a low chord elevation of 914.89' at the east abutment (low side) while the 1% annual exceedance probability (i.e. 100-yr event) water surface elevation is 918.62'. Because several interrelated variables impact hydraulic performance, the project was analyzed holistically. A decision matrix was developed focused on two primary perspectives: increasing the effective cross-sectional area and bridge types. The resulting recommendation is to utilize a concrete haunched slab bridge with moderately raised Johnson Drive profile and a channel widening to the west of approximately 30'.

The recommended alternative provides the following benefits compared to the other options covered later in this report:

- Minimizes impacts to surrounding properties and infrastructure (estimated 13 partially affected tracts and 3 possible total acquisitions, refer to Figure G.4).
- Does not require realignment of Johnson Drive or Merriam Drive.
- Profile changes along Johnson Drive can be tied into along Merriam Drive with minimal impacts.
- Provides opportunity to reconstruct and maintain access of the Turkey Creek Streamway Trail.
- Ability to provide a seamless connection to adjacent floodwalls by incorporating structural fascia walls, thereby providing some freeboard to the 100-yr event water surface elevation.

However, as with any alternative, there are some drawbacks:

- Channel widening to the west assumes closure of Campbell Lane. Access to a few residences, along with Merriam Christian Church, will be negatively impacted.
- Profile raise of Johnson Drive may require abandonment of the parking lot at the northeast corner of the intersection with Merriam Drive. Likewise, the BP gas station on the southeast corner may lose access to the westmost driveway.

These considerations are discussed in further detail throughout the report.

2. Background Information, Procedures, and Design Criteria

2.1 Background Information

2.1.1 Project Location

The project location is in Merriam, Kansas – refer to *Figure 2.1* below. The project extents are limited to Johnson Drive bridge over Turkey Creek and the length of roadway needed to tie the bridge back to existing features.

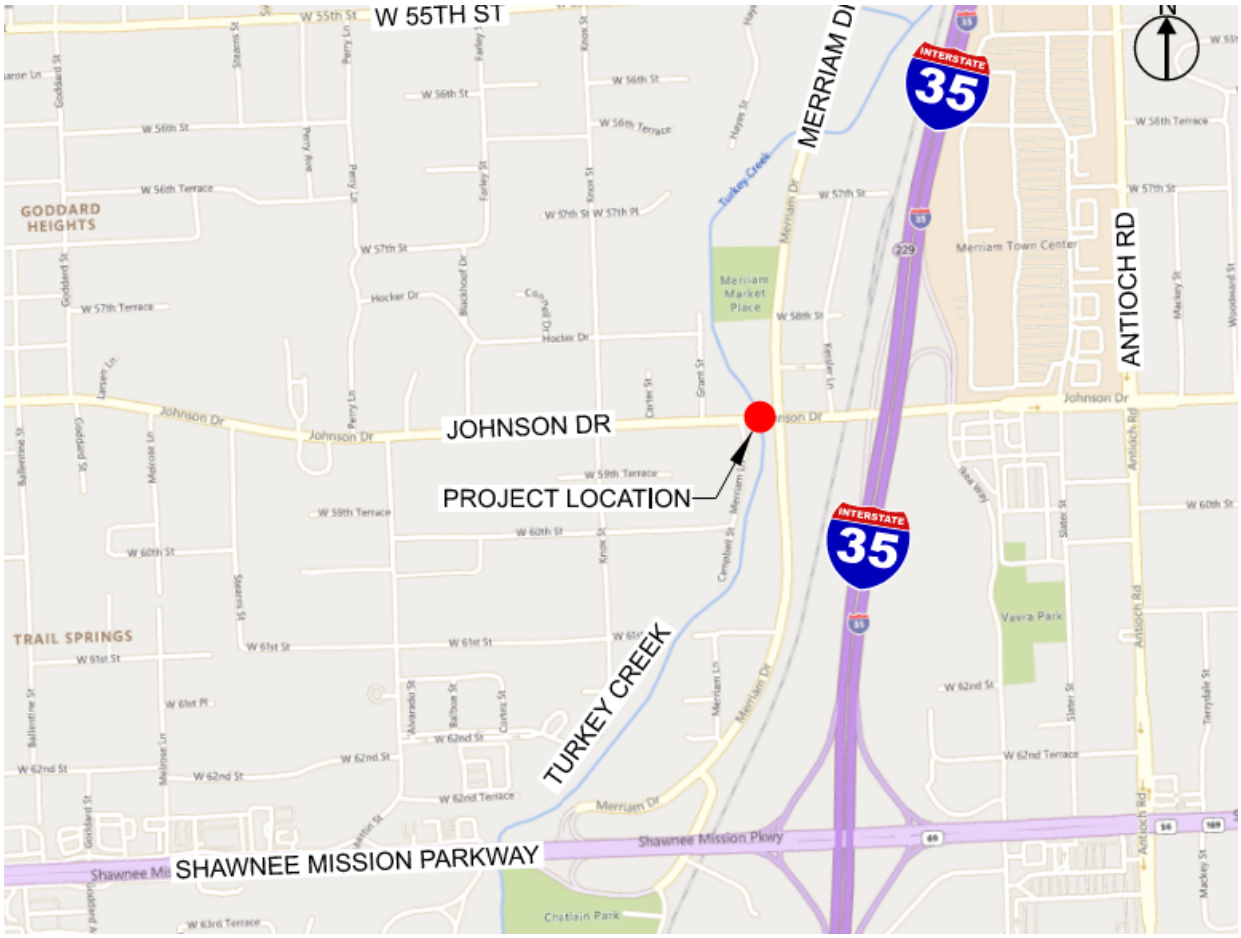


Figure 2.1 – Project Location Map
(Source: Bing Imagery)

2.1.2 Project Purpose

The Upper Turkey Creek study focuses on flood risk management at the Johnson Drive Bridge over Turkey Creek. USACE completed the Upper Turkey Creek Basin Flood Risk Management Study in 2015 (herein referred to as "2015 Feasibility Study") and identified problems such as:

- Severe flash flooding with low response time
- Loss of life due to flooding
- Property damage to commercial/industrial properties
- Damage to infrastructure
- $3.5M in estimated annual damages

This report aims to develop an approach to reduce the risk of severe flash flooding by analyzing various bridge and channel options for Johnson Drive over Turkey Creek. Proposed alternatives do not rely on emergency response crews to provide closures or evacuations due to the rapid rise in water surface elevation (WSEL).

2.1.3 Drawing Backgrounds

Figures and exhibits displayed in this report use the USACE-provided 2023 topographic and boundary survey information. Johnson County Automated Information Mapping System (AIMS) planimetrics and property information were used to supplement the 2023 topographic survey.

Existing utility drawings were compiled using as-built drawings from utility owners. The 2023 USACE Subsurface Utility Exploration (SUE) and AIMS were used to verify utility locations but are not displayed in any background drawings.

The existing terrain was modeled using AIMS contours. While 3D topographic survey was provided by USACE, no terrain model was provided.

2.2 Existing Features

2.2.1 Existing Johnson Drive Bridge

The existing bridge was constructed in 1981, and the superstructure was reconstructed in 2014. The bridge consists of 49.4'-50.4'-49.5' composite steel rolled beam spans supported by pile bent abutments and concrete column bent piers founded on drilled shafts, as seen in *Figure 2.2* to the right. The total superstructure depth is approximately 3.0' from top-of-deck to bottom-of-steel-beam.



Figure 2.2 - Existing Johnson Dr Bridge

The bridge section totals 75'-4" in width. The section includes five (5) 11' lanes and 5' sidewalks on both sides of the roadway.

The low steel elevation is approximately 911.7', while the lowest deck elevation is approximately 914.5'. The 1% annual exceedance probability (AEP) elevation is 918.47' at the upstream (south) face of the bridge and 917.95 at the downstream face. Due to the longitudinal grade of the roadway, only the east end of the bridge overtops during the 1% annual exceedance probability (AEP) high water event.

The steel beams of the existing bridge are susceptible to being damaged by large floating debris during flooding. A replacement bridge should have adequate freeboard above flood waters or be robust enough to avoid damage if impacted by debris.

2.2.2 Existing Johnson Drive Roadway

Johnson Drive is a minor arterial roadway consisting of a 5-lane section with 11-foot lanes and curb and gutter. The roadway has a posted speed limit of 35mph, and design speeds will be assumed to match the posted speed limit. Johnson Drive serves as a primary link between I-35 to the east and the cities of Merriam and Shawnee to the west.

2.2.2.1 Horizontal Geometry

Section 1.2 of AASHTO's *A Policy on Geometric Design for Highways and Streets,* 7th Edition (herein referred to as GDHS-7) explicitly states: "If some aspects of the geometric design for a road or street do not fully comply with the geometric design criteria…but the road or street is performing satisfactorily, there is no need to change those aspects of the existing geometric design for projects in which the basis roadway type will remain the same". There is currently no available data suggesting the existing roadway performs inadequately. Further, the alignment, typical section, and function of Johnson Drive are remaining unchanged. Therefore, geometric deficiencies are solely discussed in this section for

acknowledgement and to suggest a possible resolution for future consideration if determined feasible within the parameters of the project.

At the west end of the bridge, the existing lane shift taper length is 75'. Refer to *Figure 2.3* below for a figure showing this taper. AASHTO GDHS-7 Equation 3-39 (shown to the right) derives required lengths of tapers for roadways with design speeds less than 45-mph. AASHTO GDHS-7 Section 3.4.4.1 also specifies that a lane addition taper can be half the length determined by AASHTO GDHS-7 Eq. 3-39. For a design speed of 35-mph and width transition of 11', a taper length of 115' is recommended.

$$L = W * S^2/60$$ *(AASHTO GDHS-7 Eq. 3-39)*

Where:

- *L = Length of Taper (ft)*
- *W = Width of Taper (ft)*
- *S = Design Speed (mph)*

The two most reasonable options for correcting the issue are to widen the eastbound lanes to the west or restripe the taper with a shorter left turn lane at Merriam Drive. The widening option, in conjunction with the revised profile along Johnson Drive, may have adverse property impacts on the southwest corner of the Johnson Drive and Campbell Lane intersection. Restriping the taper to shorten the left turn lane should only be done after performing a traffic analysis to verify the Johnson Drive and Merriam Drive intersection would continue to operate sufficiently. Both options should be vetted in more detail if this project goes to full design.

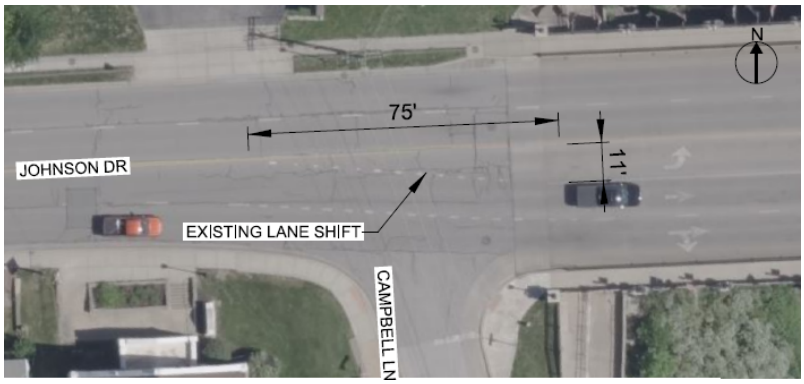


Figure 2.3 - Johnson Dr Taper at West Edge of Bridge

At the east side of the Johnson Drive and Merriam Drive intersection, another lane shift taper is used to develop a left turn lane, as shown below in *Figure 2.4*. This taper is an adequate length for the 35-mph roadway. It should be noted that BNSF Railway ROW begins approximately 60' beyond the existing end of taper, and the BNSF ROW should be avoided.

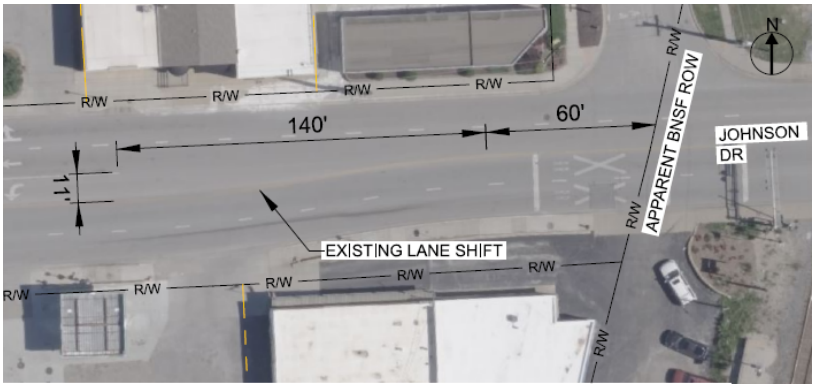


Figure 2.4 - Johnson Dr Taper at East Extent of Study

2.2.2.2 Merriam Drive Intersection

The intersection of Johnson Drive and Merriam Drive has several inadequacies or inefficiencies. The curb return radii in the western half of the intersection promote traffic turning into improper lanes – vehicles will turn into the outermost lane of the receiving roadway, as seen below in *Figure 2.5*. Merriam Drive has inefficiencies with the existing lane configuration, as shown below in *Figure 2.6*. Southbound Merriam Drive has a right turn lane with a storage length of 35', enough for approximately two (2) vehicles and not enough length to allow vehicles to enter the turn lane if there is a queue for the through movement. South of the Johnson Drive intersection, the westmost lane is dropped. This lane could be removed without any negative impacts to traffic flow. Removing these lanes could provide more room for flood control measures while providing the opportunity to correct the intersection curb return radii.

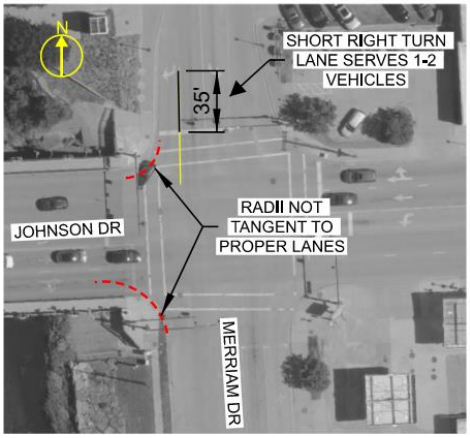


Figure 2.5 - Intersection Radii

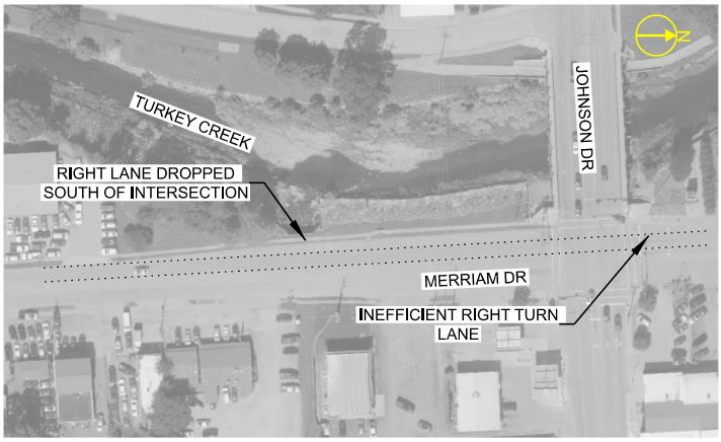


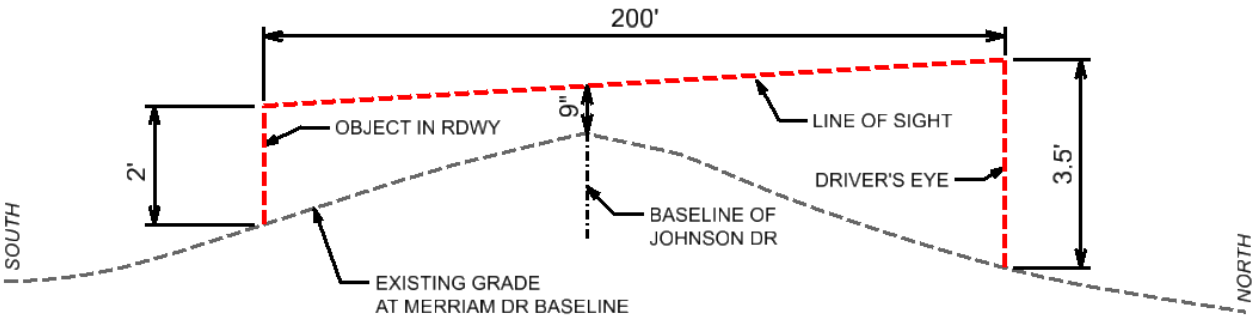
Figure 2.6 - Merriam Dr Potential Lane Reduction

2.2.2.3 Merriam Drive Roadway

The Merriam Drive roadway is also a minor arterial roadway, per the Mid-America Regional Council (MARC) Functional Classification Map. The existing posted speed limit is 30-mph; therefore, the design speed is set at 30-mph in this report. While no realignment is required, Merriam Drive can undergo a lane reduction to provide additional space for flood protection measures. In fact, the City of Merriam has

worked to develop draft plans that show Merriam Drive reduced to a three-lane section north of Johnson Drive.

Existing Merriam Drive provides adequate stopping sight distance (SSD) through the Johnson Drive intersection. SSD is discussed in Section 3.2.6 of AASHTO GDHS-7 – where an SSD of 200' is required for a 30-mph roadway. The driver's eye height should be assumed to be 3.5', and the object height should be assumed to be 2.0'. Using this analysis method, *Figure 2.7*, shown below, illustrates the existing SSD. Note that any grade raises of 9" or more for Johnson Drive are likely to impede SSD. Combinations of vertical curvature and flattening of the cross slope of Johnson Drive should be utilized to mitigate SSD impacts from any Johnson Drive grade raises.



MERRIAM DR PROFILE
(Looking West)

Figure 2.7 - Merriam Dr SSD

2.2.3 Existing Utilities

Identification of existing utilities was coordinated with the concurrent USACE Task 1: Utility Analysis project. Utilities were initially determined via OneCall, and all known utilities were contacted for as-built plans. The following utilities have been identified within study limits:

Table 2.1 - Identified Utilities

Identified Utilities				
Owner	**Facility Type**		**Owner**	**Facility Type**
AT&T	Fiber		City of Merriam	Storm Sewer
Everfast	Fiber		Evergy	Electric
Google Fiber	Fiber		Johnson County Wastewater	Wastewater
Spectrum	Fiber		Kansas Gas	Gas
Verizon	Fiber		WaterOne	Water

Notably, the existing bridge over Turkey Creek has nine (9) conduits running between girders, as seen in *Figure 2.2*. The proposed bridge section will consider providing a location for conduit along the bridge.

2.2.4 Existing Right-Of-Way (ROW)

Figure 2.8, shown below, depicts the existing ROW through Downtown Merriam. It should be noted that ROW varies constantly throughout the Johnson Drive corridor, and ROW on the east/west sides of Turkey Creek is inconsistent. As seen in the figure below, the eastern limits of the project are bound by the BNSF Railway ROW. Caution should be taken to avoid impacts to the BNSF Railway ROW.

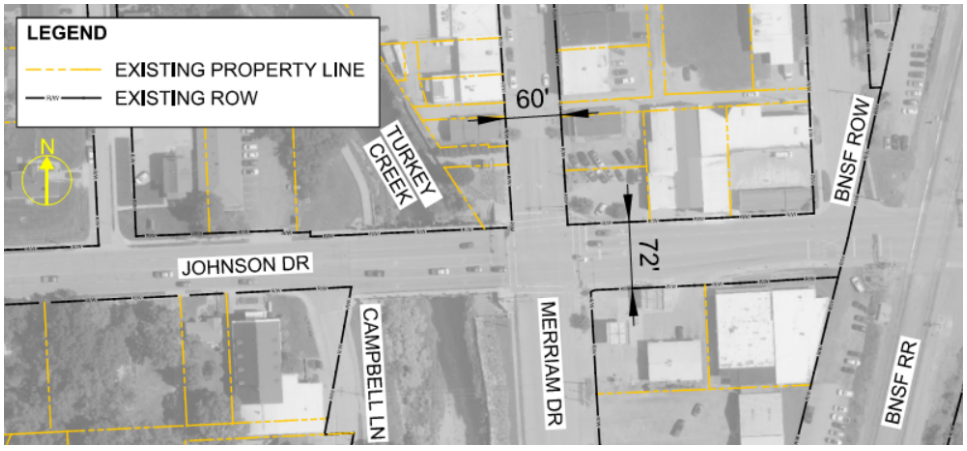


Figure 2.8 - Existing ROW

2.3 Design Criteria

Design criteria were developed in accordance with the latest guidance/policies published by AASHTO and KDOT. For a full listing of roadway design criteria and sources, refer to *Table A.1* in Appendix A.

3. Project Approach

3.1 Approach to Alternative Development

The driving factors for the overall Upper Turkey Creek Project are outlined in Section 2.1.2 of this report. Any Johnson Drive bridge project alternative should aim to address all factors driving the overall project.

In the 2015 Feasibility Study, channel widening and levees/floodwalls were discussed as Upper Turkey Creek Flood Risk Management approaches. These approaches were incorporated into Johnson Drive bridge alternative development alongside site-specific considerations.

The study team identified that the interfacing of bridge design and the hydraulic principles driving the project offers a methodology to analyze project impacts holistically. Ultimately, the goal of the project is to reduce risk for downtown Merriam's safety, properties, and infrastructure. At a high level, there are two (2) related variables that can be changed on this project to achieve the project's goals: WSEL and bridge profile elevation. Alternatives should utilize a combination of these two (2) factors to address the flooding problem.

3.1.1 Water Surface Elevation (WSEL)

The 1% AEP WSEL of Turkey Creek is the primary driving force behind the need for the Johnson Drive bridge project. In the current state, a 1% AEP event would overtop Johnson Drive and Merriam Drive and flood downtown Merriam. Any improvements will need to provide a continuous barrier to prevent future flooding.

At a simplified level, the WSEL is a function of the volumetric flow rate (constant in any alternative), velocity of flow, and cross-sectional area of the flow. Alternatives can alter the velocity of the flow through the reduction of obstructions to the waterway. While channel roughness can also alter the velocity of the flow, channel roughness was not analyzed as part of this project. Alternatives can alter the cross-sectional area of the flow through channel widening or an increase in channel depth (measured from the bottom of the channel to the low chord of the bridge). In summary, the design can widen the channel or raise the bridge to improve the cross-sectional area for Turkey Creek.

It should be noted that improvements to the hydraulic efficiency of Turkey Creek upstream or downstream of the Johnson Drive bridge may also have positive impacts on the WSEL at the Johnson Drive bridge, but analysis to the upstream and downstream reaches of Turkey Creek are outside of the scope of this study.

A lower WSEL will lead to reduced impacts to surrounding infrastructure and properties.

3.1.2 Bridge Profile Elevation

The bridge profile elevation is directly affected by bridge type through structure depth and freeboard requirements. In general, lower bridge profile elevations will lead to reduced impacts to the surrounding infrastructure and properties.

3.2 Evaluation Matrix

Evaluation matrices were created to quantitatively assess project approaches. This section of the report will discuss the development of the evaluation matrices with thorough explanations of the evaluation categories, weighting of the categories, scoring of the matrix, and alternatives considered by the matrix.

3.2.1 Evaluation Categories

Evaluation categories were developed to rank project approaches against the goals of the project. Meetings were conducted with USACE and the City of Merriam to develop evaluation categories that reflect the interests of all project stakeholders, and the following categories were determined.

3.2.1.1 Costs

Description

Costs were comparatively analyzed between alternatives for evaluation purposes. To facilitate comparisons, costs were discretized to fully understand the impacts of each alternative. For example, different bridge types will have different construction costs; however, each may also have different impacts on ROW or impacts to adjacent infrastructure. As these variables are separate yet positively correlated, the general cost factor was divided into subcategories.

Scoring

The optimal project would minimize total costs and impacts across all subcategories.

Subcategories

Subcategories for the cost grouping are explained below:

- *Bridge* – an estimate of the structure's costs
- *Impacts to Adjacent Infrastructure* – relative estimate of roadway, sidewalk, trail, stormwater facilities, signals, lighting, etc. that are impacted
- *Utility Relocations* – relative estimate of how many utilities will need relocated and ability to accommodate these utility relocations
- *ROW/Easements* – estimate of total property impacts incurred
- *Miscellaneous/Aesthetics* – grouping of uncategorized costs/impacts such as additional aesthetic features, flood control measures, etc.

3.2.1.2 Capacity for Levee Over Floodwall

Description

This category offers insight into how an alternative may influence the ability to utilize a levee rather than a floodwall. Levees are generally preferable to floodwalls due to maintenance considerations, appearance, constructability, and cost. However, levees require much more room to construct. Therefore, an alternative that requires the acquisition of ROW (e.g. such as the closure of a road due to widening the channel to the west) may provide a serendipitous benefit such as more available room in which to construct a levee rather than a floodwall.

Scoring

Alternatives that provide more ROW or more opportunity to construct levees instead of floodwalls rate higher.

3.2.1.3 Maintenance of Traffic (MOT)/Construction Length

Description

Johnson Drive is a minor arterial roadway in Johnson County and is the primary east/west access to downtown Merriam. Any closures of Johnson Drive will result in delayed travel times to/from I-35 for populations west of Turkey Creek. If the Johnson Drive bridge is closed for construction, traffic will have to detour south to Shawnee Mission Parkway to access I-35; this is approximately a 1.1-mile detour that would have to pass through two (2) additional traffic signals. The MOT category is a measure of the ability to manage traffic and travel delays during construction. One factor affecting MOT is the ability to maintain traffic on the existing structure during construction of the new structure. Overall impacts also factor into the MOT/construction length where greater impacts will lengthen the timeline, and larger profile grade raises will reduce the ability to accommodate traffic during construction.

Scoring

An optimal project would reduce the total traffic impacts and length of any construction activities.

3.2.1.4 Impacts to Natural Environment

Description

In any project, limiting impacts to the natural environment is ideal. For this specific study, there is extra importance due to the proximity to a waterway (Turkey Creek) and the stakeholders' relationships with Turkey Creek. The Turkey Creek Streamway Trail, a very popular trail in the region, follows parallel to Turkey Creek, and passes under the Johnson Drive Bridge. A major draw of the Turkey Creek Streamway Trail is the natural feeling provided by Turkey Creek's streambed. This category offers insight into how much an alternative will impact the existing natural environment.

Scoring

An optimal project would reduce the impacts to the stream and preserve natural areas.

3.2.2 Evaluation Category Weighting

To balance the evaluation matrix in the best interests of the project, each category was weighted within the matrices. Weights range from 1 to 5, with a higher weight indicating increased importance for the project. Discussion on how the weight is applied is provided in the next section of this report. Weights are as shown below in *Table 3.1*.

Table 3.1 – Weighting by Category

Weighting by Category			
Costs		**Other Categories**	
Category	**Weight**	**Category**	**Weight**
Bridge	2	Capacity for Levee Over Floodwall	5
Impacts to Adjacent Infrastructure	2	MOT/Construction Length	3
Utility Relocations	2	Impacts to Natural Environment	1
ROW/Easements	2	-	-
Miscellaneous/Aesthetics	1	-	-
Total	**9**	**Total**	**9**

It should be noted that the cost categories are equivalent in total weight (9) to all other categories (9). Intentionally, costs were weighted as a very high factor for this study due to overall Upper Turkey Creek project costs. Costs are also an indicator of overall impacts to existing features – costs are positively correlated with expected impacts.

While costs and impacts gain the majority of the weight in this evaluation matrix, weight needed to be provided to other considerations such as flood control approach, intermediate-term impacts to downtown Merriam traffic, and impacts to nature.

3.2.3 Evaluation Matrix Scoring

Each alternative was analyzed in each evaluation category. A score of 0-5 was assigned and placed to the right of the alternative in the matrix. A score of 0 indicates no direct influence on the evaluation category, a score of 1 indicates poor performance in the evaluation category, and a score of 5 indicates exceptional performance in the evaluation category. Overall, a high total score indicates more potential to address the issues at a project-specific level. While scores are based in engineering judgment, some subjectivity is built into the matrix. All scores represent the engineers' best judgment in comparison with other alternatives.

After scores have been assigned, a total score is developed using *Equation 3-1*, shown below:

$$\sum_{i=1}^{X} C_i * W_i = T \qquad (Eq.\ 3\text{-}1)$$

Where:

- C_i = *Category Score in Column i*
- W_i = *Weight of Column i*
- T = *Total Score*
- X = *Final Column in Evaluation Matrix*

A sample calculation for an individual alternative can be found in Appendix C.

3.3 Alternatives Considered

Two (2) main methodologies to address the problems are identified within this report. These methodologies are Project Approach and Bridge Type. Further discussion on the methodologies is included in the following sections of this report. Three (3) alternatives were considered for each methodology, for a total of six (6) alternatives considered. These methodologies are not exclusive – alternatives from each methodology can be combined. The purpose of this alternative development method is to identify the most effective approach to address all project-specific issues.

3.3.1 Project Approach

The Project Approach alternatives are intended to identify how different approaches can influence the engineering principles discussed in Section 3.2 of this report. Essentially, the evaluation matrix examined the impacts incurred by reducing the WSEL or increasing the roadway profile through three (3) alternatives. Hydraulic analysis for channel alternatives within the Project Approach was completed by USACE.

3.3.1.1 Raise Bridge Deck

Overview

This alternative focuses solely on raising the bridge profile to accommodate the necessary WSEL and does not alter the Turkey Creek channel. The base assumption for comparison is to raise the bridge deck to the optimal profile as discussed in Section 4.3.4.

Pros

- No impacts to Turkey Creek.
- Bridge costs expected to be lower due to no increase in structure length.
- Assumed to offer a reduction in WSEL due to no pressurized flow (this was not modeled).

Cons

- Results in most impacts to infrastructure, properties, and utilities due to grade raise required for the bridge profile.
- Likely to be most costly alternative.

Score: 27

3.3.1.2 Widen Creek Channel to West

Overview

This alternative widens the existing Turkey Creek channel 30' to the west.

Pros

- Offers a reduction of approximately 4" in WSEL. Additionally, the lower WSEL propagates upstream for about a half mile.
- Although the profile elevation change is minor, it is expected to decrease impacts to infrastructure, utilities, and ROW/Easements along Johnson Drive and Merriam Drive.
- May allow for space alongside the west bank of Turkey Creek for levees instead of flood walls to the south.
- Any additional channel improvements downstream would likely improve the returns gained by widening to the west.

Cons

- Longest bridge of any project approach alternative analyzed.
- Campbell Lane would be required to be closed permanently, negatively impacting access to Merriam Christian Church and a few residences to the south.
- Access to Merriam Christian Church would need to be reimagined; likely being reconstructed from Johnson Drive.
- More utility relocations along Campbell Lane would be added to the project.

Score: 38

3.3.1.3 Widen Creek Channel to East

Overview

This alternative widens the existing Turkey Creek channel to the east 10' through the reduction of the westmost turn/merge lanes of Merriam Drive.

Pros

- Offers a very minor reduction in WSEL due to increased cross-sectional area.
- Impacts to infrastructure, utilities, and ROW/Easements may be marginally reduced due to the decreased bridge profile elevation.

Cons

- Bridge costs would be moderately raised due to increased length of the bridge.
- Utility relocations along Merriam Drive would be added to the project.
- Benefits realized don't justify the costs to implement.
- Would eliminate the option to improve the radial returns at the Merriam Drive intersection.
- May make the construction of flood walls more challenging on the east side of Turkey Creek.

Score: 30

3.3.2 Bridge Type

The Bridge Type alternatives were intended to identify how different bridge types may reduce impacts to existing features and potentially provide hydraulic efficiencies that could reduce the WSEL.

A steel rolled beam structure with bents (like the existing bridge structure) was briefly considered but ruled out due to significant limitations. Structure depth, susceptibility to damage from debris, floating concerns, and freeboard requirements compare less favorably to the other alternatives outlined below.

3.3.2.1 Reinforced Concrete Box Culvert

Overview

This alternative was developed by USACE and suggests replacing the existing girder bridge with an RCB culvert. Refer to *Figure B.1* in Appendix B for a graphic representation of the RCB culvert.

Pros

- Low bridge cost.
- Functions similar to a concrete haunched slab bridge.

Cons

- Span and size required eliminates precast RCB options and would necessitate forms, which would be no different than a haunched slab bridge.
- Would have more obstructions (i.e. walls) in the channel than other options.
- Walls typically built on strip (spread) footings, which is more difficult to do in a channel than drilled shafts.

- If constructing a bottom floor, would need to phase out construction more and redirect channel flow around initial box being constructed, which is invasive to the natural channel.
- Kansas Division of Water Resources will require at least one cell, if not all cells, to be buried below natural channel bed to allow for aquatic wildlife passage, negating any positive effects that could be realized with the smooth cell floors.
- More likely to experience debris accumulation at upstream end of RCB than other options.

Score: 14

3.3.2.2 Shallow Superstructure (Haunched Concrete Slab Bridge)

Overview

This alternative was developed to analyze the effects of using a bridge type with a shallow structure depth. By reducing the structure depth, the required bridge profile elevation for a given WSEL is reduced. The primary bridge type considered for this alternative is the haunched concrete slab bridge.

Pros

- Relatively low bridge cost.
- Lower bridge deck elevation will lead to reduced impacts to adjacent infrastructure and utilities.
- Relatively quick construction that regional construction crews are accustomed to.
- Very robust structure; nearly impervious to damage from floating debris.
- Requires less freeboard than other options.

Cons

- Haunched concrete slab bridges require construction falsework that reduces the design waterway during construction.

Score: 16

3.3.2.3 Remove Midspan Bents

Overview

This alternative was developed to analyze the effects on hydraulics by removing midspan bents. However, a hydraulic analysis conducted by USACE determined that removing the bents from the Johnson Drive Bridge had little effect on the WSEL. Bridge types primarily considered for this alternative included truss and arch bridges.

Pros

- High aesthetic value – would add a gateway architecture piece to downtown Merriam.
- Minimized natural impacts – no structural elements located in the stream would minimize the bridge's influences on the stream.

Cons

- High bridge cost due to complexity of structures and steel components.

- Ineffective at lowering grade raise required for Johnson Drive Bridge; would have the greatest impacts to surrounding infrastructure.
- Would be the most expensive overall option.

Score: 8

3.4 Recommended Alternative Selection

The suggested design approach combines the top-scoring project approach (widen creek channel to west) and top-scoring bridge type (haunched concrete slab bridge) to produce a recommended alternative. For more discussion on the recommended alternative, refer to later sections of this report.

4. Design Approach

This section discusses the development of the recommended alternative, starting with the project approach and bridge type identified in the previous section.

4.1 USACE Hydraulic Analysis

To develop the recommended alternative civil site design, the water surface profile for Turkey Creek was needed. USACE completed hydraulic analysis of two options: the existing bridge (shown in *Figure D.1* of Appendix D) and a concrete haunched slab bridge (shown in *Figure D.2* of Appendix D).

Another model (no exhibit included in this report) was ran by USACE that combined the concrete haunched slab bridge with a channel widened 30' to the west of the existing west abutment of the Johnson Drive Bridge. A water surface profile (shown in *Figure D.3* of Appendix D) was provided by USACE to show the assumed water surface profile at the upstream edge of the Johnson Drive bridge.

Table 4.1 – Key Water Surface Elevations

Key Water Surface Elevations		
Location	**Widening WSEL**	**No-Widening WSEL**
West Abutment	918.07	918.47
East Abutment	918.27	918.62

The key WSELs from each model are shown in *Table 4.1* to the left. A widened channel drops the water surface profile by an average of 0.38', effectively reducing the required bridge profile and flood protection elevation by the same amount. It should be noted that in both alternative models ran, the models assumed pressurized flow at the bridge.

4.2 Typical Sections

Typical sections were developed for Johnson Drive, the Johnson Drive Bridge, and Merriam Drive to show a section view of the proposed civil features. Refer to Appendix E for typical section exhibits.

It should be noted that the profile grade line of Johnson Drive in exhibits is drawn at the baseline for the roadway, and an elevation drop will occur from the baseline to the upstream edge of the bridge, as seen in *Figure E.1*. This drop is calculated at a rate of 2% over 35', for a total drop of 0.70'.

4.3 Vertical Design

4.3.1 Profile Grade Criteria

Criteria for the vertical design is shown in *Table A.1* in Appendix A. Minimum sag vertical curve K-values are a controlling factor for the tie-ins of Johnson Drive and Merriam Drive east of the Turkey Creek bridge.

4.3.2 Existing Profile Grade Elevations

The existing profile grade elevations (as found in the topographic survey from 2023 provided by USACE) at the east and west abutments are shown to the right in *Table 4.2.* Elevations were checked against AIMS surface contours and verified to be accurate.

Table 4.2 – Existing Elevations

Existing Profile Grade Elevations	
Location	**Elevation**
West Abutment	919.78
East Abutment	914.89

4.3.3 Profile Grade Constraints

To determine the extents of the horizontal geometric layout necessary to accommodate a raised bridge over Turkey Creek, the bridge profile must first be created. In *Figure 4.1*, shown below, the factors influencing the profile grade of the roadway over Turkey Creek are displayed.

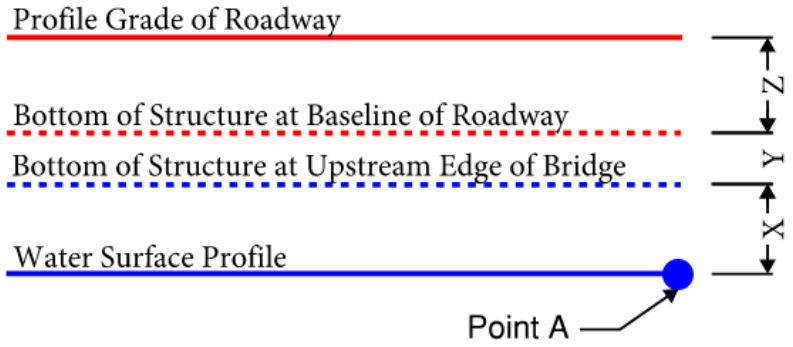


Figure 4.1 – Factors Influencing Bridge Profile Grade

The bridge profile has a critical point for determining elevation, indicated in *Figure 4.1* above by "Point A". Point A is where the elevation difference between the bridge low chord and the water surface is at its lowest. Assuming that the widened channel will be the base condition moving forward, this critical point occurs at the east abutment of the Johnson Drive bridge with an elevation of 918.27 (*Figure 4.1*).

Using the critical point, the elevation of the roadway at the east abutment can be calculated using *Equation 2*, shown below:

$$EL_{PGL} = WSEL_A + X + Y + Z \text{ (Eq-2)}$$

Where:

- EL_{PGL} = *Elevation at Profile Grade Line of Johnson Drive*
- $WSEL_A$ = *Water Surface Elevation at Point A*
- X = *Freeboard*
- Y = *Elevation drop from baseline of bridge to upstream edge (Refer to Section 4.2 of this report for information relating to the elevation drop)*
- Z = *Depth of Structure*

4.3.4 Optimal Profile Grade

Optimally, the bridge would be able to clear the 1% AEP WSEL and provide freeboard. If preferred values of freeboard are met (X = 2'), the equation for the elevation of the Johnson Drive profile grade line is as follows in Equation 2:

$$EL_{PGL} = 918.27 + 2' + (2\% * 35') + 18.5" = 922.51 \text{ (Eq-2)}$$

The plan and profile of Johnson Drive required to meet the profile grade elevation shown in *Equation 2* is shown in *Figure F.1* of Appendix F.

While this option is hydraulically optimal, the impacts to surrounding facilities are greatly increased by the significant increase in profile grade. *Table 4.3*, shown below, summarizes the grade difference between the optimal profile grade elevation and the existing grades.

Table 4.3 – Optimal Profile Raise

Elevation Differential			
Location	**Optimal Profile Elevation**	**Existing Grade**	**Grade Difference**
East Abutment	922.51'	914.89'	7.62'

4.3.5 USACE-Provided Conceptual Profile

USACE provided the study team with a proposed conceptual profile for Johnson Drive (refer to *Figure B.2* in Appendix B). The profile was received in image format. Therefore, to analyze the profile and assess potential impacts, elevations were interpolated using the embedded grid.

The profile elevation at the east abutment was estimated to be approximately 916.3', which sits approximately 2.3' below the no-widening WSEL (918.62') noted in *Table 4.1*. This was supported in

subsequent discussions with USACE, who confirmed pressurized flow was assumed with their conceptual design.

Although the USACE profile is an improvement over the existing condition, elevations at the east abutment fall short of what is required to raise the bridge completely out of the 1% WSEL. Using the recommended haunch slab bridge, along with the premise that pressurized flow would be acceptable, HNTB analyzed several profile options to reduce the risk of flooding while balancing impacts to surrounding infrastructure and property.

4.3.6 Allowable Profile Grade

Haunched concrete slab bridges offer many advantages when compared to other bridge types when considering hydraulic forces exerted upon the bridge.

- Buoyant forces are minimized due to the geometry of the bridge – negligible amounts of air are trapped between the bridge and the water during a flood.
- Resistance to overturning due to weight and width of bridge - water pressure acting upon the bridge can be accommodated in design.
- Barrier walls can be constructed at bridge fascias to tie into floodwalls to reduce or eliminate overtopping of the bridge.

Therefore, due to the ability to resist forces from floodwaters and the ability to maintain floodwall continuity across the bridge, the haunched concrete slab bridge can have the freeboard reduced to 1', or even eliminated. An alternative showing a profile grade where the bridge is partially in the water is depicted in *Figure F.2* in Appendix F.

While not analyzed in the draft phase of this study, the bridge profile will have effects on the WSEL. If the bridge profile provides more length along the bridge with freeboard, there could be improved WSELs. However, additional interference with the flow due to structures will lead to worsened WSELs at the Johnson Drive bridge. Further iteration of hydraulic analysis is needed to determine the water surface profile of any suggested bridge profiles.

5. Recommended Civil Design

The recommended civil design combines the following recommendations throughout this report:

- Widen Turkey Creek channel to the west (Section 3.3.1.2)
- Replace the existing bridge with a concrete haunched slab bridge (Section 3.3.2.2)
- Allow bridge to be under pressurized flow (Section 4.3.5)
- Profile changes to Johnson Drive and Merriam Drive (Section 4.3.6)

The drawings for the recommended civil site layout can be found in Appendix F in *Figures F.2*, *F.3*, and *F.4*.

It should be noted that all estimated property impacts are based on USACE-provided topographic survey and, where necessary to fill in gaps, contours and elevation data obtained from AIMS. The smallest contour intervals are limited to 2' and it is unknown what level of error they contain. Visual examination in the field suggests there may be opportunity to mitigate many of the impacts during more detailed design.

5.1 Bridge Profile Over Turkey Creek

For a view of the Johnson Drive profile, refer to *Figure F.2* in Appendix F. The proposed profile is located below the 1% AEP WSEL for most of the length of the bridge; however, the west portion of the bridge clears the 1% AEP WSEL. To minimize the risk of flooding, it is recommended to include integral structural barrier walls along the bridge fascia. The barrier walls would tie-in to the adjacent floodwalls to provide continuity and reduce the potential for floodwater to overtop the bridge.

5.2 Potential Impacts East of Turkey Creek

The downtown Merriam area is expected to be moderately impacted. Refer to *Figure F.3* in Appendix F for an exhibit showing civil features east of Turkey Creek.

North of Johnson Drive, six (6) properties are estimated to be impacted. Of the six (6) properties impacted, four (4) include structures that may impacted by the roadway improvements. Structures were identified as impacted if grades were raised by 6" or more at any point in the building footprint. If grades were raised by less than 6", it was assumed that design of the roadway and sidewalk may be able to mitigate the grade raise and avoid impacts. Refer to Figure G.4 in Appendix G for conceptual property impacts for the recommended option. It should be noted that many of the buildings have a step up from the sidewalk into the doorway – sidewalk grade raises up to the door level should be considered a practical approach to keep the structure intact. If sidewalk grades are raised above the door level, the property should be considered for a total acquisition.

South of Johnson Drive, the BP gas station on the southeast corner may lose driveway access immediately east of the intersection due to profile grade changes. At this location, Johnson Drive is anticipated to be

approximately 1.5' higher than the existing elevation. If realized, there is potential for this to cause a significant loss of use for the business and could result in the property as a total acquisition.

5.3 Potential Impacts West of Turkey Creek

Due to the widened Turkey Creek channel, Campbell Lane will need to be closed from Johnson Drive to W 60th Street. Refer to *Figure F.4* in Appendix F for an exhibit depicting civil features west of Turkey Creek

Currently, Campbell Lane only serves one (1) property between 60th Street and Johnson Drive – the Merriam Christian Church. To continue service to the church's parking lot, there are a couple of possibilities. One option would be to convert Campbell Lane to a one-way driveway in tandem with the existing driveway located off Johnson Drive. However, this option limits the available space for levee construction. An alternative solution would be to reconstruct and widen the existing driveway off Johnson Drive to allow two-way traffic. In this scenario, parking lot access to Campbell Lane would be closed and all access to the church would enter and exit from Johnson Drive. The existing driveway sits primarily on church owned property. The house between the church parcels, located at 9417 Johnson Drive, appears to potentially be vacant. Purchase of the house could be investigated to provide a wider, safer commercial entrance along Johnson Drive.

The other primary consideration for channel widening is the Turkey Creek Streamway Trail. The trail is a key feature for the area and is a critical component of the City's overall pedestrian connectivity plan. Relocation along the new west abutment is assumed as noted on Figure D.3. Design should account for vertical clearance and interaction with the abutment and flood wall. South of the Johnson Drive bridge, the trial could either be shifted west to the vacated Campbell Lane or incorporated into a levee, depending on the flood protection measure implemented.

5.4 Potential Utility Impacts

As noted in Section 2.2.3, an associated and concurrent USACE project is performing in-depth utility analysis and coordination for the Johnson Drive corridor. Full utility coordination and relocation plans require more time and effort than what was covered by this report. Therefore, the information provided is for reference only and no relocation quantities are included, as that level of detail is still unknown.

One important consideration to note is that relocation costs are typically only reimbursed when utilities are located within private easement. When located within ROW, relocation costs are nearly always encumbered by the utility. Therefore, without a more thorough investigation into potential easements, it is assumed that any utility located within existing ROW would be fully responsible for relocation costs.

5.4.1 Evergy

Facilities are generally aerial, with all poles located within ROW, either in the green space at the back of curb or in the adjacent sidewalk. Unless there is a crash history associated with a specific pole, these

conditions are typically allowed to persist. Because there is no roadway widening, we are assuming the current location of all poles is acceptable.

The more likely scenario that may require some pole modifications revolves around a lack of required vertical clearance due to the Johnson Drive profile raise. This could be due to the electrical wires themselves, or the attached fiber utilities located below. There are two poles that may not meet vertical requirements: at the southeast corner of the Johnson Drive and Merriam Drive intersection and approximately 50-ft north of the bridge. However, more information is needed to determine if proposed conditions would be acceptable or if attached utilities could relocate.

For a conservative approach, these two locations could be estimated to be reset with taller poles. However, as noted, it is assumed these costs would be incurred by Evergy.

5.4.2 Johnson County Wastewater

There are two manholes that will need to be raised (southeast corner of the bridge in the approach pavement and approximately 50-ft north of the bridge, next to the Evergy pole). A third manhole located about 17-ft from the edge of pavement in the northwest corner of the Johnson Drive and Merriam Drive intersection may need to be raised and/or modified (e.g. locking/sealed) due to Turkey Creek WSELs. This location would not need to be raised due to profile changes as design components could limit impacts.

Compensability for wastewater manholes is less straightforward. Typically, this is a bid item that is included in the construction of the proposed improvements with the project.

5.4.3 Kansas Gas / WaterOne

Individual valves within the roadway will need to be raised, however, gas and water lines should be located sufficiently deep enough that relocation is not required due to construction activity alone. A common practice in these situations is for contractors to locate and protect lines in place. However, it is possible that either or both utilities may request to have their line(s) relocated for several reasons: concern over the additional 1-ft to 2-ft of coverage, aging line(s) that needs to be replaced, or a desire to upsize their line(s). In such instances, gas and water utilities often take advantage of construction projects as an opportunity to make improvements.

Both gas and water utilities appear to be located within existing ROW and any relocation costs are assumed to be incurred by the respective utility.

5.4.4 AT&T / Everfast / Google / Spectrum / Verizon

All known fiber utility locations are within ROW. Relocation costs are assumed to be the responsibility of each utility owner.

5.5 Quantities

Quantities associated with the recommended design are shown in Appendix G.

APPENDIX A – DESIGN CRITERIA

Table A.1 – Design Criteria

UTC Civil Design Criteria				
Criteria	**Johnson Dr**	**Merriam Dr**	**Sidestreets**	**Source**
Roadway Classification	Minor Arterial	Minor Arterial	Local	MARC Functional Classification
Design Speed (mph) [Min./Used]	30/35	30/30	20/25	AASHTO GDHS-7 5.3.1.1/7.3.2.1
Posted Speed (mph)	35	30	25	Existing Posted Speed
Typical Section				
Lane Width (ft)	11’	11’	12’	AASHTO GDHS-7 7.3.3.2/5.3.2.1
Vertical Design				
Min. Vertical Grade	1%	1%	1%	KC APWA Section 5203
Max. Vertical Grade	7%	7%	8%	AASHTO GDHS-7 5.3.1.5/7.3.2.6
K-Values (min.)				
Sag Vertical[1] (Preferred/Comfort)	49 / 27	37 / 20	26 / 14	AASHTO GDHS-7 Table 3-37
Crest Vertical	29	19	12	AASHTO GDHS-7 Table 3-35
Stopping Sight Distance (ft)	250	200	155	AASHTO GDHS-7 Table 3-1
Min. Curve Length (ft)	105	90	75	AASHTO GDHS-7 3.4.6.2.1
Max. Grade Break	1%	1%	1%	AASHTO GDHS-7 3.4.6.3
Bridge Design				
Freeboard (Over 100-Yr WSP)	2’	2’	N/A	KDOT Bridge Manual 2.3.3.1.7
Pedestrian & Cyclist				
Sidewalk/SUP Cross Slope	1%			AASHTO GDBF 6.6.5
Sidewalk Width (ft)	5			PROWAG R302
SUP Width (ft)	10			AASHTO GDBF 6.4.3
Constrained SUP Width (ft)	8			AASHTO GDBF 6.4.3
Max. Vertical Grade	5%			PROWAG R302.4.1
Min. Vertical Grade	0.5%			AASHTO GDBF 6.6.6
Ramp Max Grade	8.3%			PROWAG R407.2
Ramp Max Rise (Inch)	30			PROWAG R407.5
Miscellaneous				
Curb Return Radii (ft)	35	35	30	KC APWA Section 5203
Clear Zone (ft)	20	12-16		AASHTO RDG Table 3-1
Cul-de-sac Radius (ft)	30			AASHTO GDHS-7 Figure 5-1

1. Comfort criteria defined by $K = V^2/46.5$

SOURCES:

- "AASHTO GDBF" = AASHTO. *Guide for the Development of Bicycle Facilities.* 5th Edition. (2024)
- "AASHTO GDHS-7" = AASHTO. *A Policy on Geometric Design of Highways and Streets.* 7th Edition. (2018)
- "KDOT Bridge Manual" = Kansas Department of Transportation. *Bridge Design Manual.* (2021)
- "KC APWA Section 5203" = Kansas City Metropolitan Chapter of the American Public Works Association. *Design Criteria.* "Section 5200 Streets". (1996)
- "MARC Functional Classification" = Mid-America Regional Council. Roadway Functional Classification System. https://marc-gis.maps.arcgis.com/apps/webappviewer/
- "PROWAG" = U.S. Access Board. *Public Right-of-Way Accessibility Guidelines.* (2023) / 36 CFR Part 1190

APPENDIX B – USACE-PROVIDED DRAWINGS

Figure B.1 – RCB Bridge Concept

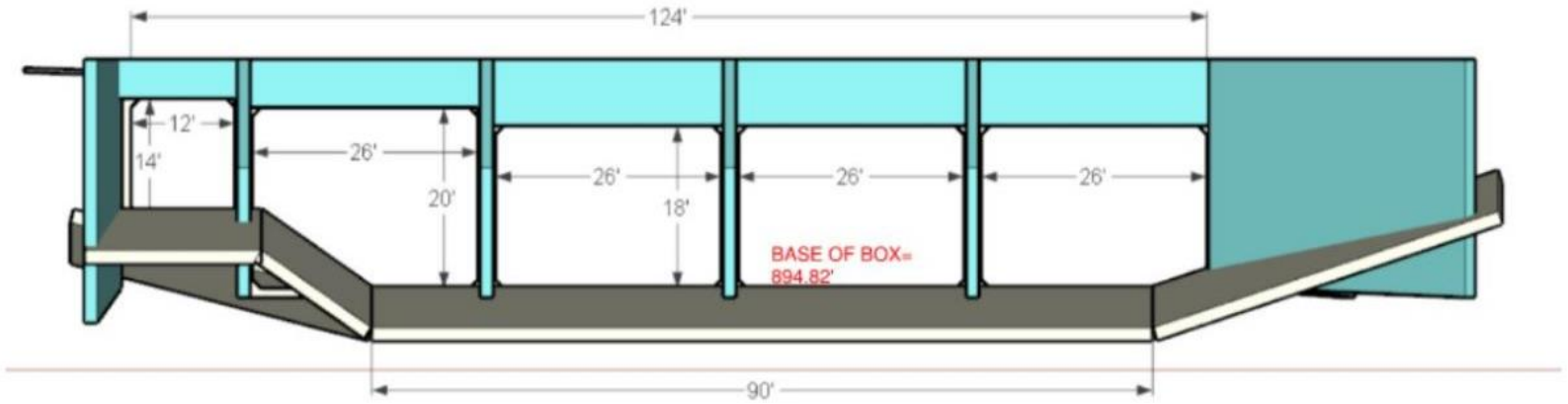
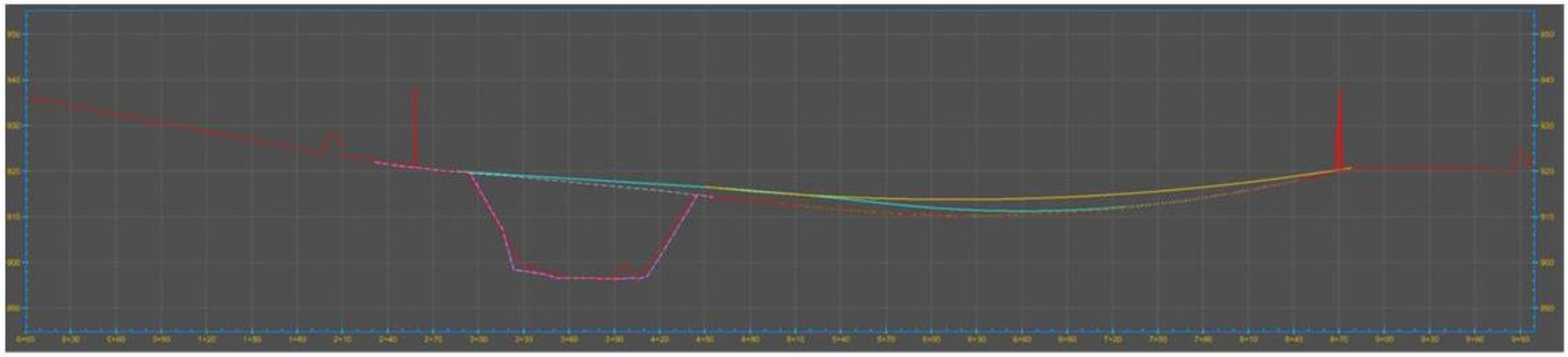


Figure B.2 – USACE Johnson Drive Profile



Yellow = Meets AASHTO curve requirements
Light Blue = matches existing vertical curves

APPENDIX C – EVALUATION MATRIX

Table C.1 – Project Approach Evaluation Matrix

Alternative	Evaluation Category								TOTAL SCORE	Concept Notes	Recommended Alternative
	Costs										
	Bridge	Impacts to Adjacent Infrastructure	Utility Relocations	ROW/Easements	Miscellaneous/Aesthetics	Capacity for Levee Over Floodwall	MOT/Construction Length	Impacts to Natural Environment			
Name	2	2	2	2	1	5	3	1			
Project Approach											
Raise Bridge Deck	3	1	1	1	1	1	3	0	27	Involves raising grades of all surrounding roadways to tie into raised bridge.	
Widen Creek Channel to West	1	4	3	2	2	2	2	0	38	Would involve closure of Campbell Lane from Johnson Drive to 60th St.	
Widen Creek Channel to East	2	2	2	2	1	2	1	0	30	Two Options: A. Fix intersection corner radii by narrowing Merriam Dr to a one-way section north of Johnson Dr. B. Keep existing, inadequate corner radii but reduce scale of issue by removing west-most lane of Merriam Dr.	

Table C.2 – Bridge Type Evaluation Matrix

Alternative	Evaluation Category								TOTAL SCORE	Recommended Alternative
	Costs									
	Bridge	Impacts to Adjacent Infrastructure	Utility Relocations	ROW/Easements	Miscellaneous/Aesthetics	Capacity for Levee Over Floodwall	MOT/Construction Length	Impacts to Natural Environment		
Name	2	2	2	2	1	5	3	1		
Bridge Type										
RCB *(Reinforced Concrete Box Culvert)*	5	0	3	0	0	0	5	1	14	
Shallow Superstructure *(Concrete Haunched Slab Bridge)*	4	3	3	0	0	0	4	2	16	
Remove Midspan Bents *(Truss/Arch Bridges)*	2	0	1	0	0	0	2	3	8	

SAMPLE CALCULATION (RAISE BRIDGE DECK)

Alternative	Evaluation Category: Costs: Bridge	Impacts to Adjacent Infrastructure	Utility Relocations	ROW/Easements	Miscellaneous/Aesthetics	Capacity for Levee Over Floodwall	MOT/Construction Length	Impacts to Natural Environment	TOTAL SCORE	Concept Notes	Recommended Alternative
Name	2	2	2	2	1	5	3	1			
Project Approach											
Raise Bridge Deck	3	1	1	1	1	1	3	0	27	Involves raising grades of all surrounding roadways to tie into raised bridge.	

$3 * 2 = 6$

Eq 3-1:

- $\sum_{i=1}^{X} C_i * W_i = T$

Values Plugged into Eq 3-1:

- $T = (3 * 2) + (1 * 2) + (1 * 2) + (1 * 2) + (1 * 1) + (1 * 5) + (3 * 3) + (0 * 1) = 27$

APPENDIX D – HYDRAULIC ANALYSIS

Figure D.1 – Existing Bridge Section Analyzed

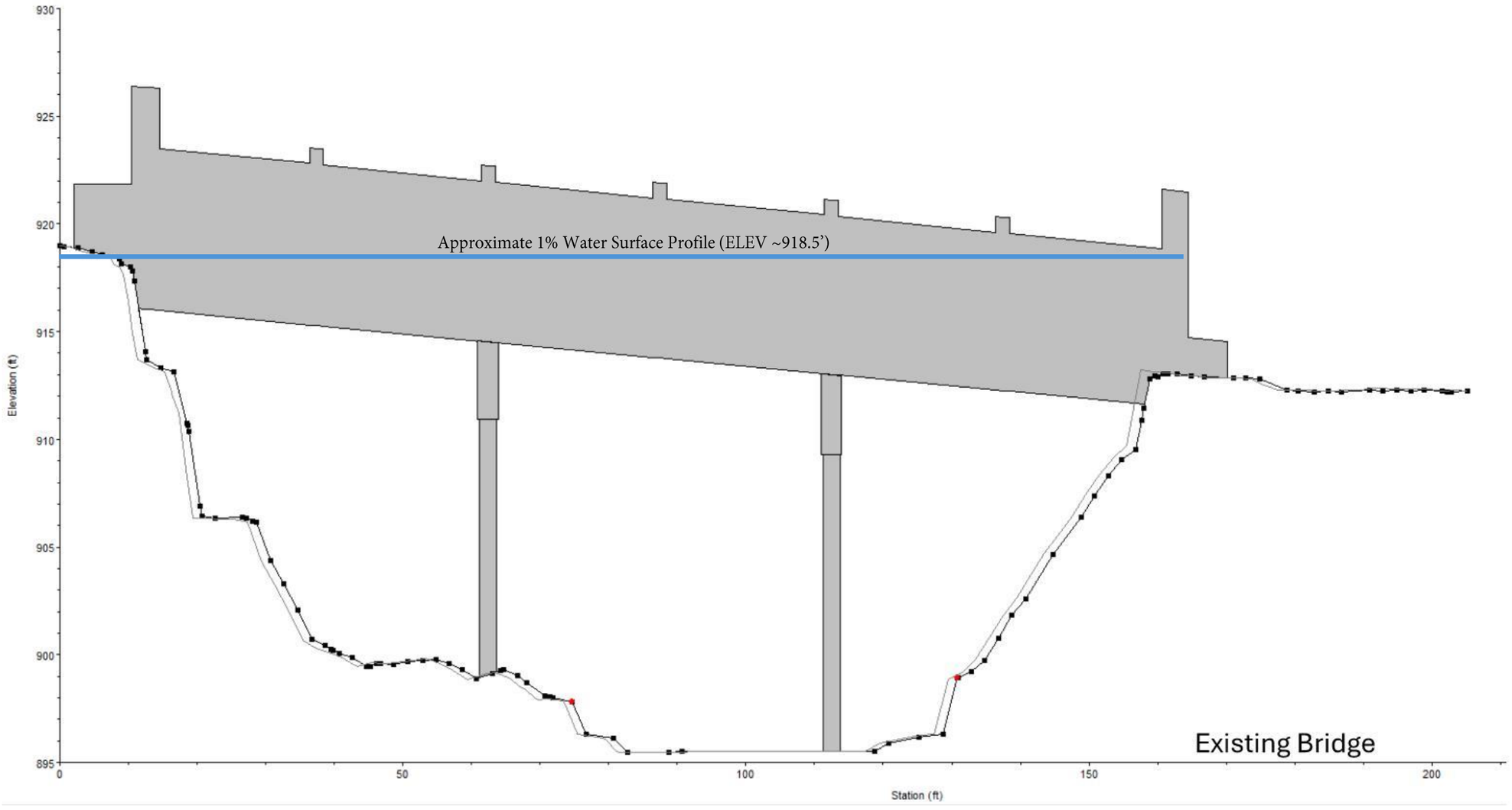


Figure D.2 – Concrete Haunched Slab Bridge

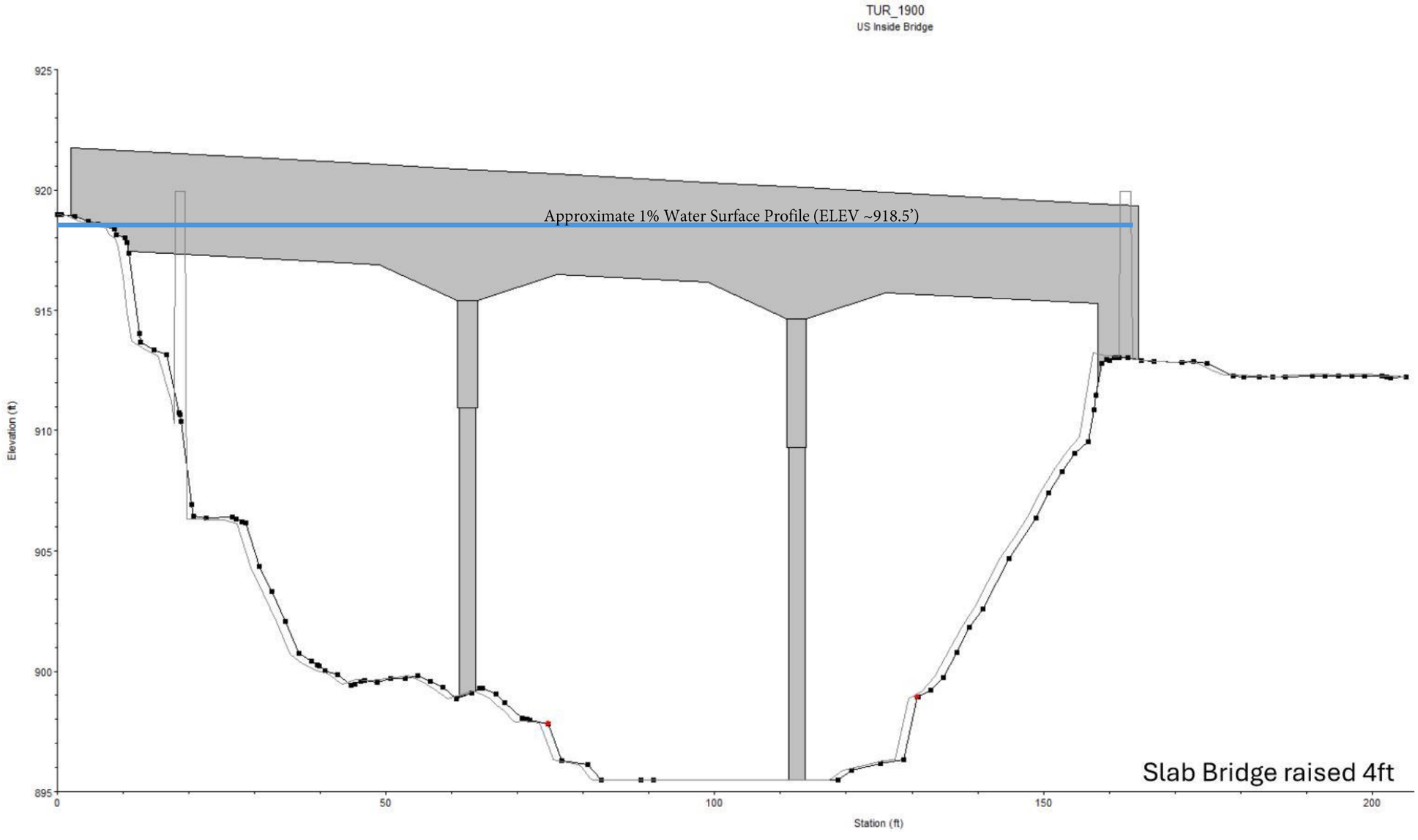
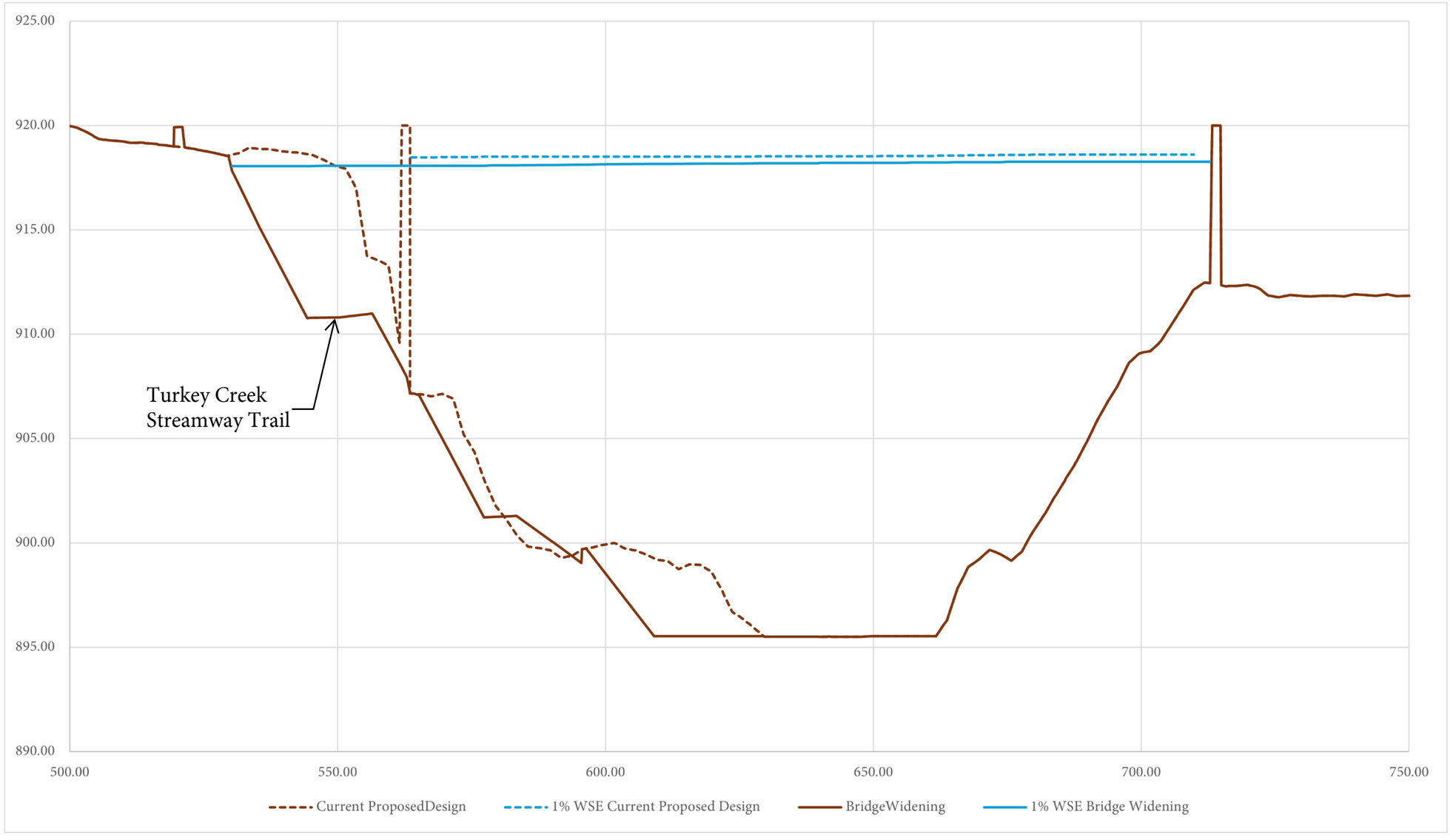
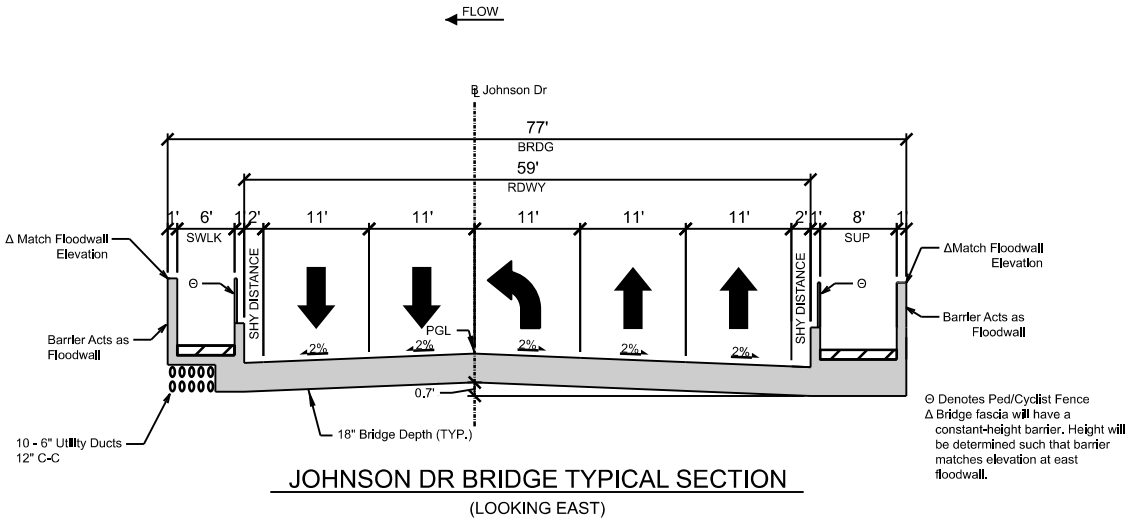


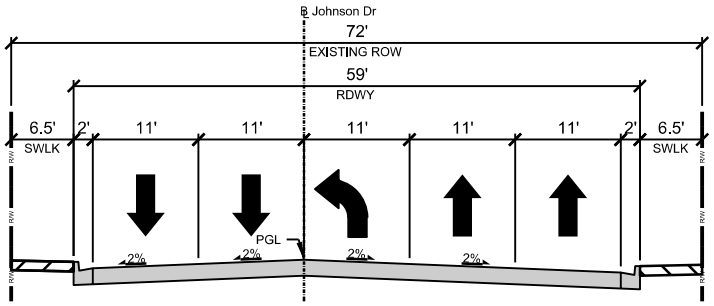
Figure D.3 – Water Surface Profiles



APPENDIX E – PROPOSED TYPICAL SECTIONS



JOHNSON DR BRIDGE TYPICAL SECTION
(LOOKING EAST)



JOHNSON DR TYPICAL SECTION
(LOOKING EAST)

Figure E.1 – Johnson Drive Typical Sections

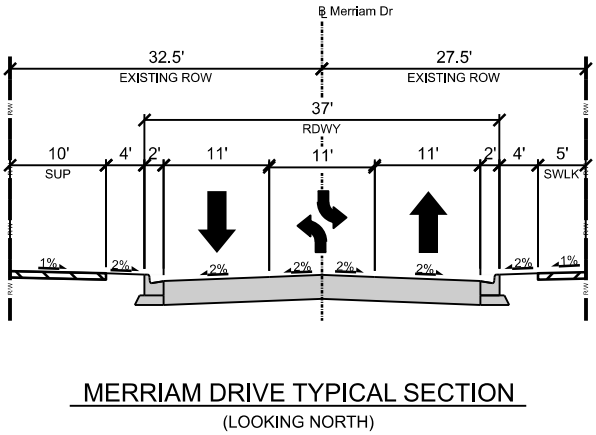
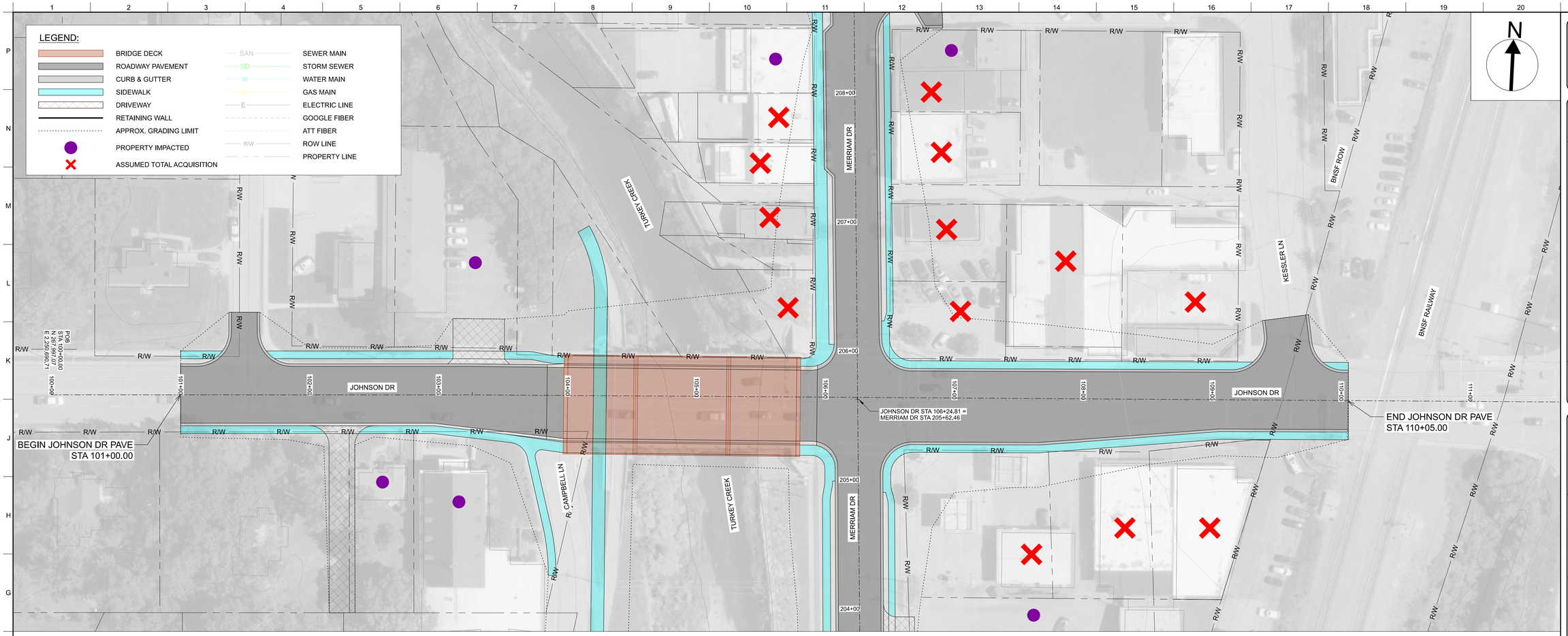


Figure E.2 – Merriam Drive Typical Section

APPENDIX F – CONCEPT EXHIBITS

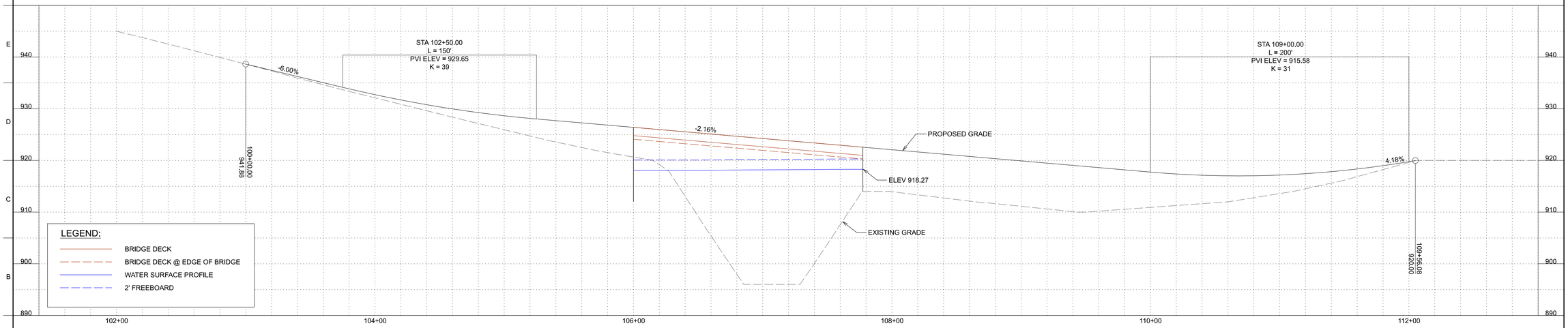


1 JOHNSON DRIVE PLAN

SCALE: 1" = 40'

SCALE

40' 0 40' 80'



2 JOHNSON DRIVE - OPTIMAL PROFILE

SCALE: H 1"=40' V 1" = 10'

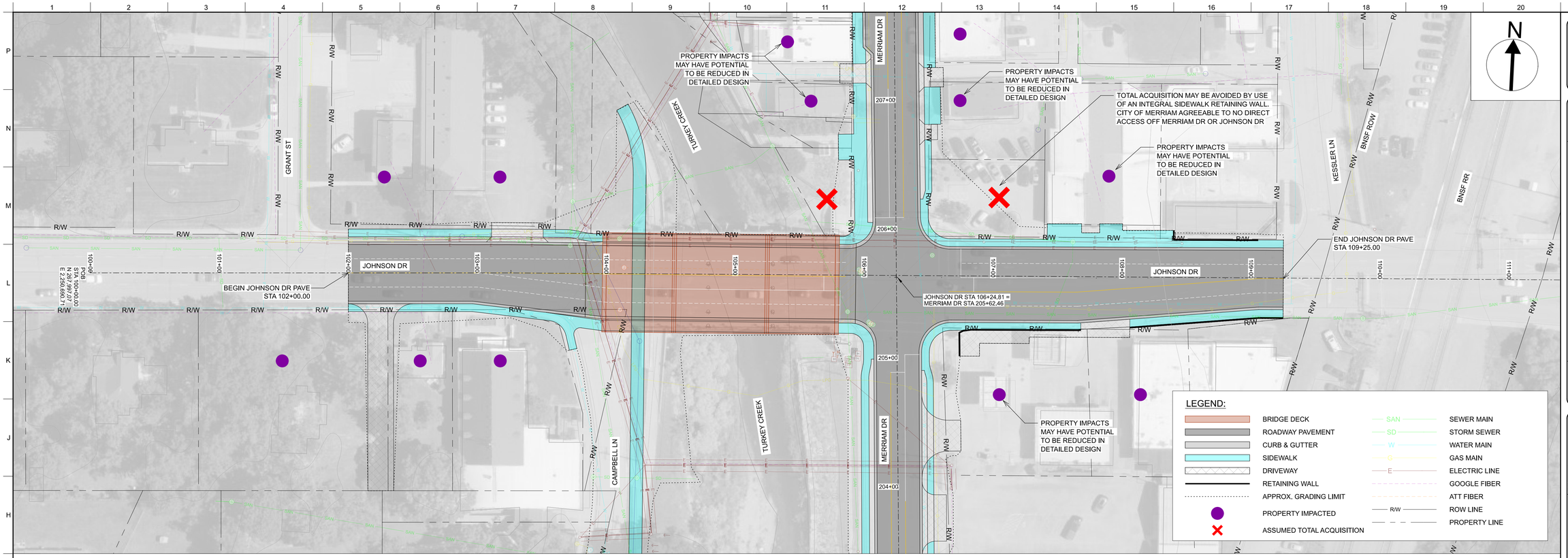
MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SIZE:
ISSUE DATE:
SOLICITATION NO.:
CONTRACT NO.:

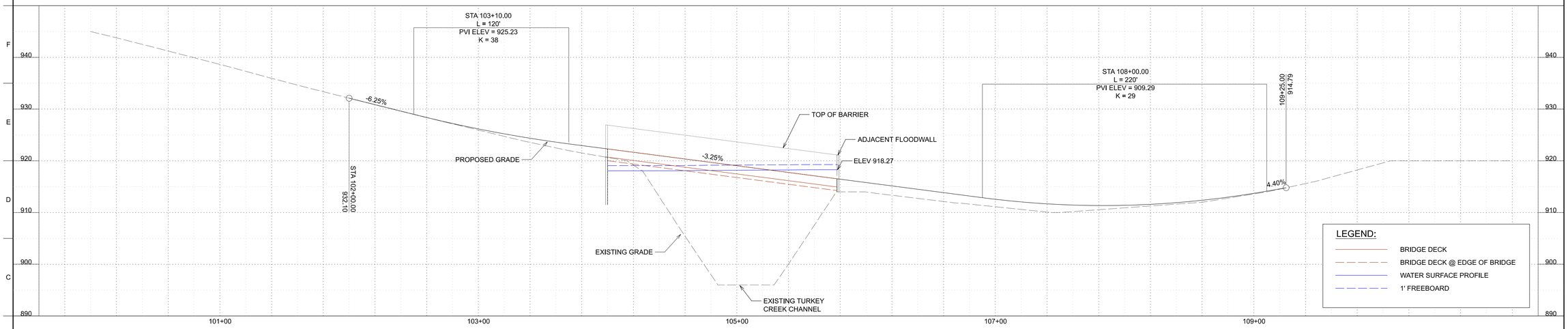
JOHNSON DRIVE - PLAN AND PROFILE
OPTIMAL PROFILE

SHEET ID

FIGURE
F.1



1 JOHNSON DR PLAN
SCALE: 1" = 40'



2 JOHNSON DR - RECOMMENDED PROFILE
SCALE: H 1"=40' V 1" = 10'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

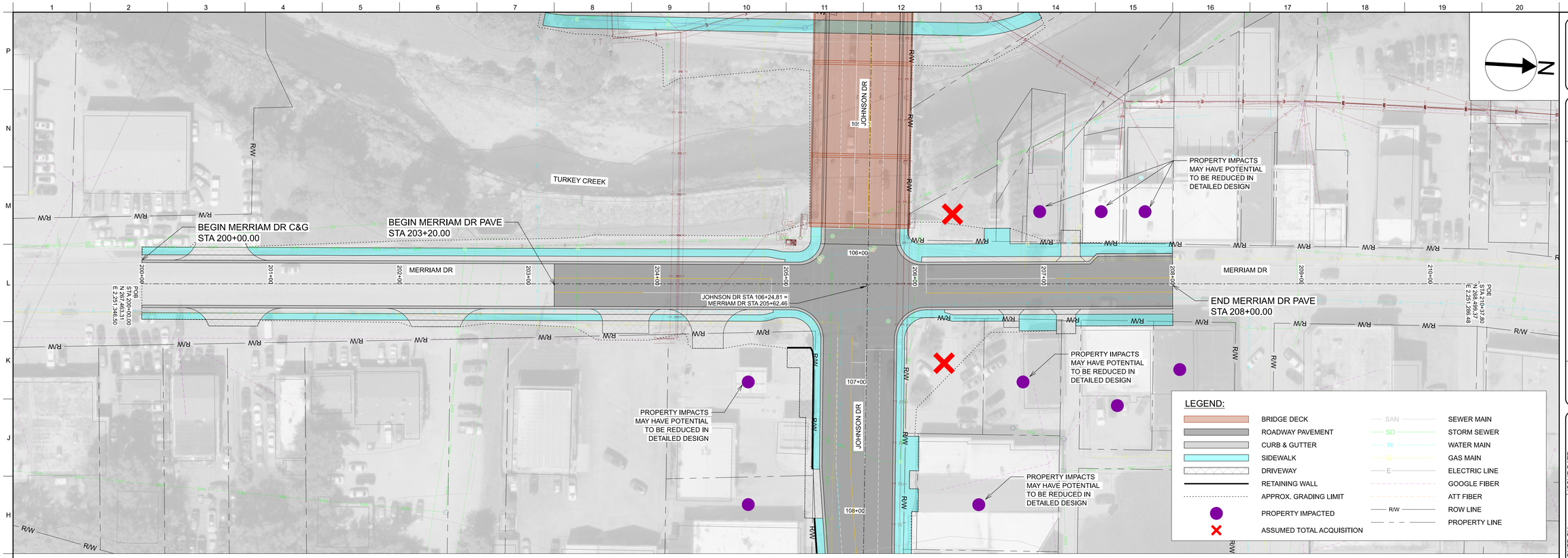
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SIZE:
ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
715 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

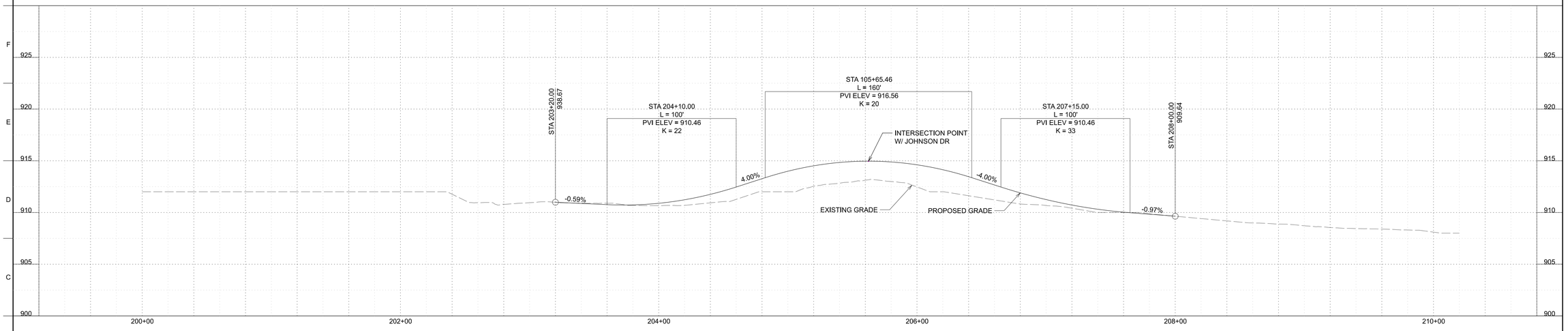
JOHNSON DRIVE - PLAN AND PROFILE
RECOMMENDED ALTERNATIVE

SHEET ID
FIGURE F.2



1 MERRIAM DR PLAN

SCALE: 1" = 40'



2 MERRIAM DR RECOMMENDED PROFILE

SCALE: H 1"=40' V 1" = 5'

US Army Corps of Engineers®

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
715 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:
SIZE:

MERRIAM DRIVE - PLAN AND PROFILE
RECOMMENDED ALTERNATIVE

SHEET ID

FIGURE
F.3



1 CAMPBELL LN PLAN
SCALE: 1" = 40'

SCALE
40' 0 40' 80'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SIZE:
ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
715 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

CAMPBELL LANE PLAN
RECOMMENDED ALTERNATIVE

SHEET ID
FIGURE F.4

APPENDIX G – QUANTITIES

Table G.1 -Quantities

UTC Johnson Drive Replacement Quantities				
Group	**Item**	**Qty**	**Unit**	**Assumptions***
Roadway Paving	2" Asphalt Surface	530	TON	No pavement design completed, density of 150 PCF
	8" Asphalt Base	2,100	TON	No pavement design completed, density of 150 PCF
	6" Granular Base	5,510	SY	Under roadway & approach slab, 1' beyond BOC
	8" Modified Subgrade	5,510	SY	Under roadway & approach slab, 1' beyond BOC
	Curb and Gutter, Type B	2,660	LF	
Other Infrastructure	PCC Driveway (6")	1,085	SY	
	Sidewalk (4")	2,040	SY	Includes trail.
	Integral Sidewalk Retaining Wall (ISRW)	460	SF	Estimated average height of 1.5'
	Tract 10 ISRW	280	SF	Estimated average height of 2.0' See Figure G.4 for Tract numbers.
	15" RCP	120	LF	
	18" RCP	490	LF	
	Curb Inlet	5	EA	
	Storm Outfall	1	EA	
Bridge	Bridge	14,130	SF	Proposed bridge is haunched concrete slab bridge. Estimated at $203/SF (2024), includes structural fascia walls.
	Approach Slab	1,560	SF	Assumes 13' length & BOC-to-BOC width (see Figure F.2)
Lump Sum Items	Roadway Lighting	1	LS	Assumes 19 streetlights to be remove/reset, new bridge lighting, and all existing lighting is sufficient. Assumes light fixtures are not damaged.
	Sodding	1	LS	Non-paved area = 1.3 Acres
	Traffic Signal	1	LS	Assume full replacement of 4 signal assemblies
	Signing and Striping	1	LS	Assume Signing/Striping is 10% of total roadway paving costs.
	Erosion Control	1	LS	Disturbed Area = 3.4 Acres. Includes paved areas.
	Traffic Control	1	LS	Assume Traffic Control is 15% of total Roadway Paving costs.

(TABLE CONTINUES ON NEXT PAGE)

Demolition	Remove Pavement	7,410	SY	Existing pavement depths/types not known. Assume 18” depth.
	Remove Curb and Gutter	3,670	LF	
	Remove Driveway	1,070	SY	
	Remove Bricks	325	SY	Brick pavers in downtown Merriam
	Remove Sidewalk	1,640	SY	Trail included in quantity.
	Remove Bridge Approach	1,191	SF	
	Remove Bridge	11,174	SF	
	Remove 15” RCP	65	LF	
	Remove 15” CMP	90	LF	
	Remove 18” CMP	535	LF	
	Remove Curb Inlet	7	EA	Includes manholes, inlets, and aprons.
	Modify Storm Structure	4	EA	Includes changing top of storm structures to/from inlets.
	Remove Storm Outfall	2	EA	
	Remove Retaining Wall	865	LF	Includes large block, small block, integral sidewalk walls
Earthwork	Cut	2,210	CY	Refer to following sheet for basis of quantity.
	Fill	2,360	CY	Refer to following sheet for basis of quantity.

Table G.2 - ROW Quantities

UTC Johnson Drive Bridge Replacement ROW Quantities					
Tract	Type	Area [SF]	Owner	Tax ID	Address
3	TCE	398	Karma Holdings LLC	JP84000000 000B1	9424 Johnson Dr
4	TCE	950	Turkey Creek Ventures LLC	JP84000000 000B3	9400 Johnson Dr
6	TCE	4,212	Merriam Christian Church	JP46000000 0007	9503 Johnson Dr
7	TCE	1,802	Merriam Properties LLC	JP46000000 0006	9417 Johnson Dr
8	TCE	608	Merriam Christian Church	JP46000000 0004	9401 Johnson Dr
8b*	Acquisition	3,560	Merriam Christian Church	JP46000000 0004	9401 Johnson Dr
9	Acquisition	2,691	City Of Merriam	JP84000009 006C	N/A
10	Acquisition	5,399	City Of Merriam	JR84000010 0010A	N/A
11	TCE	1,322	AJ Smith Property LLC	JP84000010 0013	9320 Johnson Dr
12	TCE	544	AJ Smith Property LLC	JP84000010 0016	9214 Johnson Dr
13	TCE	3,556	Sood Brothers LLC	JP14000003 0002B	5903 Merriam Dr
14	TCE	780	Merriam Real Estate LLC	JP14000003 0001	9309 Johnson Dr
33	TCE	600	Hernandez, Angel	JP84000010 0010B	5853 Merriam Dr
34	TCE	282	Leap, Billy D.	JP84000010 0008B	5847 Merriam Dr
35	TCE	318	True Auto Parts, Inc	JP84000010 0006	5837 Merriam Dr
47	TCE	2,895	Merriam Christian Church	JF241212-4028	N/A

*Approximated area that may be legally owned by the City of Merriam due to KS Statute 60-503 (Adverse Possession).

G.1 Earthwork Estimation Notes:

Earthwork for the project is based upon conceptual corridor models developed. It should be noted that the existing grade surface was determined from AIMS contours, and surface elevations may not accurately reflect true conditions. All earthwork should be considered conceptual for order-of-magnitude purposes only.

The following assumptions were made for earthwork calculations:

- No fill factor assumed.
- Average end-area method used. Assumed widening to tie-in to existing channel cross-sections 100' upstream and downstream of bridge cross-section.
- Channel widening estimated based on cross-section provided by USACE (see Figure D.3).
- Autodesk Civil 3D TIN Volume Surface volumes are used to compute volumes for any proposed roadway grade changes.

G.1.1 – Channel Widening Volumes

Table G.3 - Channel Widening Volumes

Station*** (Turkey Creek)	Description	Cut Area [SF]	Fill Area [SF]	Cut Volume [CF]	Fill Volume [CF]	Cut Volume [CY]	Fill Volume [CY]
0+00	Start widening	0	0	0	0	0	0
1+00	Full Widening	259*	8*	12,950	400	480	15
2+00	Full Widening	259*	8*	25,900	800	960	30
3+00	End Widening	0	0	12,590	400	480	15
			TOTAL	**51,800**	**1,600**	**2,000****	**100****

*Refer to Figure G.3 for area derivation.

**Due to limited accuracy of input information for channel widening, rounded to one significant digit.

***Stations are arbitrary for measurement purposes and do not match USACE stationing.

Sample Calculations:

$$Cut\ Volume = (Area_2 + Area_1) * (STA_2 - STA_1) * \frac{1}{2}$$

[For Station 1+00] → $(259\ SF + 0\ SF) * (100\ LF - 0\ LF) * \frac{1}{2} = 12{,}950\ CF$

$$12{,}950\ CF * \left({}^{1\ CY}/_{27\ CF}\right) = 480\ CY$$

G.1.2 Civil 3D Generated Volumes:

West of Turkey Creek:

Surface Properties - West-Volume-Surface

Information | Definition | Analysis | Statistics

Statistics	Value
General	
TIN	
Volume	
Base Surface	UTC_AIMS-Exist-Grade
Comparison Surface	UTC-Johnson-Bridge_West-Surface
Cut Factor	1.000
Fill Factor	1.000
Cut volume (adjusted)	70.06 Cu. Yd.
Fill volume (adjusted)	286.01 Cu. Yd.
Net volume (adjusted)	215.96 Cu. Yd.<Fill>
Cut volume (unadjus...	70.06 Cu. Yd.
Fill volume (unadjust...	286.01 Cu. Yd.
Net volume (unadjus...	215.96 Cu. Yd.<Fill>

Due to limited accuracy of input:

- Round Cut Volume to 70 CY
- Round Fill Volume to 290 CY

East of Turkey Creek:

Surface Properties - East-Volume-Surface

Information | Definition | Analysis | Statistics

Statistics	Value
General	
TIN	
Volume	
Base Surface	UTC_AIMS-Exist-Grade
Comparison Surface	UTC-Johnson-Bridge_East-Surface
Cut Factor	1.000
Fill Factor	1.000
Cut volume (adjusted)	134.01 Cu. Yd.
Fill volume (adjusted)	1966.94 Cu. Yd.
Net volume (adjusted)	1832.93 Cu. Yd.<Fill>
Cut volume (unadjus...	134.01 Cu. Yd.
Fill volume (unadjust...	1966.94 Cu. Yd.
Net volume (unadjus...	1832.93 Cu. Yd.<Fill>

Due to limited accuracy of input:

- Round Cut Volume to 140 CY
- Round Fill Volume to 1,970 CY



SCALE

40' 0 40' 80'

1 DEMOLITION PLAN (1 of 2)

SCALE: 1" = 40'

LEGEND:

	REMOVE PAVEMENT		REMOVE BRICK PAVING
	REMOVE SIDEWALK		REMOVE BRIDGE APPROACH
	REMOVE DRIVEWAY		REMOVE BRIDGE
SS	REMOVE STORM SEWER		REMOVE CURB AND GUTTER
LP	REMOVE/RESET STREET LIGHT		
	APPROX. GRADING LIMIT	E	ELECTRIC LINE
SAN	SEWER MAIN		GOOGLE FIBER
SD	STORM SEWER		ATT FIBER
W	WATER MAIN	R/W	ROW LINE
G	GAS MAIN		PROPERTY LINE

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SIZE:

ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
715 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

DEMOLITION PLAN
RECOMMENDED ALTERNATIVE

SHEET ID
FIGURE G.1



1 DEMOLITION PLAN (2 of 2)

SCALE: 1" = 40'

SCALE

40' 0 40' 80'

LEGEND:

REMOVE PAVEMENT	REMOVE BRICK PAVING
REMOVE SIDEWALK	REMOVE BRIDGE APPROACH
REMOVE DRIVEWAY	REMOVE BRIDGE
REMOVE STORM SEWER	REMOVE CURB AND GUTTER
REMOVE/RESET STREET LIGHT	
APPROX. GRADING LIMIT	ELECTRIC LINE
SEWER MAIN	GOOGLE FIBER
STORM SEWER	ATT FIBER
WATER MAIN	ROW LINE
GAS MAIN	PROPERTY LINE

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SIZE:
ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:

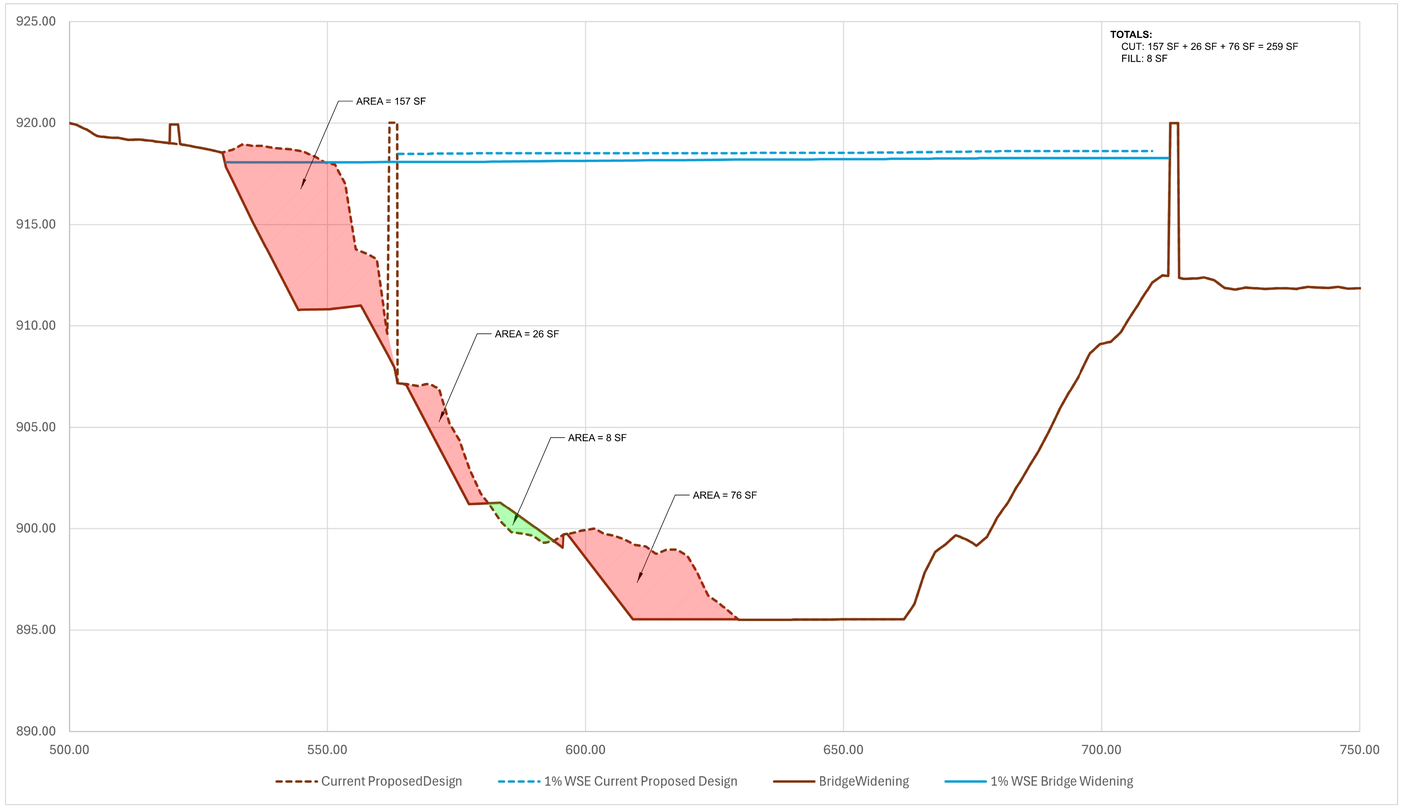
U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
HNTB CORPORATION
715 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

DEMOLITION PLAN
RECOMMENDED ALTERNATIVE

SHEET ID

FIGURE G.2



1 USACE-PROVIDED CHANNEL WIDENING CROSS SECTION

NOT TO SCALE

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SIZE:

ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:

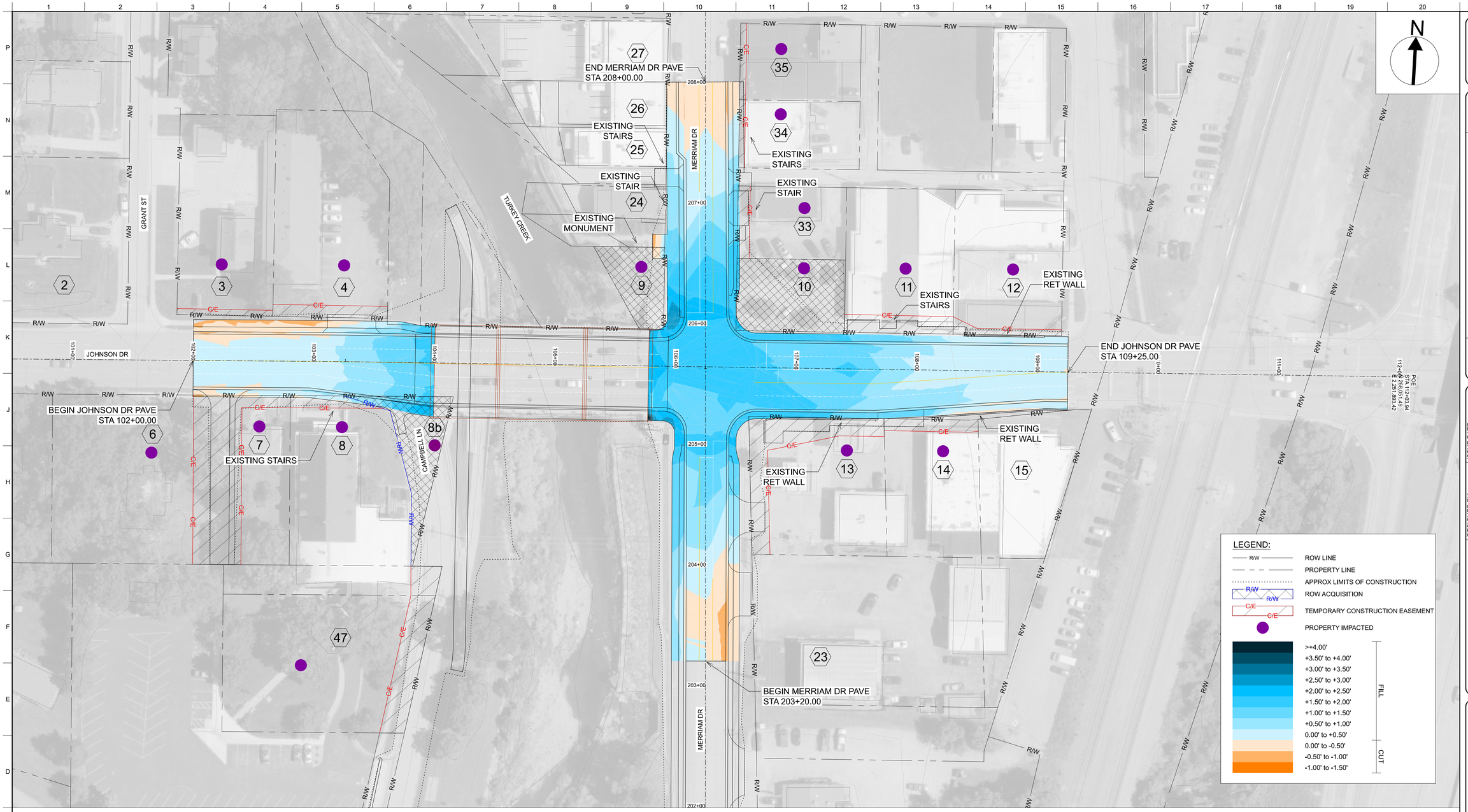
U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
716 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

CHANNEL WIDENING CROSS SECTION

SHEET ID

FIGURE G.3



1 ESTIMATED GRADING PLAN

SCALE: 1" = 40'

NOTE:
1) ELEVATION CHANGES AND GRADING LIMITS ARE SHOWN FOR REFERENCE ONLY TO ESTIMATE PROPERTY IMPACTS. SHADING LIMITS DO NOT ADDRESS TIE-INS OR REPRESENT ACTUAL ANTICIPATED CONSTRUCTION LIMITS.
2) TCE = TEMPORARY CONSTRUCTION EASEMENT

TRACT	TYPE	AREA [SF]	OWNER	TAX ID	ADDRESS
3	TCE	398	KARMA HOLDINGS LLC	JP84000000 000B1	9424 JOHNSON DR
4	TCE	950	TURKEY CREEK VENTURES LLC	JP84000000 000B3	9400 JOHNSON DR
6	TCE	4,212	MERRIAM CHRISTIAN CHURCH	JP46000000 0007	9503 JOHNSON DR
7	TCE	1,802	MERRIAM PROPERTIES LLC	JP46000000 0006	9417 JOHNSON DR
8	TCE	608	MERRIAM CHRISTIAN CHURCH	JP46000000 0004	9401 JOHNSON DR
8b*	ACQUISITION	3,560	MERRIAM CHRISTIAN CHURCH	JP46000000 0004	9401 JOHNSON DR
9	ACQUISITION	2,691	CITY OF MERRIAM	JP84000009 006C	N/A
10	ACQUISITION	5,399	CITY OF MERRIAM	JR84000010 0010A	N/A
11	TCE	1,322	AJ SMITH PROPERTY LLC	JP84000010 0013	9320 JOHNSON DR
12	TCE	544	AJ SMITH PROPERTY LLC	JP84000010 0016	9214 JOHNSON DR
13	TCE	3,556	SOOD BROTHERS LLC	JP14000003 0002B	5903 MERRIAM DR
14	TCE	780	MERRIAM REAL ESTATE LLC	JP14000003 0001	9309 JOHNSON DR
33	TCE	600	HERNANDEZ, ANGEL	JP84000010 0010B	5853 MERRIAM DR
34	TCE	282	LEAP, BILLY D.	JP84000010 0008B	5847 MERRIAM DR
35	TCE	318	TRUE AUTO PARTS, INC	JP84000010 0006	5837 MERRIAM DR
47	TCE	2,895	MERRIAM CHRISTIAN CHURCH	JF241212-4028	N/A

TABLE NOTE:
*APPROXIMATED AREA THAT MAY BE LEGALLY OWNED BY THE CITY OF MERRIAM DUE TO KS STATUTE 60-503 (ADVERSE POSSESSION)

SCALE
40' 0 40' 80'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
ISSUE DATE: JULY 2025
SOLICITATION NO.:
CONTRACT NO.:
SIZE:

U.S. ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
601 E. 12TH STREET
KANSAS CITY, MO 64106

HNTB
715 KIRK DRIVE
KANSAS CITY, MO 64105-1310
TELEPHONE (816) 472-1201

EARTHWORK DEPTH PLAN
RECOMMENDED ALTERNATIVE

SHEET ID
FIGURE G.4

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Johnson Drive Bridge Design

UPPER TURKEY CREEK

MERRIAM, KS

Include << .\A - User Defined Units and Functions.mcdx

GENERAL

Traffic Information

$ADT := 13920$ *(average daily traffic; SI&A Data, NBI Item 29)*

$ADTT_{percent} := 10\%$ *(average daily truck traffic in percentage of ADT; assumed)*

$ADTT := (ADTT_{percent}) \cdot (ADT) = 1392$ *(average daily truck traffic)*

$v_{PSL} := 35\ mph$ *(posted speed limited)*

$v_{DSL} := 45\ mph$ *(design speed limited; assumed)*

$N_L := 5$ *(number of design lanes)*

$n_L := 3$ *(number lanes with traffic in the same direction)*

Material Properties

$\gamma_c := 150\ pcf$ *(unit weight of concrete)*

$\gamma_w := 62.4\ pcf$ *(unit weight of water)*

Hydraulic Properties

$Q_{US} := 17810\ cfs$ *(upstream flowrate; NWK H&H)*

$V_{US} := 8.8\ fps$ *(maximum upstream velocity; NWK H&H)*

Note: The flat face of the flood wall will have a drag coefficient similar to a square-ended pier.

$(C_D)_{flood.wall} := 1.4$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

$(C_D)_{pier} := 0.7$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

Wind Design Properties

$roughness :=$ ground surface roughness d *(ground surface roughness; LRFD 3.8.1.1.4)*

$wind.exposure :=$ wind exposure category d *(wind exposure category; LRFD 3.8.1.1.5)*

$V_{III} := 115\ mph$ *(design 3-second gust wind speed for Strength III; LRFD Table 3.8.1.1.2-1)*

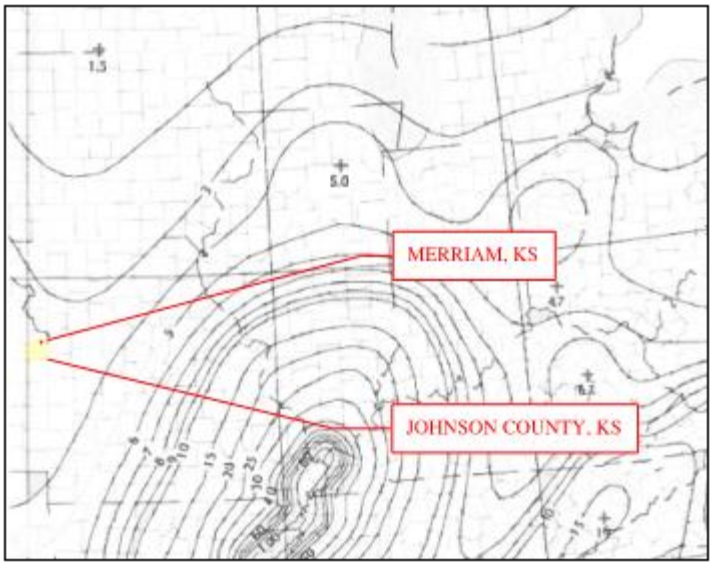
$G := 1.00$ *(gust effect factor; LRFD 3.8.1.2.1)*

$(C_D)_W := 1.3$ *(gust effect factor; LRFD Table 3.8.1.2.1-2)*

Seismic Design Properties

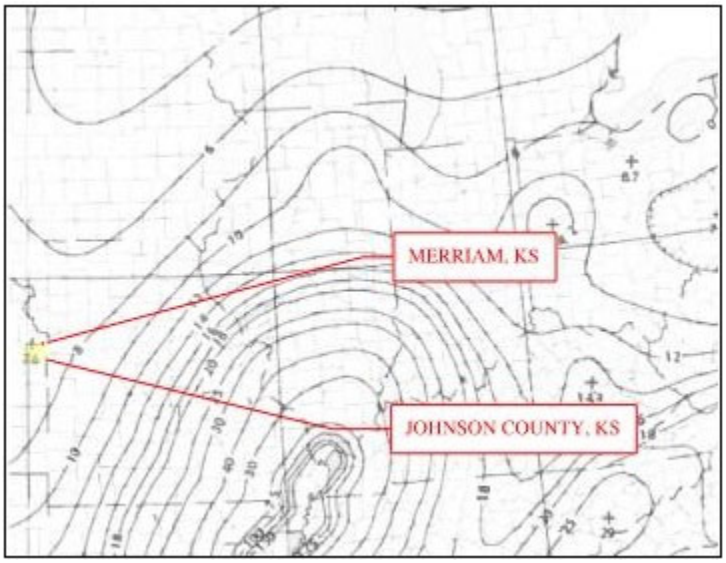
$site.class :=$ "d" *(site classification, assumed)*

LRFD Figure 3.10.2.1-1:



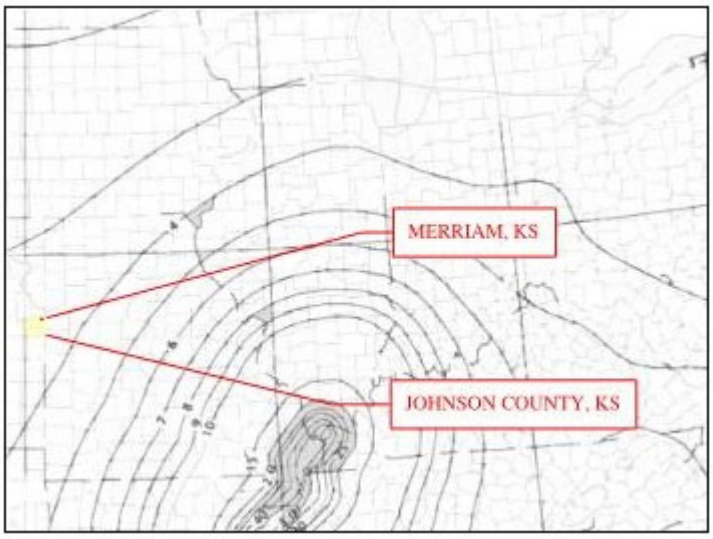
$PGA := 3.5\%$ *(peak seismic ground acceleration coefficient on rock; LRFD 9th Ed. Figure 3.10.2.1-1)*

LRFD Figure 3.10.2.1-2:



$S_S := 7.6\%$ *(horizontal response spectral acceleration coefficient at 0.2-s period on rock - site class b; LRFD 9th Ed. Figure 3.10.2.1-2)*

LRFD Figure 3.10.2.1-3:



$S_1 := 3.7\%$ *(horizontal response spectral acceleration coefficient at 1-s period on rock - site class b; LRFD 9th Ed. Figure 3.10.2.1-3)*

Load and Resistance Factors

Load Modifiers:

$\eta_D := 1.00$ *(load modifying factor for ductility; LRFD 1.3.3)*

$\eta_R := 1.00$ *(load modifying factor for redundancy; LRFD 1.3.4)*

$\eta_I := 1.00$ *(load modifying factor for operational classification; LRFD 1.3.5)*

$(\eta_{strength})_{max} := \max\left(\eta_D \cdot \eta_R \cdot \eta_I, 0.95\right) = 1$ *(total load modification for loads for which a maximum of γ is appropriate for strength limit states; LRFD Eq. 1.3.2.1-2)*

$(\eta_{strength})_{min} := min\left(\frac{1}{\eta_D \cdot \eta_R \cdot \eta_I}, 1.0\right) = 1.00$ *(total load modification for loads for which a minimum of γ is appropriate for strength limit states; LRFD Eq. 1.3.2.1-3)*

Dynamic Load Allowance & Multiple Presence Factors

$IM := 1.33$ *(dynamic load allowance; LRFD Table 3.6.2.1-1)*

$$\gamma_{ADTT} := \begin{cases} 0.9 & \text{if } ADTT \le 100 \\ 0.95 & \text{else if } 100 \le ADTT \le 1000 \\ 1.0 & \text{else} \end{cases}$$ *(load reduction for low average daily truck traffic; LRFD C3.6.1.1.2)*

$\gamma_{ADTT} = 1$

$(m_1)_o := 1.2$ *(multiple presence factor for single lane loaded; LRFD Table 3.6.1.1.2-1)*

$(m_2)_o := 1.0$ *(multiple presence factor for two lanes loaded; LRFD Table 3.6.1.1.2-1)*

$(m_3)_o := 0.85$ *(multiple presence factor for three lanes loaded; LRFD Table 3.6.1.1.2-1)*

$(m_4)_o := 0.65$ *(multiple presence factor for four or more lanes loaded; LRFD Table 3.6.1.1.2-1)*

$m_1 := \gamma_{ADTT} \cdot (m_1)_o = 1.20$ *(multiple presence factor for single lane loaded adjusted for low ADTT)*

$m_2 := \gamma_{ADTT} \cdot (m_2)_o = 1.00$ *(multiple presence factor for two lanes loaded adjusted for low ADTT)*

$m_3 := \gamma_{ADTT} \cdot (m_3)_o = 0.85$ *(multiple presence factor for three lanes loaded adjusted for low ADTT)*

$m_4 := \gamma_{ADTT} \cdot (m_4)_o = 0.65$ *(multiple presence factor for three lanes loaded adjusted for low ADTT)*

$$N_i = \begin{bmatrix} 1 \\ 2 \\ 3 \\ 4 \\ 5 \end{bmatrix} \qquad m_i = \begin{bmatrix} 1.2 \\ 1 \\ 0.85 \\ 0.65 \\ 0.65 \end{bmatrix} \qquad N_i m_i = \begin{bmatrix} 1.2 \\ 2 \\ 2.55 \\ 2.6 \\ 3.25 \end{bmatrix}$$

$g_{dist} := \max\left(N_i m_i\right) = 3.25$ *(distribution of traffic loads)*

DIMENSIONS

General Dimensions

General Bridge Dimensions:

$L_b := 160\ ft$ *(length of bridge)*

$L_1 := \frac{L_b}{3} = 53.3\ ft$ *(length of span 1)*

$L_2 := \frac{L_b}{3} = 53.3\ ft$ *(length of span 2)*

$L_3 := \frac{L_b}{3} = 53.3\ ft$ *(length of span 3)*

Roadway Dimensions:

$d_e := 2\ ft$ *(distance from the exterior girder to the interior edge of curb)*

$x_{WL} := 2\ ft$ *(distance from first wheel line to curb; LRFD 3.6.1.3.1)*

Deck Dimensions:

$b_{slab} := 76\ ft$ *(width of bridge deck slab)*

$(t_{slab})_{min} := 1\ ft + 6\ in$ *(minimum thickness of slab)*

$(t_{slab})_{max} := 2\ ft + 0\ in$ *(maximum thickness of slab)*

$t_{slab} := \frac{(t_{slab})_{min} + (t_{slab})_{max}}{2} = 21\ in$ *(average thickness of slab)*

Component Dimensions

ABUTMENT/BENT LOCATIONS

location	type	x (ft)	N_p	t_s (in)
abutment 1	drilled pier	-	5	18
bent 1	drilled pier	53.33	5	18
bent 2	drilled pier	106.67	5	18
abutment 2	drilled pier	160.00	5	18

$x := \text{submatrix}\left(output, 1, n_i, n_{id}+1, n_{id}+1\right) \cdot ft$ *(distance from left abutment bearing to each pier/abutments)*

$N_p := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right)$ *(number of piles/caissons at each interior pier/abutments)*

$t_s := \text{submatrix}\left(output, 1, n_i, n_{id}+3, n_{id}+3\right) \cdot in$ *(thickness of slab at each interior pier/abutments)*

$(b_t)_{a1} := \frac{L_1}{2} = 26.7\ ft$ *(width of deck tributary to abutment 1)*

$(b_t)_{b1} := \frac{L_1}{2} + \frac{L_2}{2} = 53.3\ ft$ *(width of deck tributary to bent 1)*

$(b_t)_{b2} := \frac{L_2}{2} + \frac{L_3}{2} = 53.3\ ft$ *(width of deck tributary to bent 2)*

$(b_t)_{a1} := \frac{L_1}{2} = 26.7\ ft$ *(width of deck tributary to abutment 1)*

$(b_t)_i := \left[(b_t)_{a1} \quad (b_t)_{b1} \quad (b_t)_{b2} \quad (b_t)_{a1} \right]^T$ *(summary)*

Safety Barrier:

$(h_{rail})_{head} := 1\ ft + 2\ in$ *(height of kdot coral rail head)*

$(b_{rail})_{head} := 1\ ft + 0\ in$ *(width of kdot coral rail head)*

$(h_{rail})_{curb} := 0\ ft + 6\ in$ *(height of kdot coral rail curb)*

$(b_{rail})_{curb} := 1\ ft + 0\ in$ *(width of kdot coral rail curb)*

$(h_{rail})_{post} := 0\ ft + 7\ in$ *(height of kdot coral rail post)*

$(b_{rail})_{post} := 0\ ft + 10\ in$ *(width of kdot coral rail post)*

$(d_{rail})_{post} := 3\ ft + 0\ in$ *(width of kdot coral rail post)*

$(s_{rail})_{post} := 10\ ft + 0\ in$ *(spacing of kdot coral rail post center-to-center)*

Flood Wall Dimensions:

$L_{wall} := L_b = 160\ ft$ *(length of flood wall)*

$b_{wall} := 12\ in$ *(width of flood wall)*

$h_{wall} := 60\ in$ *(height of flood wall)*

Sidewalk:

$t_{sw} := 0\ in$ *(thickness of sidewalk)*

$b_{sw} := 0\ in$ *(width of sidewalk)*

Piers and Bent Cap:

PIER AND PIER CAP DIMENSIONS						
location	type	N_{shaft}	d_{shaft} (in)	L_{shaft} (ft)	b_{cap} (in)	h_{cap} (in)
abutment 1	drilled pier	5	36	29	36	66
bent 1	drilled pier	5	36	29	36	66
bent 2	drilled pier	5	36	29	36	66
abutment 2	drilled pier	5	36	29	36	66

$N_{shaft} := \text{submatrix}\left(output, 1, n_i, n_{id}+1, n_{id}+1\right)$ *(number caissons at intermediate piers/abutments)*

$d_{shaft} := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right) \cdot in$ *(diameter of caissons)*

$L_{shaft} := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right) \cdot in$ *(length of caissons)*

$b_{cap} := \text{submatrix}\left(output, 1, n_i, n_{id}+4, n_{id}+4\right) \cdot in$ *(width of bent caps)*

$h_{cap} := \text{submatrix}\left(output, 1, n_i, n_{id}+5, n_{id}+5\right) \cdot in$ *(gross height of bent caps including region integral with slab)*

$h'_{cap} := h_{cap} - t_s$ *(net height of bent caps excluding region integral with slab)*

$L_{cap} := b_{slab} - 1\ ft = 75\ ft$ *(length of pier caps)*

$A_{cap} := \overrightarrow{b_{cap} \cdot h'_{cap}}$ *(cross-sectional area of bent cap)*

$A_{shaft} := \dfrac{\pi \cdot d_{shaft}^{\ 2}}{4}$ *(cross-sectional area of shafts)*

CROSS-SECTIONAL AREAS DRILLED PIERS & BENT CAPS			
location	type	A_{cap} (ft^2)	A_{cap} (ft^2)
abutment 1	drilled pier	12.00	7.07
bent 1	drilled pier	12.00	7.07
bent 2	drilled pier	12.00	7.07
abutment 2	drilled pier	12.00	7.07

Elevations

$(EL_{TOD})_{A1} := 914\ ft$ *(top of deck at abutment 1)*

$(EL_{TOD})_{A2} := 920\ ft$ *(top of deck at abutment 2)*

$EL_{base} := 895\ ft$ *(elevation at bottom of footing/pile cap)*

$EL_{USR} := (EL_{TOD})_{A1} - t_{slab} = 912.3\ ft$ *(bottom of slab elevation, upstream right)*

$EL_{USL} := (EL_{TOD})_{A2} - t_{slab} = 918.3\ ft$ *(bottom of slab elevation, upstream left)*

$EL_{DSR} := (EL_{TOD})_{A1} - t_{slab} = 912.3\ ft$ *(bottom of slab elevation, downstream right)*

$EL_{DSL} := (EL_{TOD})_{A2} - t_{slab} = 918.3\ ft$ *(bottom of slab elevation, downstream left)*

$WSE_{USR} := 918.43\ ft$ *(water surface elevation, upstream right)*

$WSE_{USL} := 918.43\ ft$ *(water surface elevation, upstream left)*

$WSE_{DSR} := 917.63\ ft$ *(water surface elevation, downstream right)*

$WSE_{DSL} := 917.63\ ft$ *(water surface elevation, downstream left)*

$$EL_{US} := \frac{EL_{USR} + EL_{USL}}{2} = 915.3\ ft$$ *(average elevation at bottom of slab, upstream)*

$$WSE_{US} := \frac{WSE_{USR} + WSE_{USR}}{2} = 918.4\ ft$$ *(average water surface, upstream)*

$$EL_{DS} := \frac{EL_{DSR} + EL_{DSL}}{2} = 915.3\ ft$$ *(average elevation at bottom of slab, downstream)*

$$WSE_{DS} := \frac{WSE_{DSR} + WSE_{DSR}}{2} = 917.6\ ft$$ *(average water surface, downstream)*

Dimensions for Wind Analysis:

$L_{wind} := L_{wall} = 160\ ft$ *(length of exposed structure for wind loads; distance between joints)*

$h_{wind} := h_{wall} = 5\ ft$ *(height of exposed structure for wind loads)*

$EL_{base} = 895\ ft$ *(elevation of ground or water surface)*

$EL_{structure} := (EL_{TOD})_{A2} + h_{wall} = 925\ ft$ *(elevation of structure)*

$z := EL_{structure} - EL_{base} = 30\ ft$ *(measured structure height above ground or water surface)*

Concrete Component Volumes

$v_{slab} := b_{slab} \cdot t_{slab} = 133\ ft^3/ft$ *(unit volume of concrete slab)*

$v_{sw} := b_{sw} \cdot t_{sw} = 0\ ft^3/ft$ *(unit volume of concrete sidewalk)*

$v_{wall} := b_{wall} \cdot h_{wall} = 5\ ft^3/ft$ *(unit volume of floodwall)*

$v_{rail} := (h_{rail})_{head} \cdot (b_{rail})_{head} + \frac{(h_{rail})_{post} \cdot (b_{rail})_{post} \cdot (d_{rail})_{post}}{(s_{rail})_{post}} = 1.3\ ft^3/ft$ *(unit volume of kdot corral barrier)*

$v_{total} := v_{slab} + 2 \cdot \left(v_{sw} + v_{rail} + v_{wall}\right) = 145.6\ ft^3/ft$ *(total unit volume of concrete in bridge cross-section)*

$v_{cap} := \overrightarrow{A_{cap} \cdot L_{cap}}$ *(volume of pier caps at abutments/bents)*

$v_{shaft} := \overrightarrow{A_{shaft} \cdot L_{shaft}}$ *(volume of single shaft)*

$V_{shaft} := \overrightarrow{N_{shaft} \cdot v_{shaft}}$ *(total volume of shafts at abutments/bents)*

VOLUME OF CONCRETE IN PIERS & BENT CAPS

location	type	v_{cap} (ft^3)	v_{shaft} (ft^3)	V_{shaft} (ft^3)
abutment 1	drilled pier	900.00	21.21	106.03
bent 1	drilled pier	900.00	21.21	106.03
bent 2	drilled pier	900.00	21.21	106.03
abutment 2	drilled pier	900.00	21.21	106.03

Component Weights

$w_{slab} := v_{slab} \cdot \gamma_c = 19950\ plf$ *(unit weight of concrete slab)*

$w_{sw} := v_{sw} \cdot \gamma_c = 0\ plf$ *(unit weight of concrete sidewalk)*

$w_{wall} := v_{wall} \cdot \gamma_c = 750\ plf$ *(unit weight of concrete wall)*

$w_{rail} := v_{rail} \cdot \gamma_c = 196.9\ plf$ *(unit weight of kdot corral barrier)*

$2 \cdot \left(w_{sw} + w_{wall} + w_{rail}\right) = 1.894\ klf$ *(unit weight of dead load on deck)*

$w_{deck} := \dfrac{\overrightarrow{\left(w_{slab} + 2 \cdot \left(w_{sw} + w_{wall} + w_{rail}\right)\right) \cdot (b_t)_i}}{L_{cap}}$ *(total continuous dead load supported by bents)*

WEIGHT SUPPORTED BY ABUTMENTS/BENTS		
location	type	w_{deck} (klf)
abutment 1	drilled pier	7.77
bent 1	drilled pier	15.53
bent 2	drilled pier	15.53
abutment 2	drilled pier	7.77

$w_{shaft} := v_{shaft} \cdot \gamma_c$ *(weight of single shaft at supports)*

$(w_{DC})_{shaft} := \overrightarrow{N_{shaft} \cdot w_{shaft}}$ *(total weight of shafts at supports)*

WEIGHT OF DRILLED PIERS			
location	type	w_{shaft} (kip)	$(w_{DC})_{shaft}$ (kip)
abutment 1	drilled pier	0.97	15.90
bent 1	drilled pier	0.97	15.90
bent 2	drilled pier	0.97	15.90
abutment 2	drilled pier	0.97	15.90

$(w_{DC})_{deck} := v_{total} \cdot \gamma_c = 21.844\ klf$ *(total unit weight of dead load)*

$(w_{DC})_{slab} := \dfrac{v_{total} \cdot \gamma_c}{b_{slab}} = 287\ \dfrac{plf}{ft}$ *(unit weight of dead load per unit width of slab)*

$W_{cap} := v_{cap} \cdot \gamma_c$ *(weight of pier caps at supports)*

WEIGHT OF BENT CAPS		
location	type	W_{cap} (kip)
abutment 1	drilled pier	135.00
bent 1	drilled pier	135.00
bent 2	drilled pier	135.00
abutment 2	drilled pier	135.00

$(W_{DC})_{shaft} := \sum (w_{DC})_{shaft} = 63.617\ kip$ *(total weight of piers)*

$(W_{DC})_{cap} := \sum W_{cap} = 540\ kip$ *(total weight of pier caps)*

$(W_{DC})_{deck} := (w_{DC})_{deck} \cdot L_b + (W_{DC})_{cap} = 4035\ kip$ *(total weight of dead load at deck)*

$W_{DC} := (W_{DC})_{deck} + (W_{DC})_{shaft} = 4098.6\ kip$ *(total weight of dead load)*

$w_{DW} := 0\ plf$ *(continuous dead load, DW)*

Hydraulic Loads

Uplift:

$d_{US} := WSE_{US} - EL_{US} = 3.18\ ft$ *(depth of water at upstream side of bridge)*

$d_{DS} := WSE_{DS} - EL_{DS} = 2.38\ ft$ *(depth of water at downstream side of bridge)*

$d_{displaced} := \frac{d_{US} + d_{US}}{2} = 3.2\ ft$ *(depth of water displaced by bridge)*

$V_{displaced} := d_{displaced} \cdot b_{slab} \cdot L_b = 38668.8\ ft^3$ *(volume of displaced water)*

$F_{buoyant} := \gamma_w \cdot V_{displaced} = 2412.9\ kip$ *(buoyant force)*

$(FS_{uplift})_{deck} := \frac{(W_{DC})_{deck}}{F_{buoyant}} = 1.7$ *(uplift factor of safety, deck and pier caps only)*

$FS_{uplift} := \frac{W_{DC}}{F_{buoyant}} = 1.7$ *(uplift factor of safety, total)*

Drag:

$C_D := (C_D)_{flood.wall} = 1.4$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

$p_{wall} := \frac{C_D \cdot \gamma_w \cdot {V_{US}}^2}{2 \cdot g} = 105.1\ psf$ *(pressure of flowing water on floodwall; LRFD Eq. C3.7.3.1-1)*

$C_D := (C_D)_{pier} = 0.7$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

$p_{pier} := \frac{C_D \cdot \gamma_w \cdot {V_{US}}^2}{2 \cdot g} = 52.6\ psf$ *(pressure of flowing water on bridge caisson; LRFD Eq. C3.7.3.1-1)*

$(w_{WA})_{deck} := p_{wall} \cdot h_{wall} = 0.53\ klf$ *(continuous load on bridge at deck)*

$(P_{WA})_i := \overrightarrow{(w_{WA})_{deck} \cdot (b_t)_i}$ *(total water load on abutments & piers)*

WATER LOAD ON ABUTMENTS/BETS

location	type	P_{WA} (kip)
abutment 1	drilled pier	14.02
bent 1	drilled pier	28.04
bent 2	drilled pier	28.04
abutment 2	drilled pier	14.02

Braking Load

$L_{brake} := L_b = 160\ ft$ *(length of lane load braking distance)*

$W_{TLL} := 8\ kip + 32\ kip + 32\ kip = 72\ kip$ *(weight of design truck)*

$W_{LLL} := (0.64\ klf) \cdot L_{brake} = 102.4\ kip$ *(total weight of lane load)*

$n_L = 3$ *(number lanes with traffic in the same direction)*

$F_{BR} := n_L \cdot (\max(25\% \cdot W_{TLL}, 5\% \cdot (W_{TLL} + W_{LLL}))) = 54.0\ kip$ *(total braking load; LRFD 3.6.4)*

Bearing Reactions Due to Dead & Live Loads

Dead Loads:

DEAD LOADS			
location	type	R_{DC} (kip)	R_{DW} (kip)
abutment 1	drilled pier	466.15	-
bent 1	drilled pier	1281.37	-
bent 2	drilled pier	1281.37	-
abutment 2	drilled pier	466.15	-

$R_{DC} := \text{submatrix}(output, 1, n_i, n_{id}+1, n_{id}+1) \cdot kip$ *(reaction due to design truck/tandem)*

$R_{DW} := \text{submatrix}(output, 1, n_i, n_{id}+2, n_{id}+2) \cdot kip$ *(reaction due to design lane load)*

Live Loads:

VEHICULAR LIVE LOADS AT ABUMENTS & PIERS			
location	type	R_{TTL} (kip)	R_{LLL} (kip)
abutment 1	drilled pier	56.52	15.36
bent 1	drilled pier	68.86	40.93
bent 2	drilled pier	68.86	40.93
abutment 2	drilled pier	56.52	15.36

$R_{TLL} := \text{submatrix}(output, 1, n_i, n_{id}+1, n_{id}+1) \cdot kip$ *(reaction due to design truck/tandem)*

$R_{LLL} := \text{submatrix}(output, 1, n_i, n_{id}+2, n_{id}+2) \cdot kip$ *(reaction due to design lane load)*

$R_{LL} := \overrightarrow{g_{dist} \cdot (R_{TLL} \cdot IM + R_{LLL})}$ *(total reaction due to live loads)*

LIVE LOADS AT ABUMENTS & PIERS		
location	type	R_{LL} (kip)
abutment 1	drilled pier	294.23
bent 1	drilled pier	430.67
bent 2	drilled pier	430.67
abutment 2	drilled pier	294.23

Wind Loads

$roughness = \text{“d”}$ *(ground surface roughness; LRFD 3.8.1.1.4)*

$wind.exposure = \text{“d”}$ *(wind exposure category; LRFD 3.8.1.1.5)*

$V_{III} = 115\ mph$ *(design 3-second gust wind speed for Strength III; LRFD Table 3.8.1.1.2-1)*

$G = 1$ *(gust effect factor; LRFD 3.8.1.2.1)*

$C_D := (C_D)_W = 1.3$ *(gust effect factor; LRFD Table 3.8.1.2.1-2)*

$Z := \max(z, 33\ ft) = 33\ ft$ *(structure design height)*

$list.exposure$	$list.coefficient$
“b”	$\dfrac{\left(2.5 \cdot ln\left(\frac{Z \cdot ft^{-1}}{0.9834}\right) + 6.87\right)^2}{345.6}$
“c”	$\dfrac{\left(2.5 \cdot ln\left(\frac{Z \cdot ft^{-1}}{0.0984}\right) + 7.35\right)^2}{478.4}$
“d”	$\dfrac{\left(2.5 \cdot ln\left(\frac{Z \cdot ft^{-1}}{0.0164}\right) + 7.65\right)^2}{616.1}$

$table_{Kz} := \text{augment}(list.exposure, list.coefficient)$

$(K_z)_{strengthIII\&serviceIV} := \text{vlookup}(wind.exposure, table_{Kz}, 2)_1$ *(pressure exposure and elevation coefficient for Strength III and Service IV; LRFD Eqs. 3.8.1.2.1-2, 3 & 4)*

$(K_z)_{strengthIII\&serviceIV} = 1.2$

3-SECOND GUST WIND SPEED AND PRESSURE EXPOSURE & ELEVATION COEFFICIENT

load combination	V (mph)	K_z
strength III	115.00	1.15
strength V	80.00	1.00
service I	70.00	1.00
service IV	86.25	1.15

$wind := table.input$

$P_Z := \overrightarrow{\left(2.56 \cdot 10^{-6}\right) \cdot \left(V \cdot mph^{-1}\right)^2 \cdot K_z \cdot G \cdot C_D \cdot ksf}$ *(design wind pressure; LRFD Eq. 3.8.1.2.1-1)*

$(P_Z)_{III} := P_{Z_1} = 50.8\ psf$ *(design wind pressure; Strength III)*

$(P_Z)_V := P_{Z_2} = 21.3\ psf$ *(design wind pressure; Strength V)*

$(w_{WS})_z := P_Z \cdot h_{wind}$ *(wind on structure, transverse; LRFD 3.8.1.2.3a)*

$(w_{WS})_x := 25\% \cdot (w_{WS})_z$ *(wind on structure, longitudinal; LRFD 3.8.1.2.3a)*

$table.wind := \text{augment}\left(table.input^{\langle 1 \rangle}, (w_{WS})_z, (w_{WS})_x\right)$

$(w_{WS})_{x.III} := (w_{WS})_{x_1} = 0.1\ klf$ *(wind on structure, longitudinal; Strength III)*

$(w_{WS})_{x.V} := (w_{WS})_{x_2} = 0\ klf$ *(wind on structure, longitudinal; Strength V)*

$(w_{WL})_z := 0.10\ klf$ *(wind on live load, transverse; LRFD 3.8.1.3)*

$(w_{WL})_x := 0.04\ klf$ *(wind on live load, longitudinal; LRFD 3.8.1.3)*

Seismic Loads

$PGA = 3.5\%$ *(peak seismic ground acceleration coefficient on rock; LRFD Figure 3.10.2.1-1)*

$S_S = 7.6\%$ *(horizontal response spectral acceleration coefficient at 0.2-s period on rock - site class b; LRFD Figure 3.10.2.1-2)*

$S_1 = 3.7\%$ *(horizontal response spectral acceleration coefficient at 1-s period on rock - site class b; LRFD Figure 3.10.2.1-3)*

$site.class =$ "d" *(site classification)*

AASHTO LRFD Table 3.10.3.2-1 Site Factor, F_{pga}

site class	peak ground acceleration coeficient (PGA)				
	PGA < 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA > 0.50
a	0.8	0.8	0.8	0.8	0.8
b	1.0	1.0	1.0	1.0	1.0
c	1.2	1.2	1.1	1.0	1.0
d	1.6	1.4	1.2	1.1	1.0
e	2.5	1.7	1.2	0.9	0.9
f	site-specific geotechnical investigation required				

$F_{pga} = 1.60$ *(site factor; LRFD Table 3.10.3.2-1)*

$A_S := F_{pga} \cdot PGA = 0.1$

AASHTO LRFD Table 3.10.3.2-2 Site Factor, F_a

site class	spectral acceleration coefficient at period 0.2 sec (S_S)				
	$S_S < 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.0$	$S_S > 1.25$
a	0.8	0.8	0.8	0.8	0.8
b	1.0	1.0	1.0	1.0	1.0
c	1.2	1.2	1.1	1.0	1.0
d	1.6	1.4	1.2	1.1	1.0
e	2.5	1.7	1.2	0.9	0.9
f	site-specific geotechnical investigation required				

$F_a = 1.60$ *(site factor for short-period range of acceleration response spectrum; LRFD Table 3.10.3.2-2)*

AASHTO LRFD Table 3.10.3.2-3 Site Factor, F_v

site class	spectral acceleration coefficient at period 1 sec (S_1)				
	$S_1 < 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 > 0.5$
a	0.8	0.8	0.8	0.8	0.8
b	1.0	1.0	1.0	1.0	1.0
c	1.7	1.6	1.5	1.4	1.3
d	2.4	2.0	1.8	1.6	1.5
e	3.5	3.2	2.8	2.4	2.4
f	site-specific geotechnical investigation required				

$F_v = 2.40$ *(site factor for long-period range of acceleration response spectrum; LRFD Table 3.10.3.2-2)*

$S_{DS} := F_a \cdot S_S = 0.1$ *(horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor; LRFD Eq. 3.10.4.2-2)*

$S_{D1} := F_v \cdot S_1 = 0.1$ *(horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor; LRFD Eq. 3.10.4.2-6)*

AASHTO TABLE 3.10.6-1 - SEISMIC ZONES	
acceleration coefficient, S_{D1}	seismic zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

$seismic.zone = 1$

$$\gamma_{EQ} := \begin{cases} \text{if } A_S < 0.05 \\ \quad 0.15 \\ \text{else} \\ \quad 0.25 \end{cases}$$ *(LRFD 3.10.9.2)*

$\gamma_{EQ} = 0.3$

$w_{EQ} := \overrightarrow{\gamma_{EQ} \cdot \left((w_{DC})_{deck} + w_{DW} \right)} = 5.46 \ klf$

Design Loads

Vertical Loads:

$P_{DC} := R_{DC}$ *(design dead load, DC)*

$P_{DW} := R_{DW}$ *(design dead load, DW)*

$P_{LL} := R_{LL}$ *(design live load)*

Lateral Loads in Transverse Direction:

$(w_{EQ})_z := w_{EQ} = 5.5\ klf$ *(seismic load, transverse direction)*

$(w_{WA})_z := (w_{WA})_{deck} = 0.526\ klf$ *(water load, transverse direction)*

Lateral Loads in Longitudinal Direction:

$(w_{EQ})_x := w_{EQ} = 5.5\ klf$ *(seismic load, longitudinal direction)*

$(w_{BR})_x := \frac{F_{BR}}{L_b} = 0.3\ klf$ *(braking load)*

LOAD COMBINATION FACTORS										
limit state	type	η	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	strength	1.00	1.25	1.50	1.75	1.75	1.00	-	-	-
strength III	strength	1.00	1.25	1.50	-	-	1.00	1.00	-	-
strength V	strength	1.00	1.25	1.50	1.35	1.35	1.00	1.00	1.00	-
extreme event I	extreme event	1.00	1.25	1.50	1.00	1.00	1.00	-	-	1.00
service I	service	1.00	1.25	1.50	1.00	1.00	1.00	1.00	1.00	-

$limit.state := \text{submatrix}\left(output, 1, n_i, 1, 1\right)$

$limit.state.type := \text{submatrix}\left(output, 1, n_i, 2, 2\right)$

$\eta := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right)$

$\gamma := \text{submatrix}\left(output, 1, n_i, n_{id}+3, n_{oj}+1\right)$

$Q_{DC} := /f/conform\left(P_{DC}, location\right)$

$Q_{DW} := /f/conform\left(P_{DW}, location\right)$

$Q_{LL} := /f/conform\left(P_{LL}, location\right)$

$Q_{BR} := /f/conform\left(n/a, location\right)$

$Q_{WA} := /f/conform\left(n/a, location\right)$

$Q_{WS} := /f/conform\left(n/a, location\right)$

$Q_{WL} := /f/conform\left(n/a, location\right)$

$Q_{EQ} := /f/conform\left(n/a, location\right)$

$gravity := \left[Q_{DC}\ \ Q_{DW}\ \ Q_{LL}\ \ Q_{BR}\ \ Q_{WA}\ \ Q_{WS}\ \ Q_{WL}\ \ Q_{EQ}\right]^{T}$

$$Q := \left\| \begin{array}{l} m_0 \leftarrow gravity \\ \text{for } k \in 1 .. n_k \\ \quad \text{for } i \in 1 .. \text{rows}(\eta) \\ \quad\quad \text{for } j \in 1 .. \text{cols}(\gamma) \\ \quad\quad\quad m_{i,j} \leftarrow \left(m_{0_j}\right)_k \\ \quad\quad\quad Q_k \leftarrow m \\ Q \end{array} \right.$$

$$\eta\gamma P := \left\| \begin{array}{l} \text{for } k \in 1 .. n_k \\ \quad \text{for } i \in 1 .. n_i \\ \quad\quad \text{for } j \in 1 .. n_j \\ \quad\quad\quad m_{i,j} \leftarrow \eta_i \cdot \gamma_{i,j} \cdot \left(Q_k\right)_{i,j} \\ \quad\quad\quad \eta\gamma P_k \leftarrow m \\ \eta\gamma P \end{array} \right.$$

$$\Sigma\eta\gamma P := \left\| \begin{array}{l} \text{for } k \in 1 .. n_k \\ \quad \text{for } i \in 1 .. n_i \\ \quad\quad \text{for } j \in 1 .. n_j \\ \quad\quad\quad m_i \leftarrow \sum_{j=1}^{j} \left(\eta\gamma P_k\right)_{i,j} \\ \quad\quad\quad \Sigma\eta\gamma P_k \leftarrow m \\ \Sigma\eta\gamma P \end{array} \right.$$

$$/f/controlling\left(v_0, type\right) := \left\| \begin{array}{l} \text{for } i \in 1 .. \text{rows}\left(v_0\right) \\ \quad v_i \leftarrow \left\| \begin{array}{l} \text{if } limit.state.type_i = type \\ \quad\quad v_{0_i} \\ \text{else} \\ \quad\quad 0 \end{array} \right. \\ \max(v) \end{array} \right.$$

FACTORED GRAVITY LOADS - ABUTMENT 1									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	1097.59	582.69	-	514.90	-	-	-	-	-
strength III	582.69	582.69	-	-	-	-	-	-	-
strength V	979.90	582.69	-	397.21	-	-	-	-	-
extreme event I	876.92	582.69	-	294.23	-	-	-	-	-
service I	876.92	582.69	-	294.23	-	-	-	-	-

$(P_{strength})_{total.1} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 1097.6\ kip$ *(controlling total factored strength gravity loads at abutment 1)*

$(P_{serv})_{total.1} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 876.9\ kip$ *(controlling total factored service gravity loads at abutment 1)*

FACTORED GRAVITY LOADS - PIER 1									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	2355.39	1601.72	-	753.67	-	-	-	-	-
strength III	1601.72	1601.72	-	-	-	-	-	-	-
strength V	2183.12	1601.72	-	581.40	-	-	-	-	-
extreme event I	2032.39	1601.72	-	430.67	-	-	-	-	-
service I	2032.39	1601.72	-	430.67	-	-	-	-	-

$(P_{strength})_{total.2} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 2355.4\ kip$ *(controlling total factored strength gravity loads at bent 1)*

$(P_{serv})_{total.2} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 2032.4\ kip$ *(controlling total factored service gravity loads at bent 1)*

FACTORED GRAVITY LOADS - PIER 2									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	2355.39	1601.72	-	753.67	-	-	-	-	-
strength III	1601.72	1601.72	-	-	-	-	-	-	-
strength V	2183.12	1601.72	-	581.40	-	-	-	-	-
extreme event I	2032.39	1601.72	-	430.67	-	-	-	-	-
service I	2032.39	1601.72	-	430.67	-	-	-	-	-

$(P_{strength})_{total.3} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 2355.4\ kip$ *(controlling total factored strength gravity loads at bent 2)*

$(P_{serv})_{total.3} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 2032.4\ kip$ *(controlling total factored service gravity load at bent 2)*

FACTORED GRAVITY LOADS - ABUTMENT 2									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	1097.59	582.69	-	514.90	-	-	-	-	-
strength III	582.69	582.69	-	-	-	-	-	-	-
strength V	979.89	582.69	-	397.21	-	-	-	-	-
extreme event I	876.92	582.69	-	294.23	-	-	-	-	-
service I	876.92	582.69	-	294.23	-	-	-	-	-

$(P_{strength})_{total.4} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 1097.6\ kip$ *(controlling total factored strength gravity loads at abutment 2)*

$(P_{serv})_{total.4} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 876.9\ kip$ *(controlling total factored service gravity load at abutment 2)*

$(P_{strength})_{total} := \left[(P_{strength})_{total.1} \quad (P_{strength})_{total.2} \quad (P_{strength})_{total.3} \quad (P_{strength})_{total.4} \right]^{T}$

(summary of controlling total factored strength gravity loads)

$P_{strength} := \frac{(P_{strength})_{total}}{N_{shaft}}$ *(factored service gravity loads on each caisson)*

$(P_{serv})_{total} := \left[(P_{serv})_{total.1} \quad (P_{serv})_{total.2} \quad (P_{serv})_{total.3} \quad (P_{serv})_{total.4} \right]^{T}$ *(summary of controlling total factored service gravity loads)*

$P_{serv} := \frac{(P_{serv})_{total}}{N_{shaft}}$ *(factored service gravity loads on each caisson)*

CONTROLLING GRAVITY LOADS AT ABUMENTS & PIERS

location	type	$(P_{serv})_{total}$ (kip)	P_{serv} (kip)	$(P_{strength})_{total}$ (kip)	$P_{strength}$ (kip)
abutment 1	drilled pier	876.9	175.4	1097.6	219.5
bent 1	drilled pier	2032.4	406.5	2355.4	471.1
bent 2	drilled pier	2032.4	406.5	2355.4	471.1
abutment 2	drilled pier	876.9	175.4	1097.6	219.5

$Q_{DC} := /f/conform(n/a, limit.state)$

$Q_{DW} := /f/conform(n/a, limit.state)$

$Q_{LL} := /f/conform(n/a, limit.state)$

$Q_{BR} := /f/conform(n/a, limit.state)$

$Q_{WA} := /f/conform((w_{WA})_z, limit.state)$

$$Q_{WS} := \left\| \begin{array}{l} \text{for } i \in 1..n_i \\ \quad w_{WS_i} \leftarrow \left\| \begin{array}{l} \text{try} \\ \quad \text{vlookup}\left(limit.state_i, table.wind, 2\right)_1 \\ \text{on error} \\ \quad n/a \end{array} \right. \\ w_{WS} \end{array} \right.$$

$Q_{WL} := /f/conform((w_{WL})_z, limit.state)$

$Q_{EQ} := /f/conform((w_{EQ})_z, limit.state)$

$Q := \text{augment}(Q_{DC}, Q_{DW}, Q_{LL}, Q_{BR}, Q_{WA}, Q_{WS}, Q_{WL}, Q_{EQ})$

$$(\eta\gamma R)_z := \left\| \begin{array}{l} \text{for } i \in 1..n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1..n_j \\ \quad \left\| \eta\gamma R_{z_{i,j}} \leftarrow \eta_i \cdot \gamma_{i,j} \cdot Q_{i,j} \right\| \end{array} \right\| \\ \eta\gamma R_z \end{array} \right\|$$

$$\Sigma\eta\gamma R_z := \left\| \begin{array}{l} \text{for } i \in 1..n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1..n_j \\ \quad \left\| \Sigma\eta\gamma R_{z_i} \leftarrow \sum_{j=1}^{j} (\eta\gamma R)_{z_{i,j}} \right\| \end{array} \right\| \\ \Sigma\eta\gamma R_z \end{array} \right\|$$

FACTORED LATERAL LOADS: TRANSVERSE DIRECTION (KLF)									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	0.526	-	-	-	-	0.526	-	-	-
strength III	0.780	-	-	-	-	0.526	0.254	-	-
strength V	0.732	-	-	-	-	0.526	0.106	0.100	-
extreme event I	5.987	-	-	-	-	0.526	-	-	5.461
service I	0.707	-	-	-	-	0.526	0.082	0.100	-

$(R_{strength})_z := /f/controlling\left(\Sigma\eta\gamma R_z, \text{“strength”}\right) = 0.8\ klf$ *(controlling total factored strength transverse lateral load)*

$(R_{extreme})_z := /f/controlling\left(\Sigma\eta\gamma R_z, \text{“extreme event”}\right) = 6\ klf$ *(controlling total factored extreme event transverse lateral load)*

$(R_{serv})_z := /f/controlling\left(\Sigma\eta\gamma R_z, \text{“service”}\right) = 0.7\ klf$ *(controlling total factored service transverse lateral load)*

$(R_u)_z := \max\left((R_{strength})_z, (R_{extreme})_z\right) = 6\ klf$ *(transverse lateral design load)*

$(V_u)_{z.total} := (R_u)_z \cdot (b_t)_i$ *(transverse lateral design lateral loads at each support)*

$(V_u)_z := \dfrac{(V_u)_{z.total}}{N_{shaft}}$ *(transverse lateral design lateral load at each caisson)*

TRANSVERSE DESIGN LATERAL LOADS AT ABUMENTS & PIERS			
location	type	$(V_u)_{z.total}$ (kip)	$(V_u)_z$ (kip)
abutment 1	drilled pier	159.6	31.9
bent 1	drilled pier	319.3	63.9
bent 2	drilled pier	319.3	63.9
abutment 2	drilled pier	159.6	31.9

$Q_{WA} := /f/conform\left(n/a, limit.state\right)$

$$Q_{WS} := \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} w_{WS_i} \leftarrow \left\| \begin{array}{l} \text{try} \\ \quad \text{vlookup}\left(limit.state_i, table.wind, 3\right)_1 \\ \text{on error} \\ \quad n/a \end{array} \right. \end{array} \right. \\ w_{WS} \end{array} \right.$$

$Q_{WL} := /f/conform\left((w_{WL})_x, limit.state\right)$

$Q_{EQ} := /f/conform\left((w_{EQ})_x, limit.state\right)$

$Q := \text{augment}\left(Q_{DC}, Q_{DW}, Q_{LL}, Q_{BR}, Q_{WA}, Q_{WS}, Q_{WL}, Q_{EQ}\right)$

$$(\eta\gamma R)_x := \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. n_j \\ \quad \left\| \eta\gamma R_{x_{i,j}} \leftarrow \eta_i \cdot \gamma_{i,j} \cdot Q_{i,j} \right. \end{array} \right. \\ \eta\gamma R_x \end{array} \right.$$

$$\Sigma\eta\gamma R_x := \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. n_j \\ \quad \left\| \Sigma\eta\gamma R_{x_i} \leftarrow \sum_{j=1}^{j} (\eta\gamma R)_{x_{i,j}} \right. \end{array} \right. \\ \Sigma\eta\gamma R_x \end{array} \right.$$

FACTORED LATERAL LOADS: LONGITUDINAL TRANSVERSE DIRECTION (KLF)

limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	-	-	-	-	-	-	-	-	-
strength III	0.06	-	-	-	-	-	0.064	-	-
strength V	0.07	-	-	-	-	-	0.027	0.040	-
extreme event I	5.46	-	-	-	-	-	-	-	5.46
service I	0.06	-	-	-	-	-	0.020	0.040	-

$(R_{strength})_x := /f/controlling\left(\Sigma\eta\gamma R_x, \text{“strength”}\right) = 0.1\ klf$ *(controlling total factored strength transverse lateral load)*

$(R_{extreme})_x := /f/controlling\left(\Sigma\eta\gamma R_x, \text{“extreme event”}\right) = 5.5\ klf$ *(controlling total factored extreme event transverse lateral load)*

$(R_{serv})_x := /f/controlling\left(\Sigma\eta\gamma R_x, \text{“service”}\right) = 0.1\ klf$ *(controlling total factored service transverse lateral load)*

$(R_u)_x := \max\left((R_{strength})_x, (R_{extreme})_x\right) = 5.5\ klf$ *(transverse lateral design load)*

$(V_u)_{x.total} := (R_u)_x \cdot (b_t)_i$ *(transverse lateral design lateral loads at each support)*

$(V_u)_x := \frac{(V_u)_{x.total}}{N_{shaft}}$ *(transverse lateral design lateral load at each caisson)*

LONGITUDINAL DESIGN LATERAL LOADS AT ABUMENTS & PIERS			
location	type	$(V_u)_{x.total}$ (kip)	$(V_u)_x$ (kip)
abutment 1	drilled pier	145.6	29.1
bent 1	drilled pier	291.3	58.3
bent 2	drilled pier	291.3	58.3
abutment 2	drilled pier	145.6	29.1

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Johnson Drive Bridge Design

UPPER TURKEY CREEK

MERRIAM, KS

GENERAL

National Bridge Inventory (NBI) Data

condition.rating := "9 – excellent condition" *(condition rating of superstructure; SI&A Data, NBI Item 59)*

Deck Information

concrete := "normal weight" *(type of concrete weight)*

type := "grade 60 – 60 ksi" *(type of non-prestressed reinforcing steel)*

concrete.strength := "5.0 ksi" *(compressive strength of concrete)*

wearing.surface := "bituminous" *(type of wearing surface material)*

Material Properties

Reinforcement Strengths:

MATERIAL STRENGTH OF REINFORCING STEEL		
type of reinforcing steel	f_y (ksi)	f_u (ksi)
structural grade - 33 ksi	33	55
intermediate grade - 40 ksi	40	70
rail/hard grade - 50 ksi	50	80
grade 60 - 60 ksi	***60***	***90***
not provided - constructed prior to 1954	33	55
not provided - constructed during or after 1954	40	70

$f_y = 60\ ksi$ *(yield strength of reinforcing steel; MBE Table 6A.5.2.2-1)*

$f_u = 90\ ksi$ *(ultimate strength of reinforcing steel; CRSI)*

Concrete Strengths:

$f'_c = 5\ ksi$ *(compressive strength of concrete; MBE Table 6A.5.2.1-1)*

f_{ct} := "not specified" *(splitting tensile strength; enter "not specified" if not available)*

Unit Weight of Materials:

$$(\gamma_c)_{nw} := \begin{cases} \text{if } f'_c \le 5.0\ ksi \\ \quad 145\ pcf \\ \text{else if } 5.0\ ksi < f'_c \le 15.0\ ksi \\ \quad 140\ pcf + (0.001\ kcf) \cdot \left(f'_c \cdot ksi^{-1}\right) \end{cases}$$

(unit weight of normal weight concrete; LRFD Table 3.5.1-1)

$(\gamma_c)_{nw} = 145\ pcf$

UNIT WEIGHT OF CONCRETE	
type	$(\gamma_c)_y$ (pcf)
light weight	110
sand-light weight	120
normal weight	***145***

$(\gamma_c)_p = 145\ pcf$ *(unit weight of plain concrete; LRFD Table 3.5.1-1)*

$\gamma_c := (\gamma_c)_p + 5\ pcf = 150\ pcf$ *(unit weight of reinforced concrete; LRFD Table 3.5.1-1 & C3.5.1)*

UNIT WEIGHT OF WEARING SURFACE	
type	γ_{ws} (pcf)
bituminous	***140***
concrete	145
timber - hard wood	60
timber - soft wood	50
gravel	140
none	-

$\gamma_{ws} = 140\ pcf$ *(unit weight of wearing surface; LRFD Table 3.5.1-1)*

Modulus of Elasticity:

$E_s := 29000\ ksi$ *(modulus of elasticity of steel; LRFD 5.4.3.2)*

DESIGN FACTORS

Load and Resistance Factors

Strength I Load Factors:

$\gamma_{DC} := 1.25$ *(dead load factor, DC for strength I; MBE Table 6A.4.2.2-1 & LRFR Table 3.4.1-2)*

$\gamma_{DW} := 1.50$ *(maximum dead load factor, DW for strength I; MBE Table 6A.4.2.2-1 & LRFR Table 3.4.1-2)*

$(\gamma_L)_{inv} := 1.75$ *(live load factor for strength I, inventory limit state; MBE Table 6A.4.2.2-1)*

$(\gamma_L)_{op} := 1.35$ *(live load factor for strength I, operating limit state; MBE Table 6A.4.2.2-1)*

Resistance Factors:

$(\varphi_f)_t := 0.90$ *(flexural resistance factor for tension controlled sections; LRFD 5.5.4.2)*

$(\varphi_f)_c := 0.75$ *(flexural resistance factor for compression controlled sections; LRFD 5.5.4.2)*

$\varphi_c :=$ *(condition factor; MBE Table 6A.4.2.3-1)*

```
if condition.rating ≥ 6
    1.0
else if condition.rating = 5
    0.95
else
    0.85
```

$\varphi_c = 1.00$

$\varphi_s := 1.00$ *(system factor, positive flexure; TCX Guidance)*

Other Factors

$IM := 1.33$ *(impact factor; MBE 6A.4.4.3)*

$\lambda :=$ *(concrete density modification factor; MBE 6A.4.4.3)*

```
if f_ct = "not specified"
    max(0.75, min(7.5 · (γ_c)_p · kcf^-1, 1.0))
else
    min(4.7 · f_ct / √f'_c, 1.0)
```

$\lambda = 1.0$

Member Dimensions

$L := \frac{160\ ft}{3} = 53.3\ ft$ *(length of span)*

$W := 76\ ft$ *(edge-to-edge width)*

$S := \frac{160\ ft}{3}$ *(spacing of bent caps)*

$b_{clr} := 76\ ft - 2\ ft - 10\ ft - 2\ ft = 62\ ft$ *(clear distance between curbs)*

$b := 12\ in$ *(unit width of concrete deck in direction of traffic)*

$(t_{slab})_{min} := 1\ ft + 6\ in$ *(thickness of concrete slab at midspan)*

$(t_{slab})_{max} := 2\ ft + 0\ in$ *(thickness of concrete slab at bents)*

$t := (t_{slab})_{min} = 18\ in$ *(thickness of concrete deck at midspan)*

$t_{ws} := 0\ in$ *(thickness of asphalt overlay)*

$cover_t := 2\ in$ *(concrete cover over top reinforcing bars in slab)*

$cover_b := 1\ in$ *(concrete cover over bottom reinforcing bars in slab)*

$cover := cover_t - t_{ws} = 2\ in$

$S_x := \frac{b \cdot t^2}{6}$ *(section modulus)*

$x_1 := 1\ ft + 5\ ft + 1\ ft = 7\ ft$ *(distance between the edge of the deck and the inside face of the downstream barrier at monolith 3)*

Design Lanes:

$w := min\left(b_{clr}\right) = 62\ ft$ *(clear roadway between curbs; LRFD 3.6.1.1.1)*

$N_L := \mathbf{if}\left(w \geq 18\ ft, \max\left(2, \mathrm{trunc}\left(\frac{w}{12\ ft}\right)\right), 1\right)$ *(number of design lanes; LRFD 3.6.1.1.1 & MBE 6A.2.3.2)*

$N_L = 5$

$b_L := \mathbf{if}\left(w \geq 18\ ft, min\left(\frac{w}{2}, 12\ ft\right), min\left(w, 12\ ft\right)\right)$ *(width of design lanes; LRFD 3.6.1.1.1 & MBE 6A.2.3.2)*

$b_L = 12\ ft$

Reinforcement Properties

$bar_b :=$ "#10" *(size of bottom bars)*

$bar_t :=$ "#10" *(size of top bars)*

$bar := bar_b$ *(size of flexural bars)*

$d_b = 1.27\ in$ *(diameter of bottom bars)*

$d_t = 1.27\ in$ *(diameter of top bars)*

$d = 1.27\ in$ *(diameter of flexural bars)*

$a_s := \frac{\pi \cdot d^2}{4} = 1.267\ in^2$ *(cross-sectional area of bar)*

$s_s := 5\ in$ *(spacing of reinforcing bars)*

$n := 1$ *(number of bars in section)*

$A_s := n \cdot \left(\frac{a_s}{s_s}\right) \cdot b = 3\ in^2$ *(total cross-sectional area of positive flexural reinforcing steel)*

$\bar{y}_s := cover + \frac{d}{2} = 2.64\ in$ *(location of centroid of flexural reinforcing steel)*

$d_s := t - \bar{y}_s = 15.37\ in$ *(distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement)*

LIVE LOAD DISTRIBUTION

$L_1 := min\left(L, 60\ ft\right) = 53.3\ ft$ *(modified span length)*

$W_{1.s} := min\left(W, 30\ ft\right) = 30\ ft$ *(modified edge-to-edge width for single-lane loading)*

$W_{1.m} := min\left(W, 60\ ft\right) = 60\ ft$ *(modified edge-to-edge width for multilane loading)*

Effective Strip Width (Interior Strip)

$E_s := \left(10 + 5 \cdot \sqrt{L_1 \cdot W_{1.s} \cdot ft^{-2}}\right) \cdot in = 210\ in$ *(equivalent width for single-lane loading; LRFD Eq. 4.6.2.3-1)*

$E_m := min\left(\left(84 + 1.44 \cdot \sqrt{L_1 \cdot W_{1.m} \cdot ft^{-2}}\right) \cdot in, \frac{12 \cdot W}{N_L}\right) = 165\ in$ *(equivalent width for multilane loading; LRFD Eq. 4.6.2.3-2)*

$E_1 := min\left(E_s, E_m\right) = 165\ in$ *(effective strip width for interior strips)*

Effective Strip Width (Edge Strip)

$E_2 := min\left(x_1 + 0.25 \cdot E_s + 12\ in, 0.5 \cdot E_s, 72\ in\right)$

$E_2 = 72\ in$ *(effective edge strip width - LRFD Eq. 4.6.2.1.4b)*

Lane Load Reduction for Edge Strip

LRFD Design Article 4.6.2.1.4b states that "edge beams shall be assumed to support one line of wheels and, where appropriate, a tributary portion of the design lane load." For this reason, the live load effects due to design truck loading will be reduced by 50% when determining the total live load effects for the edge strip in the calculations below. The live load effects due to design lane loading will be reduced in proportion to the width of lane load tributary to the edge strip.

$x_{trib} := \max\left(min\left(E_2 - x_1, 10\ ft\right), 0\ ft\right) = 0\ ft$ *(portion of design lane load tributary to the edge strip)*

$R_2 := \frac{x_{trib}}{10\ ft} = 0$ *(reduction factor for lane load tributary to edge strip)*

$(b_{eq})_{pos} := \left(26 + 6.6 \cdot S \cdot ft^{-1}\right) \cdot in = 31.5\ ft$ *(width of primary strip for positive flexure; LRFD Table 4.6.2.1.3-1)*

$(b_{eq})_{neg} := \left(48 + 3.0 \cdot S \cdot ft^{-1}\right) \cdot in = 17.33\ ft$ *(width of primary strip for negative flexure; LRFD Table 4.6.2.1.3-1)*

$b_{eq} := (b_{eq})_{pos} = 31.5\ ft$ *(controlling width of primary strip)*

LOAD EFFECTS

$(M_{DC})_{total} := 2922\ kip \cdot ft$ *(moment due to dead load, total cross-section)*

$M_{DC} := \left(\frac{(M_{DC})_{total}}{W}\right) \cdot b = 38.4\ kip \cdot ft$ *(moment due to dead load, effective width)*

$M_{DW} := 0\ kip \cdot ft$

$M_{LLL} := 165\ kip \cdot ft$

$M_{TLL} := 494\ kip \cdot ft$

$(M_{LL})_1 := \frac{\left(M_{TLL} \cdot IM + M_{LLL}\right) \cdot b}{E_1} = 59.6\ kip \cdot ft$ *(total live load moment at cross-section for interior strips)*

$(M_{LL})_2 := \frac{\left(50\% \cdot M_{TLL} \cdot IM + R_2 \cdot M_{LLL}\right) \cdot b}{E_2} = 54.8\ kip \cdot ft$ *(total live load moment at cross-section for edge strips)*

$M_{LL} := \max\left((M_{LL})_1, (M_{LL})_2\right) = 59.6\ kip \cdot ft$ *(controlling total live load moment at cross-section)*

$M_u := (\gamma_L)_{inv} \cdot M_{LL} + \gamma_{DC} \cdot M_{DC} + \gamma_{DW} \cdot M_{DW} = 152.4\ kip \cdot ft$ *(ultimate moment at cross-section)*

NOMINAL FLEXURAL RESISTANCE

Flexural Resistance

Stress Block Factors:

$$\beta_1 := \begin{cases} \text{if } f'_c < 4\ ksi \\ \quad 0.85 \\ \text{else} \\ \quad \max\left(0.85 - 0.05 \cdot \frac{\langle f'_c - 4\ ksi \rangle}{ksi}, 0.65\right) \end{cases}$$

(stress block factor; LRFD 5.6.2.2)

$\beta_1 = 0.80$

$$\alpha_1 := \begin{cases} \text{if } f'_c < 10\ ksi \\ \quad 0.85 \\ \text{else} \\ \quad \max\left(0.85 - 0.02 \cdot \frac{\langle f'_c - 10\ ksi \rangle}{ksi}, 0.75\right) \end{cases}$$

(stress block factor; LRFD 5.6.2.2)

$\alpha_1 = 0.85$

$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} = 4.5\ in$

(distance between the neutral axis and the compressive face for rectangular behavior with bonded strands - LRFD Eq. 5.7.3.1.1-4)

$a := \beta_1 \cdot c = 3.6\ in$

(depth of equivalent stress block for rectangular section behavior with bonded strands)

$M_n := A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 206.4\ kip \cdot ft$

(nominal flexural resistance; LRFD 5.7.3.2.2 & 5.7.3.2.3)

Resistance Factor

$\varepsilon_t := \left(\frac{0.003}{c}\right) \cdot \langle d_s - c \rangle = 0.0073$

(net tensile strain; LRFD 5.6.2.1)

$\varepsilon_{cl} := 0.002$

(compression-controlled strain limit; LRFD 5.6.2.1)

$\varepsilon_{tl} := 0.005$

(tension-controlled strain limit; LRFD 5.6.2.1)

$\varphi_f := \max\left(min\left((\varphi_f)_c + \frac{0.15 \cdot \langle \varepsilon_t - \varepsilon_{cl} \rangle}{\langle \varepsilon_{tl} - \varepsilon_{cl} \rangle}, (\varphi_f)_t\right), (\varphi_f)_c\right)$

(flexural resistance factor prestressed members; LRFD Eq. 5.5.4.2-1)

$\varphi_f = 0.9$

$M_r := \varphi_f \cdot M_n = 186\ kip \cdot ft$

(flexural capacity)

$IC_f := \frac{M_u}{M_r} = 0.82$

$check_f := \left\| \begin{array}{l} \text{if } IC_f \le 1.0 \\ \quad \text{"okay"} \\ \text{else} \\ \quad \text{"no good"} \end{array} \right\|$

$check_f = \text{"okay"}$

Minimum Reinforcement

$M_{cr} = \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right)$ *(cracking moment; LRFD Eq. 5.6.3.3.2-1)*

$S_c = S_{nc}$ ***therefore:*** $M_{dnc}(S_c/S_{nc} - 1) := 0 \ kip \cdot ft$

Minimum Reinforcement

$M_{cr} = \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right)$ *(cracking moment; LRFD Eq. 5.6.3.3.2-1)*

$S_c = S_{nc}$ ***therefore:*** $M_{dnc}(S_c/S_{nc} - 1) := 0 \ kip \cdot ft$

$S_c := S_x$

$f_r := 0.24 \cdot \sqrt{f'_c \cdot ksi} = 0.5 \ ksi$ *(modulus of rupture; LRFD 5.4.2.6)*

$f_{cpe} := 0 \ ksi$ *(compressive stress in concrete due to effective prestress force)*

$\gamma_1 := 1.6$ *(flexural cracking variability factor)*

$\gamma_2 := 1.0$ *(prestress variability factor)*

$\gamma_3 := \frac{f_y}{f_u}$ *(ratio of specified minimum yield strength to ultimate strength)*

$M_{cr} := \overrightarrow{\gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc}(S_c/S_{nc} - 1) \right)}$ *(cracking moment; LRFD Eq. 5.6.3.3.2-1)*

$check := \left\| \begin{array}{l} \text{if } M_r \ge min\left(1.33 \cdot M_u, M_{cr}\right) \\ \quad \text{"okay"} \\ \text{else} \\ \quad \text{"reduce Mn"} \end{array} \right\|$

$check = \text{"okay"}$

$$K := \begin{cases} \text{if } M_r \geq min\left(1.33 \cdot M_u, M_{cr}\right) \\ \quad 1.0 \\ \text{else} \\ \quad min\left(1.0, \dfrac{M_r}{min\left(1.33 \cdot M_u, M_{cr}\right)}\right) \end{cases}$$

(minimum reinforcement reduction factor - MBE 6A.5.6)

$K = 1.00$

RATING FACTORS: FLEXURE

$\gamma_{DC} = 1.25$ *(dead load factor for components and attachments)*

$\gamma_{DW} = 1.50$ *(dead load factor for wearing surface)*

$R_n := M_n$ *(nominal member resistance)*

$C := \varphi_c \cdot \varphi_s \cdot \varphi_f \cdot R_n = 186\ kip \cdot ft$ *(capacity, MBE Eqs. 6A.4.2.1-2 & 6A.4.2.1-3)*

$DC := M_{DC} = 38.4\ kip \cdot ft$ *(dead load effect due to structural components and attachments)*

$DW := M_{DW} = 0\ kip \cdot ft$ *(dead load effect due to wearing surface and utilities)*

$LL := M_{LL} = 59.6\ kip \cdot ft$ *(live load effect)*

$$RF_{inv} := \frac{C - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW}{(\gamma_L)_{inv} \cdot LL}$$

(inventory design rating factor for positive flexure relative to the compression flange; MBE Eq. 6A.4.2.1-1)

$RF_{inv} = 1.32$

$$RF_{op} := \frac{(\gamma_L)_{inv}}{(\gamma_L)_{op}} \cdot RF_{inv}$$

(operating design rating factor for positive flexure)

$RF_{op} = 1.71$

Job Information

	Engineer	Checked	Approved
Name:			
Date:			

Project ID	
Project Name	

Structure Type	PLANE FRAME

Number of Nodes	4	Highest Node	103
Number of Elements	3	Highest Beam	103

Number of Basic Load Cases	956
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Generation	1	LOAD GENERATION, LOAD #1, (1 of 188
Primary	10001	LANE LOAD - ALL SPANS
Primary	10002	LANE LOAD - SPAN 1 ONLY
Primary	10003	LANE LOAD - SPAN 2 ONLY
Primary	10004	LANE LOAD - SPAN 3 ONLY
Primary	10005	LANE LOAD - SPAN 1 & 2
Primary	10006	LANE LOAD - SPAN 2 & 3
Primary	10007	LANE LOAD - SPAN 1 & 3
Primary	20001	DEAD LOAD DC

Nodes

Node	X (ft)	Y (ft)	Z (ft)
100	0	0	0
101	48.000	0	0
102	108.500	0	0
103	156.500	0	0

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
101	100	101	48.000	1	0

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
100	Fixed	Fixed	Fixed	-	-	-
101	Fixed	Fixed	Fixed	-	-	-
102	Fixed	Fixed	Fixed	-	-	-
103	Fixed	Fixed	Fixed	-	-	-

Primary Load Cases

Number	Name	Type
10001	LANE LOAD - ALL SPANS	None
10002	LANE LOAD - SPAN 1 ONLY	None
10003	LANE LOAD - SPAN 2 ONLY	None
10004	LANE LOAD - SPAN 3 ONLY	None
10005	LANE LOAD - SPAN 1 & 2	None
10006	LANE LOAD - SPAN 2 & 3	None
10007	LANE LOAD - SPAN 1 & 3	None
20001	DEAD LOAD DC	None

Moving Load Definition : Type 1

Width (ft)
-

Force (kip)	Distance (ft)
8.000	-
32.000	14.000
32.000	14.000

Moving Load Definition : Type 2

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	14.000
8.000	14.000

Moving Load Definition : Type 3

Width (ft)
-

Force (kip)	Distance (ft)
8.000	-
32.000	14.000
32.000	30.000

Moving Load Definition : Type 4

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	30.000
8.000	14.000

Moving Load Definition : Type 5

Width (ft)
-

Force (kip)	Distance (ft)
25.000	-
25.000	4.000

Moving Loads: Loads 1 to 188

	Initial Position			Increment			
Type	X (ft)	Y (ft)	Z (ft)	X (ft)	Y (ft)	Z (ft)	Range (ft)
1	-28.000	0	0	1.000	-	-	-

10001 LANE LOAD - ALL SPANS : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-
102	UNI kips/ft	GY	-0.640	-	-	-	-
103	UNI kips/ft	GY	-0.640	-	-	-	-

10002 LANE LOAD - SPAN 1 ONLY : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-

10003 LANE LOAD - SPAN 2 ONLY : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
102	UNI kips/ft	GY	-0.640	-	-	-	-

10004 LANE LOAD - SPAN 3 ONLY : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
103	UNI kips/ft	GY	-0.640	-	-	-	-

10005 LANE LOAD - SPAN 1 & 2 : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-
102	UNI kips/ft	GY	-0.640	-	-	-	-

10006 LANE LOAD - SPAN 2 & 3 : Beam Loads

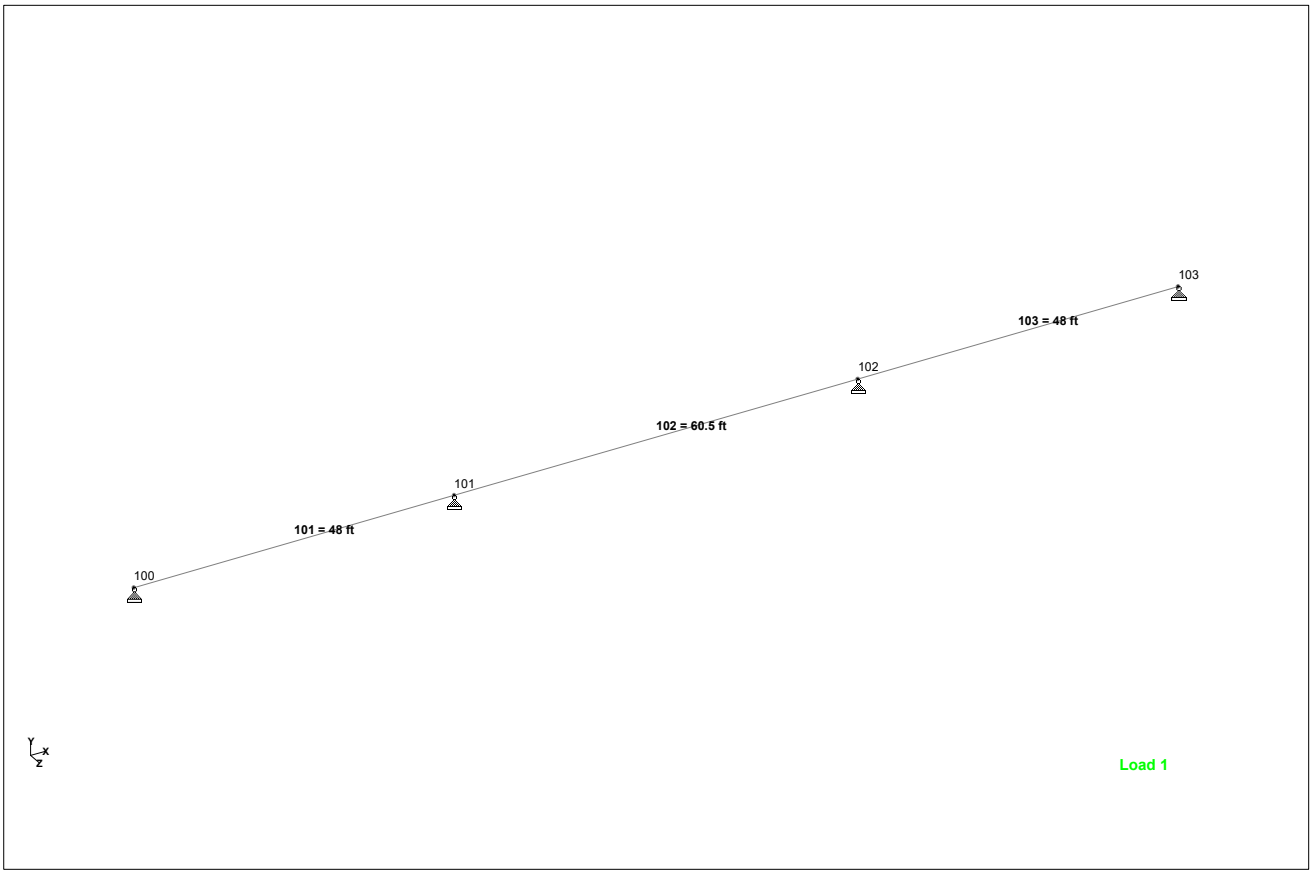
Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
102	UNI kips/ft	GY	-0.640	-	-	-	-
103	UNI kips/ft	GY	-0.640	-	-	-	-

10007 LANE LOAD - SPAN 1 & 3 : Beam Loads

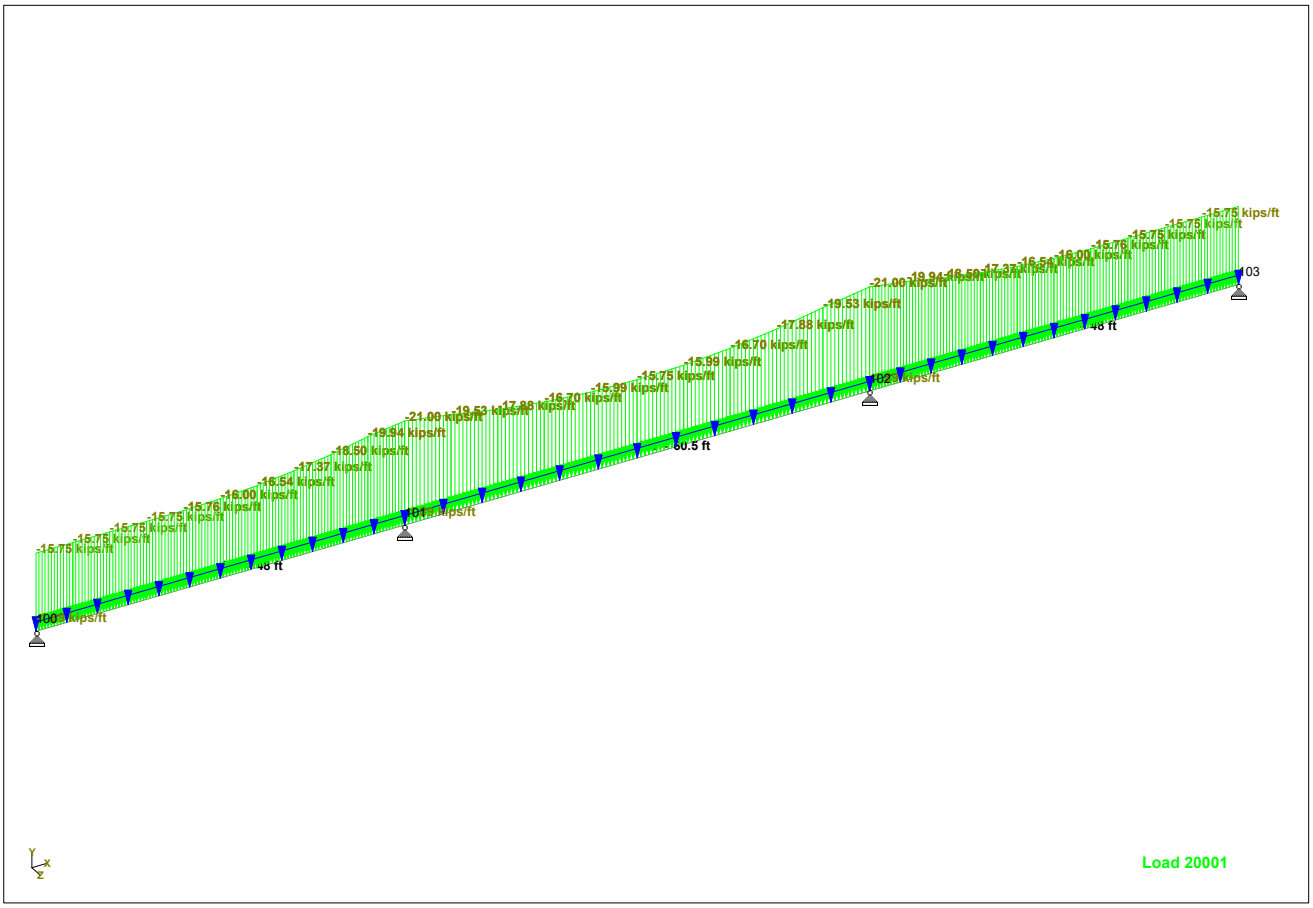
Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-
103	UNI kips/ft	GY	-0.640	-	-	-	-

20001 DEAD LOAD DC : Beam Loads

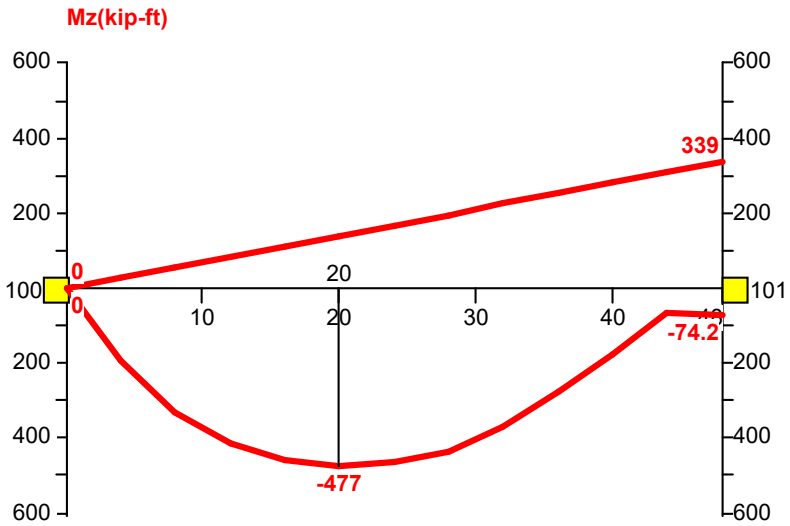
Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-1.894	-	-	-	-
	TRAP kips/ft	GY	-15.750	0	-15.750	4.799	-
	TRAP kips/ft	GY	-15.750	4.799	-15.750	9.599	-
	TRAP kips/ft	GY	-15.750	9.599	-15.750	14.398	-
	TRAP kips/ft	GY	-15.750	14.398	-15.764	19.198	-
	TRAP kips/ft	GY	-15.764	19.198	-16.003	23.997	-
	TRAP kips/ft	GY	-16.003	23.997	-16.540	28.797	-
	TRAP kips/ft	GY	-16.540	28.797	-17.374	33.596	-
	TRAP kips/ft	GY	-17.374	33.596	-18.505	38.396	-
	TRAP kips/ft	GY	-18.505	38.396	-19.937	43.195	-
	TRAP kips/ft	GY	-19.937	43.195	-21.000	48.000	-
102	UNI kips/ft	GY	-1.894	-	-	-	-
	TRAP kips/ft	GY	-21.000	0	-19.534	6.049	-
	TRAP kips/ft	GY	-19.534	6.049	-17.879	12.099	-
	TRAP kips/ft	GY	-17.879	12.099	-16.697	18.148	-
	TRAP kips/ft	GY	-16.697	18.148	-15.986	24.198	-
	TRAP kips/ft	GY	-15.986	24.198	-15.750	30.247	-
	TRAP kips/ft	GY	-15.750	30.247	-15.986	36.297	-
	TRAP kips/ft	GY	-15.986	36.297	-16.697	42.346	-
	TRAP kips/ft	GY	-16.697	42.346	-17.879	48.396	-
	TRAP kips/ft	GY	-17.879	48.396	-19.534	54.445	-
	TRAP kips/ft	GY	-19.534	54.445	-21.000	60.495	-
103	UNI kips/ft	GY	-1.894	-	-	-	-
	TRAP kips/ft	GY	-21.000	0	-19.937	4.799	-
	TRAP kips/ft	GY	-19.937	4.799	-18.505	9.599	-
	TRAP kips/ft	GY	-18.505	9.599	-17.374	14.398	-
	TRAP kips/ft	GY	-17.374	14.398	-16.540	19.198	-
	TRAP kips/ft	GY	-16.540	19.198	-16.003	23.997	-
	TRAP kips/ft	GY	-16.003	23.997	-15.764	28.797	-
	TRAP kips/ft	GY	-15.764	28.797	-15.750	33.596	-
	TRAP kips/ft	GY	-15.750	33.596	-15.750	38.396	-
	TRAP kips/ft	GY	-15.750	38.396	-15.750	43.195	-
	TRAP kips/ft	GY	-15.750	43.195	-15.750	47.995	-



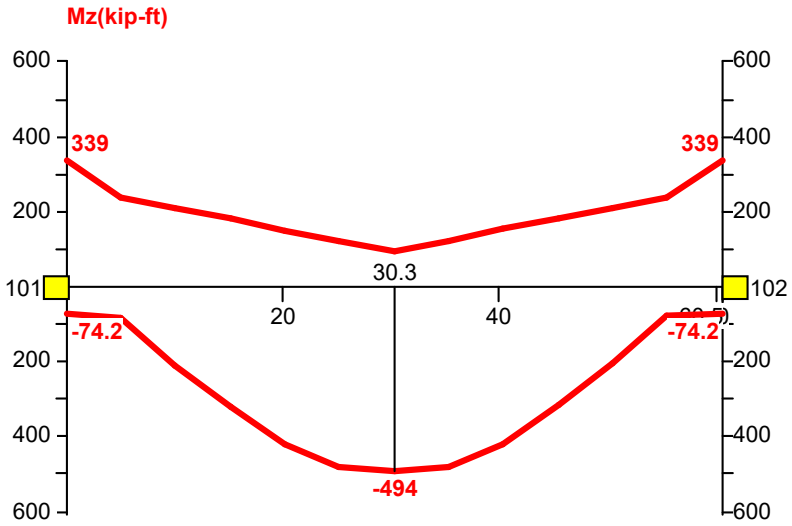
Dimensions



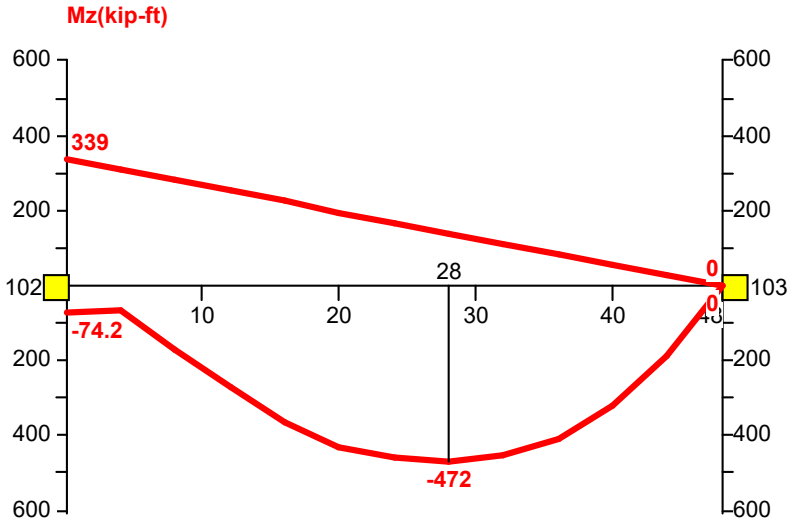
Dead Load



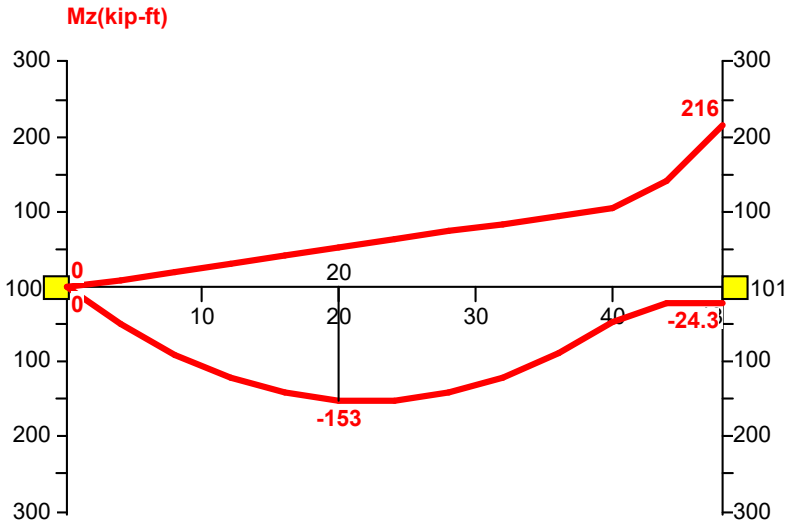
Span 1 Moment HL93 Truck/Tandem



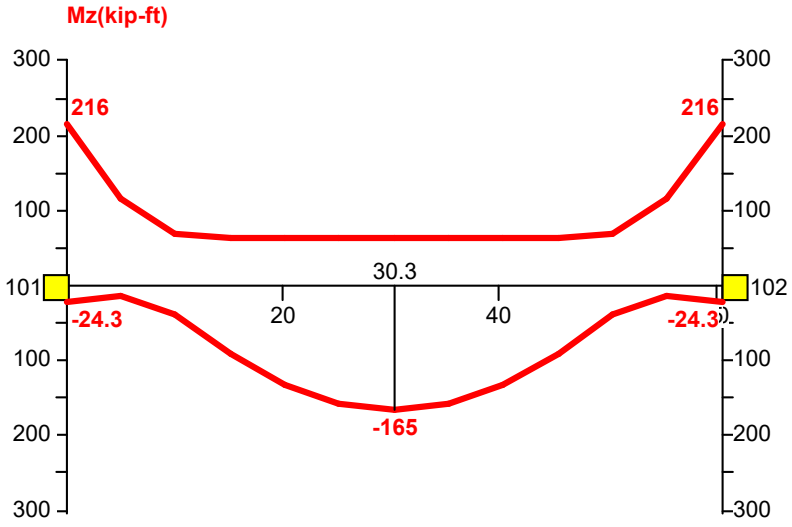
Span 2 Moment HL93 Truck/Tandem



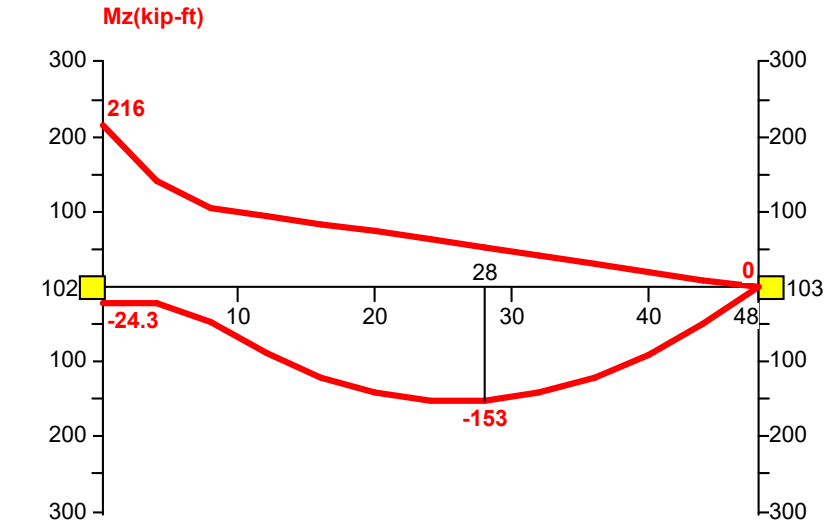
Span 3 Moment HL93 Truck/Tandem



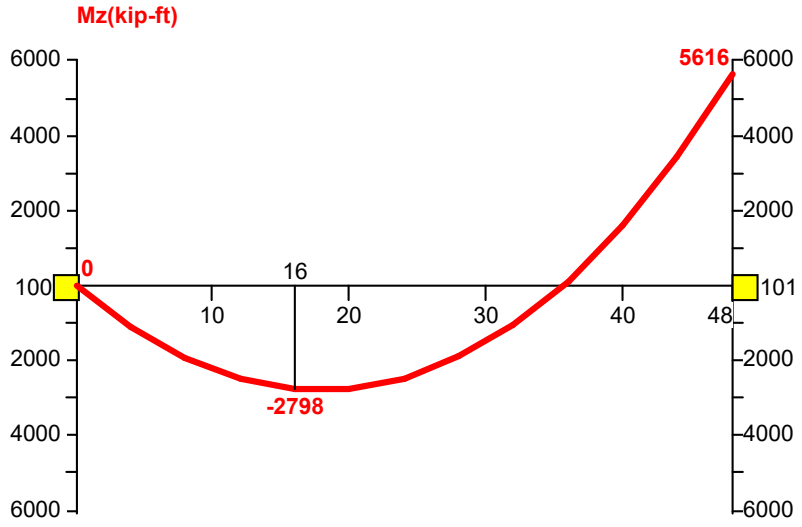
Span 1 Moment HL93 Lane Load



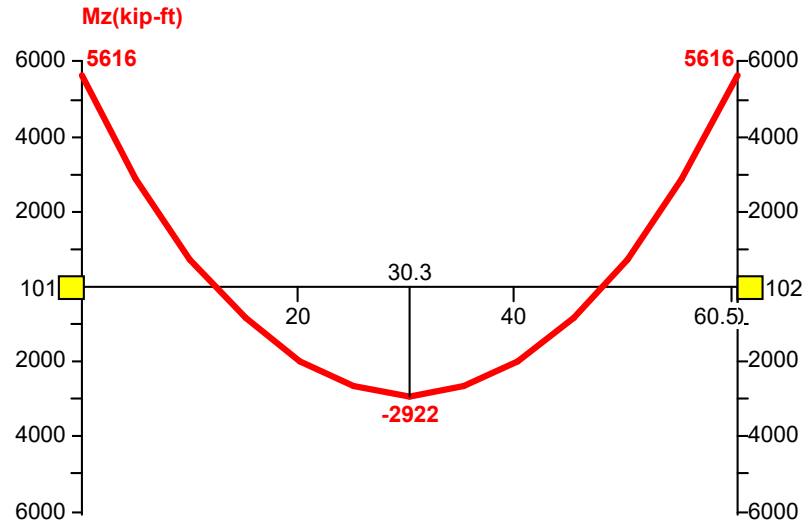
Span 2 Moment HL93 Lane Load



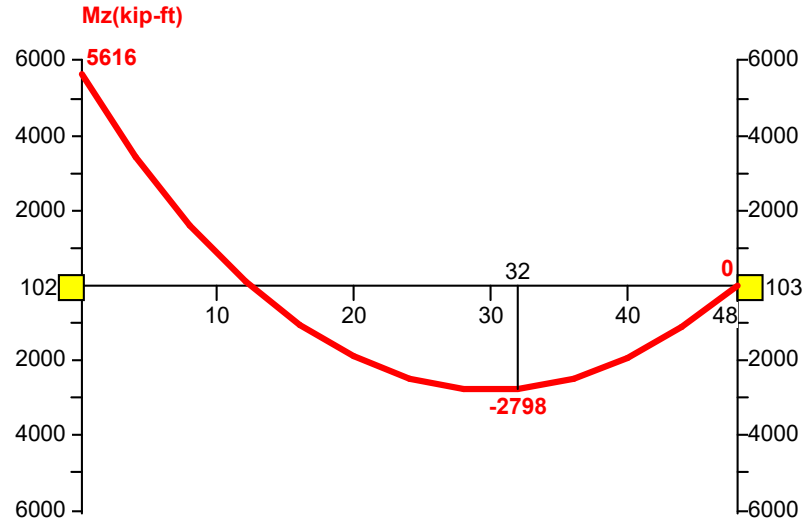
Span 3 Moment HL93 Lane Load



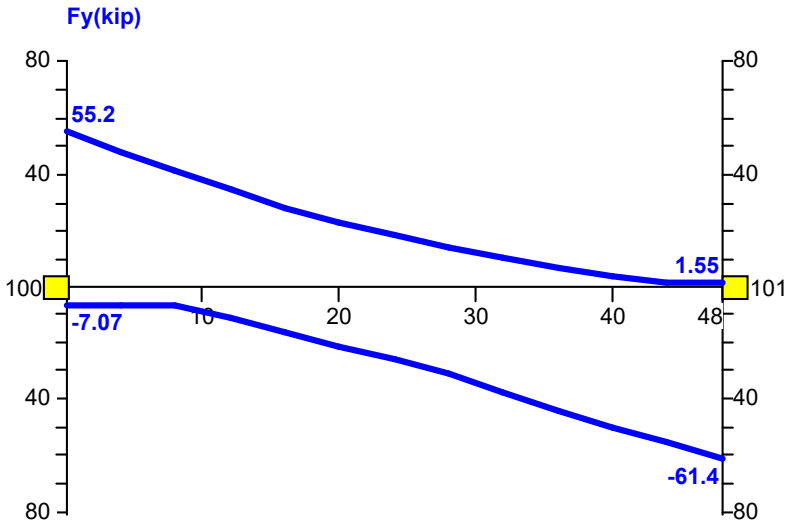
Span 1 Moment Dead Load DC



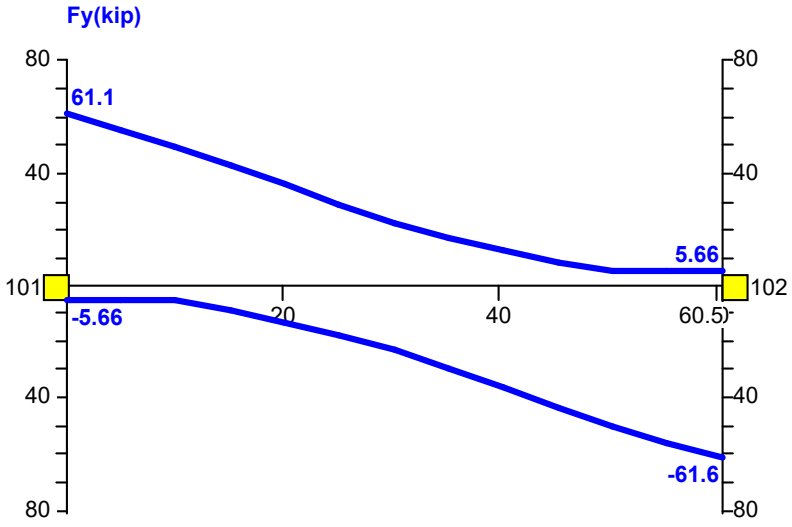
Span 2 Moment Dead Load DC



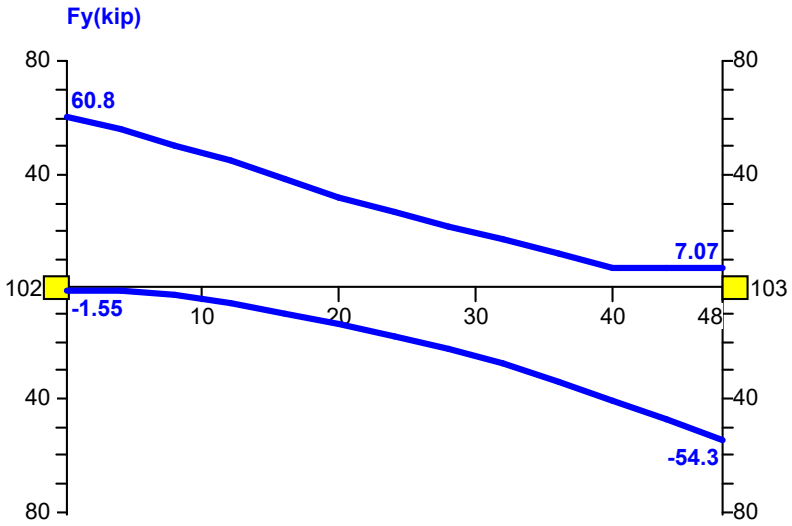
Span 3 Moment Dead Load DC



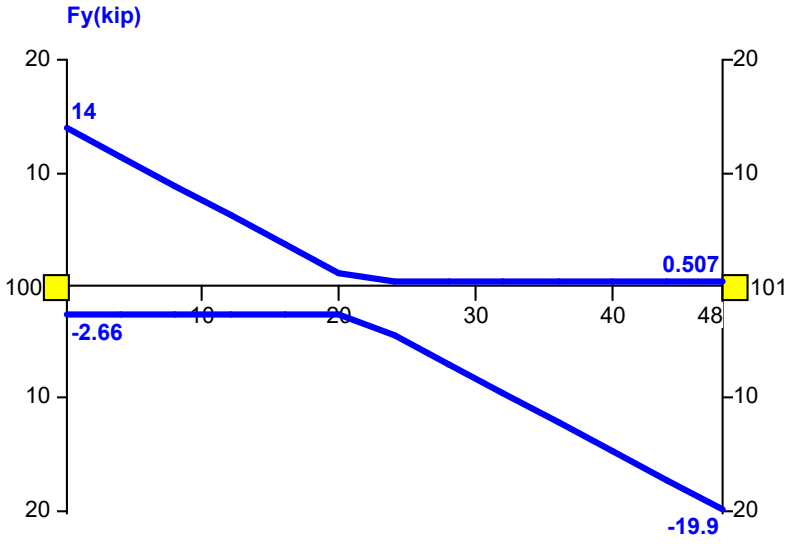
Span 1 Shear HL93 Truck/Tandem



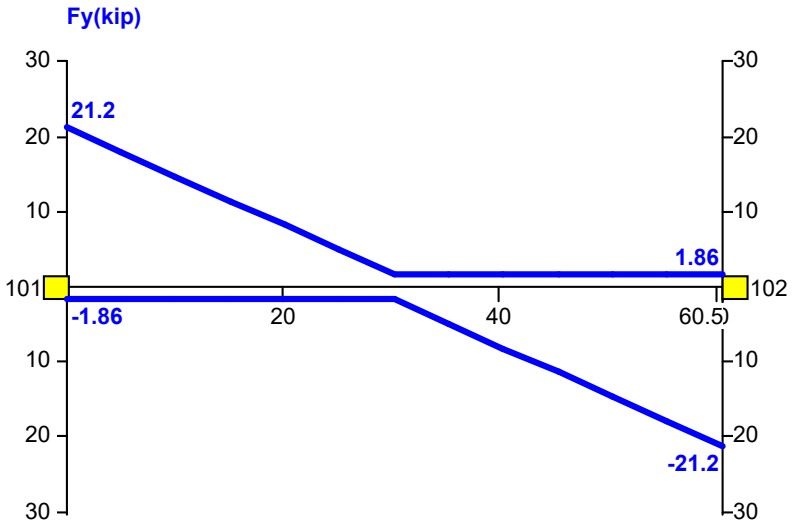
Span 2 Shear HL93 Truck/Tandem



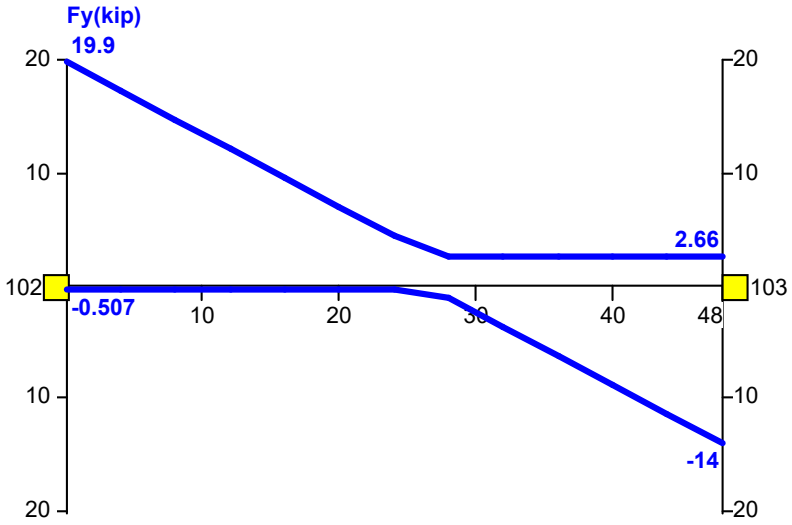
Span 3 Shear HL93 Truck/Tandem



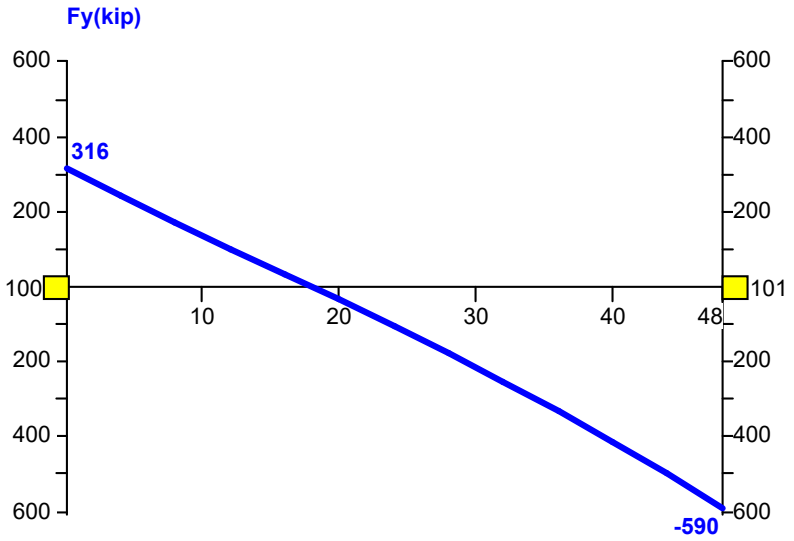
Span 1 Shear HL93 Lane Load



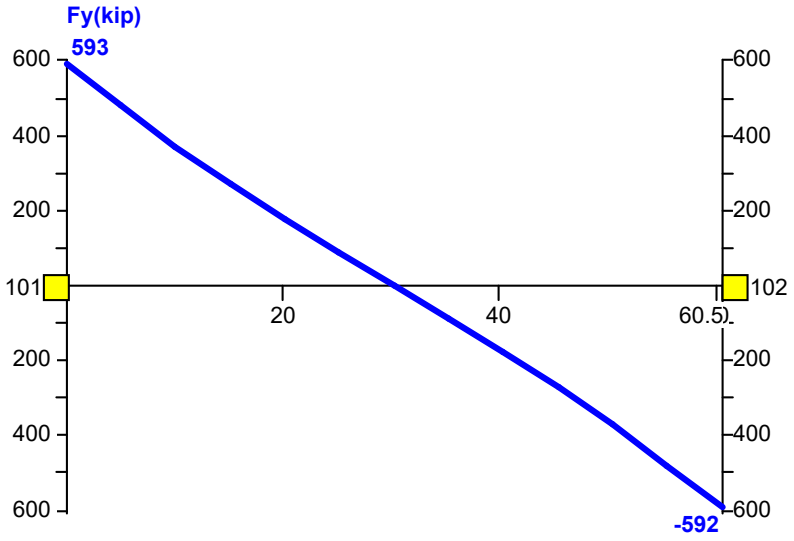
Span 2 Shear HL93 Lane Load



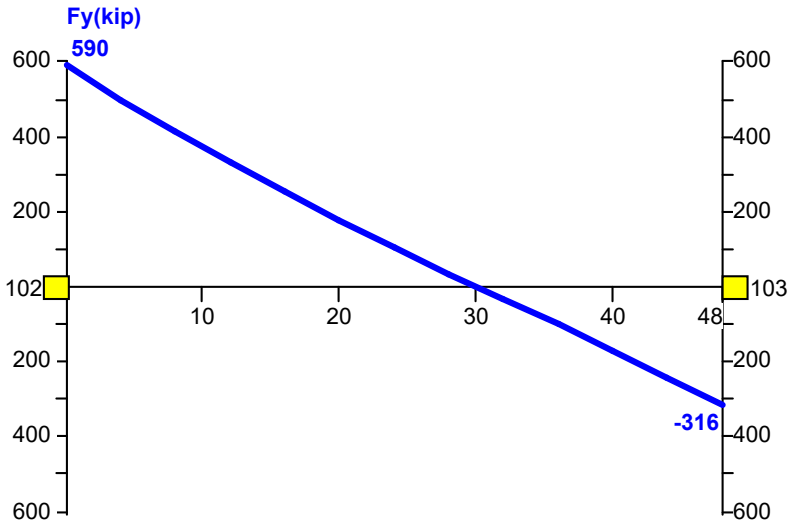
Span 3 Shear HL93 Lane Load



Span 1 Shear Dead Load DC



Span 2 Shear Dead Load DC



Span 3 Shear Dead Load DC

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
100	+ve	0	55.226	0	0	0	0
100	+ve	-	Load: 217	-	-	-	-
100	-ve	0	-7.067	0	0	0	0
100	-ve	-	Load: 280	-	-	-	-
101	+ve	0	68.510	0	0	0	0
101	+ve	-	Load: 257	-	-	-	-
101	-ve	0	-7.207	0	0	0	0
101	-ve	-	Load: 338	-	-	-	-
102	+ve	0	68.512	0	0	0	0
102	+ve	-	Load: 118	-	-	-	-
102	-ve	0	-7.204	0	0	0	0
102	-ve	-	Load: 36	-	-	-	-
103	+ve	0	54.343	0	0	0	0
103	+ve	-	Load: 157	-	-	-	-
103	-ve	0	-7.069	0	0	0	0
103	-ve	-	Load: 94	-	-	-	-

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
100	+ve	0	14.033	0	0	0	0
100	+ve	-	Load: 10007	-	-	-	-
100	-ve	0	-2.656	0	0	0	0
100	-ve	-	Load: 10003	-	-	-	-
101	+ve	0	41.068	0	0	0	0
101	+ve	-	Load: 10005	-	-	-	-
101	-ve	0	-2.365	0	0	0	0
101	-ve	-	Load: 10004	-	-	-	-
102	+ve	0	41.068	0	0	0	0
102	+ve	-	Load: 10006	-	-	-	-
102	-ve	0	-2.365	0	0	0	0
102	-ve	-	Load: 10002	-	-	-	-
103	+ve	0	14.033	0	0	0	0
103	+ve	-	Load: 10007	-	-	-	-
103	-ve	0	-2.656	0	0	0	0
103	-ve	-	Load: 10003	-	-	-	-

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
100	+ve	0	316.032	0	0	0	0
100	+ve	-	Load: 20001	-	-	-	-
100	-ve	0	0	0	0	0	0
100	-ve	-	-	-	-	-	-
101	+ve	0	1.18E+3	0	0	0	0
101	+ve	-	Load: 20001	-	-	-	-
101	-ve	0	0	0	0	0	0
101	-ve	-	-	-	-	-	-
102	+ve	0	1.18E+3	0	0	0	0
102	+ve	-	Load: 20001	-	-	-	-
102	-ve	0	0	0	0	0	0
102	-ve	-	-	-	-	-	-
103	+ve	0	315.945	0	0	0	0
103	+ve	-	Load: 20001	-	-	-	-
103	-ve	0	0	0	0	0	0
103	-ve	-	-	-	-	-	-

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Merriam Drive Bridge Design

UPPER TURKEY CREEK

MERRIAM, KS

Include << .\A - User Defined Units and Functions.mcdx

GENERAL

Traffic Information

$ADT := 10000$ *(average daily traffic; assumed)*

$ADTT_{percent} := 10\%$ *(average daily truck traffic in percentage of ADT; assumed)*

$ADTT := (ADTT_{percent}) \cdot (ADT) = 1000$ *(average daily truck traffic)*

$v_{PSL} := 35\ mph$ *(posted speed limited)*

$v_{DSL} := 45\ mph$ *(design speed limited; assumed)*

$N_L := 4$ *(number of design lanes)*

$n_L := 2$ *(number lanes with traffic in the same direction)*

Material Properties

$\gamma_c := 150\ pcf$ *(unit weight of concrete)*

$\gamma_w := 62.4\ pcf$ *(unit weight of water)*

Hydraulic Properties

$Q_{US} := 18357\ cfs$ *(upstream flowrate; NWK H&H)*

$V_{US} := 10.3\ fps$ *(maximum upstream velocity; NWK H&H)*

Note: The flat face of the flood wall will have a drag coefficient similar to a square-ended pier.

$(C_D)_{flood.wall} := 1.4$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

$(C_D)_{pier} := 0.7$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

Wind Design Properties

$roughness :=$ ground surface roughness d *(ground surface roughness; LRFD 3.8.1.1.4)*

$wind.exposure :=$ wind exposure category d *(wind exposure category; LRFD 3.8.1.1.5)*

$V_{III} := 115\ mph$ *(design 3-second gust wind speed for Strength III; LRFD Table 3.8.1.1.2-1)*

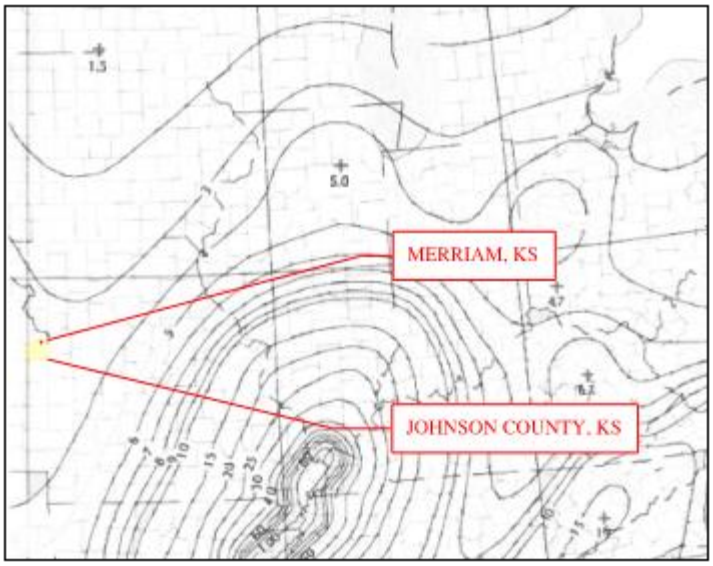
$G := 1.00$ *(gust effect factor; LRFD 3.8.1.2.1)*

$(C_D)_W := 1.3$ *(gust effect factor; LRFD Table 3.8.1.2.1-2)*

Seismic Design Properties

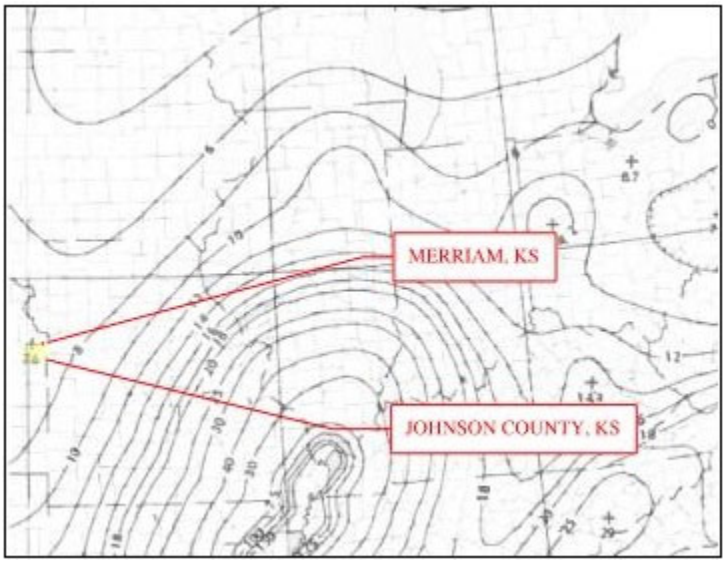
$site.class :=$ "d" *(site classification, assumed)*

LRFD Figure 3.10.2.1-1:



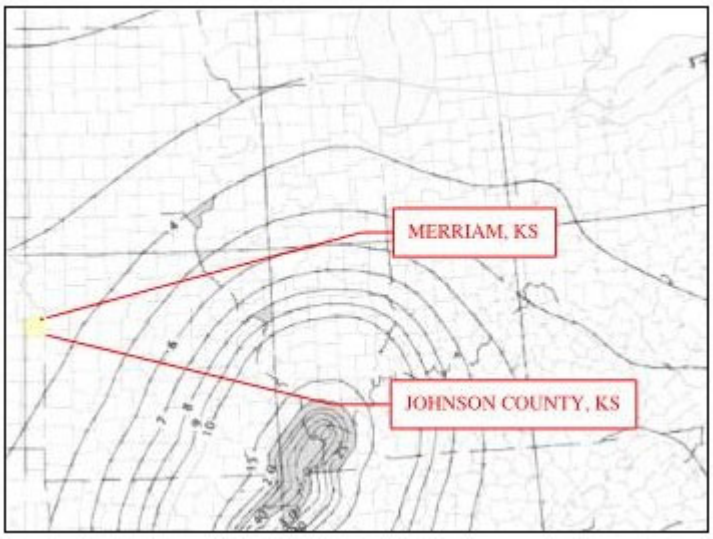
$PGA := 3.5\%$ *(peak seismic ground acceleration coefficient on rock; LRFD 9th Ed. Figure 3.10.2.1-1)*

LRFD Figure 3.10.2.1-2:



$S_S := 7.6\%$ *(horizontal response spectral acceleration coefficient at 0.2-s period on rock - site class b; LRFD 9th Ed. Figure 3.10.2.1-2)*

LRFD Figure 3.10.2.1-3:



$S_1 := 3.7\%$ *(horizontal response spectral acceleration coefficient at 1-s period on rock - site class b; LRFD 9th Ed. Figure 3.10.2.1-3)*

Load and Resistance Factors

Load Modifiers:

$\eta_D := 1.00$ *(load modifying factor for ductility; LRFD 1.3.3)*

$\eta_R := 1.00$ *(load modifying factor for redundancy; LRFD 1.3.4)*

$\eta_I := 1.00$ *(load modifying factor for operational classification; LRFD 1.3.5)*

$(\eta_{strength})_{max} := \max\left(\eta_D \cdot \eta_R \cdot \eta_I, 0.95\right) = 1$ *(total load modification for loads for which a maximum of γ is appropriate for strength limit states; LRFD Eq. 1.3.2.1-2)*

$(\eta_{strength})_{min} := min\left(\frac{1}{\eta_D \cdot \eta_R \cdot \eta_I}, 1.0\right) = 1.00$ *(total load modification for loads for which a minimum of γ is appropriate for strength limit states; LRFD Eq. 1.3.2.1-3)*

Dynamic Load Allowance & Multiple Presence Factors

$IM := 1.33$ *(dynamic load allowance; LRFD Table 3.6.2.1-1)*

$$\gamma_{ADTT} := \begin{cases} 0.9 & \text{if } ADTT \le 100 \\ 0.95 & \text{else if } 100 \le ADTT \le 1000 \\ 1.0 & \text{else} \end{cases}$$ *(load reduction for low average daily truck traffic; LRFD C3.6.1.1.2)*

$\gamma_{ADTT} = 1$

$(m_1)_o := 1.2$ *(multiple presence factor for single lane loaded; LRFD Table 3.6.1.1.2-1)*

$(m_2)_o := 1.0$ *(multiple presence factor for two lanes loaded; LRFD Table 3.6.1.1.2-1)*

$(m_3)_o := 0.85$ *(multiple presence factor for three lanes loaded; LRFD Table 3.6.1.1.2-1)*

$(m_4)_o := 0.65$ *(multiple presence factor for four or more lanes loaded; LRFD Table 3.6.1.1.2-1)*

$m_1 := \gamma_{ADTT} \cdot (m_1)_o = 1.14$ *(multiple presence factor for single lane loaded adjusted for low ADTT)*

$m_2 := \gamma_{ADTT} \cdot (m_2)_o = 0.95$ *(multiple presence factor for two lanes loaded adjusted for low ADTT)*

$m_3 := \gamma_{ADTT} \cdot (m_3)_o = 0.81$ *(multiple presence factor for three lanes loaded adjusted for low ADTT)*

$m_4 := \gamma_{ADTT} \cdot (m_4)_o = 0.62$ *(multiple presence factor for three lanes loaded adjusted for low ADTT)*

$$N_i = \begin{bmatrix} 1 \\ 2 \\ 3 \\ 4 \end{bmatrix} \qquad m_i = \begin{bmatrix} 1.14 \\ 0.95 \\ 0.81 \\ 0.62 \end{bmatrix} \qquad N_i m_i = \begin{bmatrix} 1.14 \\ 1.9 \\ 2.42 \\ 2.47 \end{bmatrix}$$

$g_{dist} := \max\left(N_i m_i\right) = 2.47$ *(distribution of traffic loads)*

DIMENSIONS

General Dimensions

General Bridge Dimensions:

$L_b := 180\ ft$ *(length of bridge)*

$L_1 := \frac{L_b}{3} = 60\ ft$ *(length of span 1)*

$L_2 := \frac{L_b}{3} = 60\ ft$ *(length of span 2)*

$L_3 := \frac{L_b}{3} = 60\ ft$ *(length of span 3)*

Roadway Dimensions:

$d_e := 2\ ft$ *(distance from the exterior girder to the interior edge of curb)*

$x_{WL} := 2\ ft$ *(distance from first wheel line to curb; LRFD 3.6.1.3.1)*

Deck Dimensions:

$b_{slab} := 70\ ft$ *(width of bridge deck slab)*

$(t_{slab})_{min} := 1\ ft + 6\ in$ *(minimum thickness of slab)*

$(t_{slab})_{max} := 2\ ft + 0\ in$ *(maximum thickness of slab)*

$t_{slab} := \frac{(t_{slab})_{min} + (t_{slab})_{max}}{2} = 21\ in$ *(average thickness of slab)*

Component Dimensions

ABUTMENT/BENT LOCATIONS				
location	type	x (ft)	N_p	t_s (in)
abutment 1	drilled pier	-	5	18
bent 1	drilled pier	53.33	5	18
bent 2	drilled pier	106.67	5	18
abutment 2	drilled pier	160.00	5	18

$x := \text{submatrix}\left(output, 1, n_i, n_{id}+1, n_{id}+1\right) \cdot ft$ *(distance from left abutment bearing to each pier/abutments)*

$N_p := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right)$ *(number of piles/caissons at each interior pier/abutments)*

$t_s := \text{submatrix}\left(output, 1, n_i, n_{id}+3, n_{id}+3\right) \cdot in$ *(thickness of slab at each interior pier/abutments)*

$(b_t)_{a1} := \frac{L_1}{2} = 30\ ft$ *(width of deck tributary to abutment 1)*

$(b_t)_{b1} := \frac{L_1}{2} + \frac{L_2}{2} = 60\ ft$ *(width of deck tributary to bent 1)*

$(b_t)_{b2} := \frac{L_2}{2} + \frac{L_3}{2} = 60\ ft$ *(width of deck tributary to bent 2)*

$(b_t)_{a1} := \frac{L_1}{2} = 30\ ft$ *(width of deck tributary to abutment 1)*

$(b_t)_i := \left[(b_t)_{a1} \quad (b_t)_{b1} \quad (b_t)_{b2} \quad (b_t)_{a1} \right]^T$ *(summary)*

Safety Barrier:

$(h_{rail})_{head} := 1\ ft + 2\ in$ *(height of kdot coral rail head)*

$(b_{rail})_{head} := 1\ ft + 0\ in$ *(width of kdot coral rail head)*

$(h_{rail})_{curb} := 0\ ft + 6\ in$ *(height of kdot coral rail curb)*

$(b_{rail})_{curb} := 1\ ft + 0\ in$ *(width of kdot coral rail curb)*

$(h_{rail})_{post} := 0\ ft + 7\ in$ *(height of kdot coral rail post)*

$(b_{rail})_{post} := 0\ ft + 10\ in$ *(width of kdot coral rail post)*

$(d_{rail})_{post} := 3\ ft + 0\ in$ *(width of kdot coral rail post)*

$(s_{rail})_{post} := 10\ ft + 0\ in$ *(spacing of kdot coral rail post center-to-center)*

Flood Wall Dimensions:

$L_{wall} := L_b = 180\ ft$ *(length of flood wall)*

$b_{wall} := 12\ in$ *(width of flood wall)*

$h_{wall} := 48\ in$ *(height of flood wall)*

Sidewalk:

$t_{sw} := 0\ in$ *(thickness of sidewalk)*

$b_{sw} := 0\ in$ *(width of sidewalk)*

Piers and Bent Cap:

PIER AND PIER CAP DIMENSIONS						
location	type	N_{shaft}	d_{shaft} (in)	L_{shaft} (ft)	b_{cap} (in)	h_{cap} (in)
abutment 1	drilled pier	5	36	26	30	60
bent 1	drilled pier	5	36	26	36	60
bent 2	drilled pier	5	36	26	36	60
abutment 2	drilled pier	5	36	26	30	60

$N_{shaft} := \text{submatrix}\left(output, 1, n_i, n_{id}+1, n_{id}+1\right)$ *(number caissons at intermediate piers/abutments)*

$d_{shaft} := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right) \cdot in$ *(diameter of caissons)*

$L_{shaft} := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right) \cdot in$ *(length of caissons)*

$b_{cap} := \text{submatrix}\left(output, 1, n_i, n_{id}+4, n_{id}+4\right) \cdot in$ *(width of bent caps)*

$h_{cap} := \text{submatrix}\left(output, 1, n_i, n_{id}+5, n_{id}+5\right) \cdot in$ *(gross height of bent caps including region integral with slab)*

$h'_{cap} := h_{cap} - t_s$ *(net height of bent caps excluding region integral with slab)*

$L_{cap} := b_{slab} - 1\ ft = 69\ ft$ *(length of pier caps)*

$A_{cap} := \overrightarrow{b_{cap} \cdot h'_{cap}}$ *(cross-sectional area of bent cap)*

$A_{shaft} := \dfrac{\pi \cdot d_{shaft}^{\ 2}}{4}$ *(cross-sectional area of shafts)*

CROSS-SECTIONAL AREAS DRILLED PIERS & BENT CAPS			
location	type	A_{cap} (ft^2)	A_{cap} (ft^2)
abutment 1	drilled pier	8.75	7.07
bent 1	drilled pier	10.50	7.07
bent 2	drilled pier	10.50	7.07
abutment 2	drilled pier	8.75	7.07

Elevations

$(EL_{TOD})_{A1} := 909\ ft$ *(top of deck at abutment 1)*

$(EL_{TOD})_{A2} := 910\ ft$ *(top of deck at abutment 2)*

$EL_{base} := 890\ ft$ *(elevation at bottom of footing/pile cap)*

$EL_{USR} := (EL_{TOD})_{A1} - t_{slab} = 907.3\ ft$ *(bottom of slab elevation, upstream right)*

$EL_{USL} := (EL_{TOD})_{A2} - t_{slab} = 908.3\ ft$ *(bottom of slab elevation, upstream left)*

$EL_{DSR} := (EL_{TOD})_{A1} - t_{slab} = 907.3\ ft$ *(bottom of slab elevation, downstream right)*

$EL_{DSL} := (EL_{TOD})_{A2} - t_{slab} = 908.3\ ft$ *(bottom of slab elevation, downstream left)*

$WSE_{USR} := 910.95\ ft$ *(water surface elevation, upstream right)*

$WSE_{USL} := 910.95\ ft$ *(water surface elevation, upstream left)*

$WSE_{DSR} := 910.63\ ft$ *(water surface elevation, downstream right)*

$WSE_{DSL} := 910.63\ ft$ *(water surface elevation, downstream left)*

$$EL_{US} := \frac{EL_{USR} + EL_{USL}}{2} = 907.8\ ft$$ *(average elevation at bottom of slab, upstream)*

$$WSE_{US} := \frac{WSE_{USR} + WSE_{USR}}{2} = 911\ ft$$ *(average water surface, upstream)*

$$EL_{DS} := \frac{EL_{DSR} + EL_{DSL}}{2} = 907.8\ ft$$ *(average elevation at bottom of slab, downstream)*

$$WSE_{DS} := \frac{WSE_{DSR} + WSE_{DSR}}{2} = 910.6\ ft$$ *(average water surface, downstream)*

Dimensions for Wind Analysis:

$L_{wind} := L_{wall} = 180\ ft$ *(length of exposed structure for wind loads; distance between joints)*

$h_{wind} := h_{wall} = 4\ ft$ *(height of exposed structure for wind loads)*

$EL_{base} = 890\ ft$ *(elevation of ground or water surface)*

$EL_{structure} := (EL_{TOD})_{A2} + h_{wall} = 914\ ft$ *(elevation of structure)*

$z := EL_{structure} - EL_{base} = 24\ ft$ *(measured structure height above ground or water surface)*

Concrete Component Volumes

$v_{slab} := b_{slab} \cdot t_{slab} = 122.5\ ft^3/ft$ *(unit volume of concrete slab)*

$v_{sw} := b_{sw} \cdot t_{sw} = 0\ ft^3/ft$ *(unit volume of concrete sidewalk)*

$v_{wall} := b_{wall} \cdot h_{wall} = 4\ ft^3/ft$ *(unit volume of floodwall)*

$v_{rail} := (h_{rail})_{head} \cdot (b_{rail})_{head} + \frac{(h_{rail})_{post} \cdot (b_{rail})_{post} \cdot (d_{rail})_{post}}{(s_{rail})_{post}} = 1.3\ ft^3/ft$ *(unit volume of kdot corral barrier)*

$v_{total} := v_{slab} + 2 \cdot \left(v_{sw} + v_{rail} + v_{wall}\right) = 133.1\ ft^3/ft$ *(total unit volume of concrete in bridge cross-section)*

$v_{cap} := \overrightarrow{A_{cap} \cdot L_{cap}}$ *(volume of pier caps at abutments/bents)*

$v_{shaft} := \overrightarrow{A_{shaft} \cdot L_{shaft}}$ *(volume of single shaft)*

$V_{shaft} := \overrightarrow{N_{shaft} \cdot v_{shaft}}$ *(total volume of shafts at abutments/bents)*

VOLUME OF CONCRETE IN PIERS & BENT CAPS

location	type	v_{cap} (ft^3)	v_{shaft} (ft^3)	V_{shaft} (ft^3)
abutment 1	drilled pier	603.75	21.21	106.03
bent 1	drilled pier	724.50	21.21	106.03
bent 2	drilled pier	724.50	21.21	106.03
abutment 2	drilled pier	603.75	21.21	106.03

Component Weights

$w_{slab} := v_{slab} \cdot \gamma_c = 18375\ plf$ *(unit weight of concrete slab)*

$w_{sw} := v_{sw} \cdot \gamma_c = 0\ plf$ *(unit weight of concrete sidewalk)*

$w_{wall} := v_{wall} \cdot \gamma_c = 600\ plf$ *(unit weight of concrete wall)*

$w_{rail} := v_{rail} \cdot \gamma_c = 196.9\ plf$ *(unit weight of kdot corral barrier)*

$2 \cdot \left(w_{wall} + w_{sw} + w_{rail}\right) = 1.594\ klf$

$w_{deck} := \frac{\overrightarrow{\left(w_{slab} + 2 \cdot \left(w_{sw} + w_{wall} + w_{rail}\right)\right) \cdot (b_t)_i}}{L_{cap}}$ *(total continuous dead load supported by bents)*

WEIGHT SUPPORTED BY ABUTMENTS/BENTS		
location	type	w_{deck} (klf)
abutment 1	drilled pier	8.68
bent 1	drilled pier	17.36
bent 2	drilled pier	17.36
abutment 2	drilled pier	8.68

$w_{shaft} := v_{shaft} \cdot \gamma_c$ *(weight of single shaft at supports)*

$(w_{DC})_{shaft} := \overrightarrow{N_{shaft} \cdot w_{shaft}}$ *(total weight of shafts at supports)*

WEIGHT OF DRILLED PIERS			
location	type	w_{shaft} (kip)	$(w_{DC})_{shaft}$ (kip)
abutment 1	drilled pier	0.97	15.90
bent 1	drilled pier	0.97	15.90
bent 2	drilled pier	0.97	15.90
abutment 2	drilled pier	0.97	15.90

$(w_{DC})_{deck} := v_{total} \cdot \gamma_c = 19.969\ klf$ *(total unit weight of dead load)*

$(w_{DC})_{slab} := \frac{v_{total} \cdot \gamma_c}{b_{slab}} = 285\ \frac{plf}{ft}$ *(unit weight of dead load per unit width of slab)*

$W_{cap} := v_{cap} \cdot \gamma_c$ *(weight of pier caps at supports)*

WEIGHT OF BENT CAPS		
location	type	W_{cap} (kip)
abutment 1	drilled pier	90.56
bent 1	drilled pier	108.68
bent 2	drilled pier	108.68
abutment 2	drilled pier	90.56

$(W_{DC})_{shaft} := \sum (w_{DC})_{shaft} = 63.617\ kip$ *(total weight of piers)*

$(W_{DC})_{cap} := \sum W_{cap} = 398.475\ kip$ *(total weight of pier caps)*

$(W_{DC})_{deck} := (w_{DC})_{deck} \cdot L_b + (W_{DC})_{cap} = 3992.85\ kip$ *(total weight of dead load at deck)*

$W_{DC} := (W_{DC})_{deck} + (W_{DC})_{shaft} = 4056.5\ kip$ *(total weight of dead load)*

$w_{DW} := 0\ plf$ *(continuous dead load, DW)*

Hydraulic Loads

Uplift:

$d_{US} := WSE_{US} - EL_{US} = 3.2\ ft$ *(depth of water at upstream side of bridge)*

$d_{DS} := WSE_{DS} - EL_{DS} = 2.88\ ft$ *(depth of water at downstream side of bridge)*

$d_{displaced} := \frac{d_{US} + d_{US}}{2} = 3.2\ ft$ *(depth of water displaced by bridge)*

$V_{displaced} := d_{displaced} \cdot b_{slab} \cdot L_b = 40320\ ft^3$ *(volume of displaced water)*

$F_{buoyant} := \gamma_w \cdot V_{displaced} = 2516\ kip$ *(buoyant force)*

$(FS_{uplift})_{deck} := \frac{(W_{DC})_{deck}}{F_{buoyant}} = 1.6$ *(uplift factor of safety, deck and pier caps only)*

$FS_{uplift} := \frac{W_{DC}}{F_{buoyant}} = 1.6$ *(uplift factor of safety, total)*

Drag:

$C_D := (C_D)_{flood.wall} = 1.4$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

$p_{wall} := \frac{C_D \cdot \gamma_w \cdot V_{US}^{\ 2}}{2 \cdot g} = 144\ psf$ *(pressure of flowing water on floodwall; LRFD Eq. C3.7.3.1-1)*

$C_D := (C_D)_{pier} = 0.7$ *(drag coefficient for square-ended pier; LRFD Table 3.7.3.1-1)*

$p_{pier} := \frac{C_D \cdot \gamma_w \cdot V_{US}^{\ 2}}{2 \cdot g} = 72\ psf$ *(pressure of flowing water on bridge caisson; LRFD Eq. C3.7.3.1-1)*

$(w_{WA})_{deck} := p_{wall} \cdot h_{wall} = 0.58\ klf$ *(continuous load on bridge at deck)*

$(P_{WA})_i := \overrightarrow{(w_{WA})_{deck} \cdot (b_t)_i}$ *(total water load on abutments & piers)*

WATER LOAD ON ABUTMENTS/BETS

location	type	P_{WA} (kip)
abutment 1	drilled pier	17.28
bent 1	drilled pier	34.57
bent 2	drilled pier	34.57
abutment 2	drilled pier	17.28

Braking Load

$L_{brake} := L_b = 180\ ft$ *(length of lane load braking distance)*

$W_{TLL} := 8\ kip + 32\ kip + 32\ kip = 72\ kip$ *(weight of design truck)*

$W_{LLL} := (0.64\ klf) \cdot L_{brake} = 115.2\ kip$ *(total weight of lane load)*

$n_L = 2$ *(number lanes with traffic in the same direction)*

$F_{BR} := n_L \cdot (\max(25\% \cdot W_{TLL}, 5\% \cdot (W_{TLL} + W_{LLL}))) = 36.0\ kip$ *(total braking load; LRFD 3.6.4)*

Bearing Reactions Due to Dead & Live Loads

Dead Loads:

DEAD LOADS			
location	type	R_{DC} (kip)	R_{DW} (kip)
abutment 1	drilled pier	466.15	-
bent 1	drilled pier	1281.37	-
bent 2	drilled pier	1281.37	-
abutment 2	drilled pier	466.15	-

$R_{DC} := \text{submatrix}(output, 1, n_i, n_{id}+1, n_{id}+1) \cdot kip$ *(reaction due to design truck/tandem)*

$R_{DW} := \text{submatrix}(output, 1, n_i, n_{id}+2, n_{id}+2) \cdot kip$ *(reaction due to design lane load)*

Live Loads:

VEHICULAR LIVE LOADS AT ABUMENTS & PIERS			
location	type	R_{TTL} (kip)	R_{LLL} (kip)
abutment 1	drilled pier	56.52	15.36
bent 1	drilled pier	68.86	40.93
bent 2	drilled pier	68.86	40.93
abutment 2	drilled pier	56.52	15.36

$R_{TLL} := \text{submatrix}(output, 1, n_i, n_{id}+1, n_{id}+1) \cdot kip$ *(reaction due to design truck/tandem)*

$R_{LLL} := \text{submatrix}(output, 1, n_i, n_{id}+2, n_{id}+2) \cdot kip$ *(reaction due to design lane load)*

$R_{LL} := \overrightarrow{g_{dist} \cdot (R_{TLL} \cdot IM + R_{LLL})}$ *(total reaction due to live loads)*

LIVE LOADS AT ABUMENTS & PIERS		
location	type	R_{LL} (kip)
abutment 1	drilled pier	223.61
bent 1	drilled pier	327.31
bent 2	drilled pier	327.31
abutment 2	drilled pier	223.61

Wind Loads

$roughness = \text{“d”}$ *(ground surface roughness; LRFD 3.8.1.1.4)*

$wind.exposure = \text{“d”}$ *(wind exposure category; LRFD 3.8.1.1.5)*

$V_{III} = 115\ mph$ *(design 3-second gust wind speed for Strength III; LRFD Table 3.8.1.1.2-1)*

$G = 1$ *(gust effect factor; LRFD 3.8.1.2.1)*

$C_D := (C_D)_W = 1.3$ *(gust effect factor; LRFD Table 3.8.1.2.1-2)*

$Z := \max(z, 33\ ft) = 33\ ft$ *(structure design height)*

$list.exposure$	$list.coefficient$
“b”	$\dfrac{\left(2.5 \cdot ln\left(\frac{Z \cdot ft^{-1}}{0.9834}\right) + 6.87\right)^2}{345.6}$
“c”	$\dfrac{\left(2.5 \cdot ln\left(\frac{Z \cdot ft^{-1}}{0.0984}\right) + 7.35\right)^2}{478.4}$
“d”	$\dfrac{\left(2.5 \cdot ln\left(\frac{Z \cdot ft^{-1}}{0.0164}\right) + 7.65\right)^2}{616.1}$

$table_{Kz} := \text{augment}(list.exposure, list.coefficient)$

$(K_z)_{strengthIII\&serviceIV} := \text{vlookup}(wind.exposure, table_{Kz}, 2)_1$ *(pressure exposure and elevation coefficient for Strength III and Service IV; LRFD Eqs. 3.8.1.2.1-2, 3 & 4)*

$(K_z)_{strengthIII\&serviceIV} = 1.2$

3-SECOND GUST WIND SPEED AND PRESSURE EXPOSURE & ELEVATION COEFFICIENT

load combination	V (mph)	K_z
strength III	115.00	1.15
strength V	80.00	1.00
service I	70.00	1.00
service IV	86.25	1.15

$wind := table.input$

$P_Z := \overrightarrow{\left(2.56 \cdot 10^{-6}\right) \cdot \left(V \cdot mph^{-1}\right)^2 \cdot K_z \cdot G \cdot C_D \cdot ksf}$ *(design wind pressure; LRFD Eq. 3.8.1.2.1-1)*

$(P_Z)_{III} := P_{Z_1} = 50.8\ psf$ *(design wind pressure; Strength III)*

$(P_Z)_V := P_{Z_2} = 21.3\ psf$ *(design wind pressure; Strength V)*

$(w_{WS})_z := P_Z \cdot h_{wind}$ *(wind on structure, transverse; LRFD 3.8.1.2.3a)*

$(w_{WS})_x := 25\% \cdot (w_{WS})_z$ *(wind on structure, longitudinal; LRFD 3.8.1.2.3a)*

$table.wind := \text{augment}\left(table.input^{\langle 1 \rangle}, (w_{WS})_z, (w_{WS})_x\right)$

$(w_{WS})_{x.III} := (w_{WS})_{x_1} = 0.1\ klf$ *(wind on structure, longitudinal; Strength III)*

$(w_{WS})_{x.V} := (w_{WS})_{x_2} = 0\ klf$ *(wind on structure, longitudinal; Strength V)*

$(w_{WL})_z := 0.10\ klf$ *(wind on live load, transverse; LRFD 3.8.1.3)*

$(w_{WL})_x := 0.04\ klf$ *(wind on live load, longitudinal; LRFD 3.8.1.3)*

Seismic Loads

$PGA = 3.5\%$ *(peak seismic ground acceleration coefficient on rock; LRFD Figure 3.10.2.1-1)*

$S_S = 7.6\%$ *(horizontal response spectral acceleration coefficient at 0.2-s period on rock - site class b; LRFD Figure 3.10.2.1-2)*

$S_1 = 3.7\%$ *(horizontal response spectral acceleration coefficient at 1-s period on rock - site class b; LRFD Figure 3.10.2.1-3)*

$site.class =$ "d" *(site classification)*

AASHTO LRFD Table 3.10.3.2-1 Site Factor, F_{pga}

site class	peak ground acceleration coeficient (PGA)				
	PGA < 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA > 0.50
a	0.8	0.8	0.8	0.8	0.8
b	1.0	1.0	1.0	1.0	1.0
c	1.2	1.2	1.1	1.0	1.0
d	1.6	1.4	1.2	1.1	1.0
e	2.5	1.7	1.2	0.9	0.9
f	site-specific geotechnical investigation required				

$F_{pga} = 1.60$ *(site factor; LRFD Table 3.10.3.2-1)*

$A_S := F_{pga} \cdot PGA = 0.1$

AASHTO LRFD Table 3.10.3.2-2 Site Factor, F_a

site class	spectral acceleration coefficient at period 0.2 sec (S_S)				
	$S_S < 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.0$	$S_S > 1.25$
a	0.8	0.8	0.8	0.8	0.8
b	1.0	1.0	1.0	1.0	1.0
c	1.2	1.2	1.1	1.0	1.0
d	1.6	1.4	1.2	1.1	1.0
e	2.5	1.7	1.2	0.9	0.9
f	site-specific geotechnical investigation required				

$F_a = 1.60$ *(site factor for short-period range of acceleration response spectrum; LRFD Table 3.10.3.2-2)*

AASHTO LRFD Table 3.10.3.2-3 Site Factor, F_v

site class	spectral acceleration coefficient at period 1 sec (S_1)				
	$S_1 < 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 > 0.5$
a	0.8	0.8	0.8	0.8	0.8
b	1.0	1.0	1.0	1.0	1.0
c	1.7	1.6	1.5	1.4	1.3
d	2.4	2.0	1.8	1.6	1.5
e	3.5	3.2	2.8	2.4	2.4
f	site-specific geotechnical investigation required				

$F_v = 2.40$ *(site factor for long-period range of acceleration response spectrum; LRFD Table 3.10.3.2-2)*

$S_{DS} := F_a \cdot S_S = 0.1$ *(horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor; LRFD Eq. 3.10.4.2-2)*

$S_{D1} := F_v \cdot S_1 = 0.1$ *(horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor; LRFD Eq. 3.10.4.2-6)*

AASHTO TABLE 3.10.6-1 - SEISMIC ZONES	
acceleration coefficient, S_{D1}	seismic zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

$seismic.zone = 1$

$$\gamma_{EQ} := \begin{cases} \text{if } A_S < 0.05 \\ \quad 0.15 \\ \text{else} \\ \quad 0.25 \end{cases}$$ *(LRFD 3.10.9.2)*

$\gamma_{EQ} = 0.3$

$w_{EQ} := \overrightarrow{\gamma_{EQ} \cdot \left((w_{DC})_{deck} + w_{DW} \right)} = 4.99 \ klf$

Design Loads

Vertical Loads:

$P_{DC} := R_{DC}$ *(design dead load, DC)*

$P_{DW} := R_{DW}$ *(design dead load, DW)*

$P_{LL} := R_{LL}$ *(design live load)*

Lateral Loads in Transverse Direction:

$(w_{EQ})_z := w_{EQ} = 5\ klf$ *(seismic load, transverse direction)*

$(w_{WA})_z := (w_{WA})_{deck} = 0.576\ klf$ *(water load, transverse direction)*

Lateral Loads in Longitudinal Direction:

$(w_{EQ})_x := w_{EQ} = 5\ klf$ *(seismic load, longitudinal direction)*

$(w_{BR})_x := \frac{F_{BR}}{L_b} = 0.2\ klf$ *(braking load)*

LOAD COMBINATION FACTORS										
limit state	type	η	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	strength	1.00	1.25	1.50	1.75	1.75	1.00	-	-	-
strength III	strength	1.00	1.25	1.50	-	-	1.00	1.00	-	-
strength V	strength	1.00	1.25	1.50	1.35	1.35	1.00	1.00	1.00	-
extreme event I	extreme event	1.00	1.25	1.50	1.00	1.00	1.00	-	-	1.00
service I	service	1.00	1.25	1.50	1.00	1.00	1.00	1.00	1.00	-

$limit.state := \text{submatrix}\left(output, 1, n_i, 1, 1\right)$

$limit.state.type := \text{submatrix}\left(output, 1, n_i, 2, 2\right)$

$\eta := \text{submatrix}\left(output, 1, n_i, n_{id}+2, n_{id}+2\right)$

$\gamma := \text{submatrix}\left(output, 1, n_i, n_{id}+3, n_{oj}+1\right)$

$Q_{DC} := /f/conform\left(P_{DC}, location\right)$

$Q_{DW} := /f/conform\left(P_{DW}, location\right)$

$Q_{LL} := /f/conform\left(P_{LL}, location\right)$

$Q_{BR} := /f/conform\left(n/a, location\right)$

$Q_{WA} := /f/conform\left(n/a, location\right)$

$Q_{WS} := /f/conform\left(n/a, location\right)$

$Q_{WL} := /f/conform\left(n/a, location\right)$

$Q_{EQ} := /f/conform\left(n/a, location\right)$

$gravity := \left[Q_{DC}\ \ Q_{DW}\ \ Q_{LL}\ \ Q_{BR}\ \ Q_{WA}\ \ Q_{WS}\ \ Q_{WL}\ \ Q_{EQ}\right]^{T}$

$$Q := \left\| \begin{array}{l} m_0 \leftarrow gravity \\ \text{for } k \in 1 .. n_k \\ \quad \left\| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(\eta) \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. \text{cols}(\gamma) \\ \quad \left\| \begin{array}{l} m_{i,j} \leftarrow \left(m_{0_j}\right)_k \end{array} \right. \\ Q_k \leftarrow m \end{array} \right. \\ Q \end{array} \right.$$

$$\eta\gamma P := \left\| \begin{array}{l} \text{for } k \in 1 .. n_k \\ \quad \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. n_j \\ \quad \left\| \begin{array}{l} m_{i,j} \leftarrow \eta_i \cdot \gamma_{i,j} \cdot \left(Q_k\right)_{i,j} \end{array} \right. \\ \eta\gamma P_k \leftarrow m \end{array} \right. \\ \eta\gamma P \end{array} \right.$$

$$\Sigma\eta\gamma P := \left\| \begin{array}{l} \text{for } k \in 1 .. n_k \\ \quad \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. n_j \\ \quad \left\| \begin{array}{l} m_i \leftarrow \sum_{j=1}^{j} \left(\eta\gamma P_k\right)_{i,j} \end{array} \right. \\ \Sigma\eta\gamma P_k \leftarrow m \end{array} \right. \\ \Sigma\eta\gamma P \end{array} \right.$$

$$/f/controlling\left(v_0, type\right) := \left\| \begin{array}{l} \text{for } i \in 1 .. \text{rows}\left(v_0\right) \\ \quad \left\| \begin{array}{l} v_i \leftarrow \left\| \begin{array}{l} \text{if } limit.state.type_i = type \\ \quad \left\| v_{0_i} \right. \\ \text{else} \\ \quad \left\| 0 \right. \end{array} \right. \end{array} \right. \\ \max(v) \end{array} \right.$$

FACTORED GRAVITY LOADS - ABUTMENT 1									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	974.01	582.69	-	391.32	-	-	-	-	-
strength III	582.69	582.69	-	-	-	-	-	-	-
strength V	884.57	582.69	-	301.88	-	-	-	-	-
extreme event I	806.30	582.69	-	223.61	-	-	-	-	-
service I	806.30	582.69	-	223.61	-	-	-	-	-

$(P_{strength})_{total.1} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 974\ kip$ *(controlling total factored strength gravity loads at abutment 1)*

$(P_{serv})_{total.1} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 806.3\ kip$ *(controlling total factored service gravity loads at abutment 1)*

FACTORED GRAVITY LOADS - PIER 1									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	2174.51	1601.72	-	572.79	-	-	-	-	-
strength III	1601.72	1601.72	-	-	-	-	-	-	-
strength V	2043.58	1601.72	-	441.87	-	-	-	-	-
extreme event I	1929.03	1601.72	-	327.31	-	-	-	-	-
service I	1929.03	1601.72	-	327.31	-	-	-	-	-

$(P_{strength})_{total.2} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 2174.5\ kip$ *(controlling total factored strength gravity loads at bent 1)*

$(P_{serv})_{total.2} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 1929\ kip$ *(controlling total factored service gravity loads at bent 1)*

FACTORED GRAVITY LOADS - PIER 2									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	2174.51	1601.72	-	572.79	-	-	-	-	-
strength III	1601.72	1601.72	-	-	-	-	-	-	-
strength V	2043.58	1601.72	-	441.87	-	-	-	-	-
extreme event I	1929.03	1601.72	-	327.31	-	-	-	-	-
service I	1929.03	1601.72	-	327.31	-	-	-	-	-

$(P_{strength})_{total.3} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 2174.5\ kip$ *(controlling total factored strength gravity loads at bent 2)*

$(P_{serv})_{total.3} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 1929\ kip$ *(controlling total factored service gravity load at bent 2)*

FACTORED GRAVITY LOADS - ABUTMENT 2									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	974.01	582.69	-	391.32	-	-	-	-	-
strength III	582.69	582.69	-	-	-	-	-	-	-
strength V	884.57	582.69	-	301.88	-	-	-	-	-
extreme event I	806.30	582.69	-	223.61	-	-	-	-	-
service I	806.30	582.69	-	223.61	-	-	-	-	-

$(P_{strength})_{total.4} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“strength”}\right) = 974\ kip$ *(controlling total factored strength gravity loads at abutment 2)*

$(P_{serv})_{total.4} := /f/controlling\left(\Sigma\eta\gamma P_k, \text{“service”}\right) = 806.3\ kip$ *(controlling total factored service gravity load at abutment 2)*

$(P_{strength})_{total} := \left[(P_{strength})_{total.1} \quad (P_{strength})_{total.2} \quad (P_{strength})_{total.3} \quad (P_{strength})_{total.4} \right]^{T}$

(summary of controlling total factored strength gravity loads)

$P_{strength} := \frac{(P_{strength})_{total}}{N_{shaft}}$ *(factored service gravity loads on each caisson)*

$(P_{serv})_{total} := \left[(P_{serv})_{total.1} \quad (P_{serv})_{total.2} \quad (P_{serv})_{total.3} \quad (P_{serv})_{total.4} \right]^{T}$ *(summary of controlling total factored service gravity loads)*

$P_{serv} := \frac{(P_{serv})_{total}}{N_{shaft}}$ *(factored service gravity loads on each caisson)*

CONTROLLING GRAVITY LOADS AT ABUMENTS & PIERS

location	type	$(P_{serv})_{total}$ (kip)	P_{serv} (kip)	$(P_{strength})_{total}$ (kip)	$P_{strength}$ (kip)
abutment 1	drilled pier	806.3	161.3	974.0	194.8
bent 1	drilled pier	1929.0	385.8	2174.5	434.9
bent 2	drilled pier	1929.0	385.8	2174.5	434.9
abutment 2	drilled pier	806.3	161.3	974.0	194.8

$Q_{DC} := /f/conform(n/a, limit.state)$

$Q_{DW} := /f/conform(n/a, limit.state)$

$Q_{LL} := /f/conform(n/a, limit.state)$

$Q_{BR} := /f/conform(n/a, limit.state)$

$Q_{WA} := /f/conform((w_{WA})_z, limit.state)$

$$Q_{WS} := \left\| \begin{array}{l} \text{for } i \in 1..n_i \\ \quad w_{WS_i} \leftarrow \left\| \begin{array}{l} \text{try} \\ \quad \text{vlookup}\left(limit.state_i, table.wind, 2\right)_1 \\ \text{on error} \\ \quad n/a \end{array} \right. \\ w_{WS} \end{array} \right.$$

$Q_{WL} := /f/conform((w_{WL})_z, limit.state)$

$Q_{EQ} := /f/conform((w_{EQ})_z, limit.state)$

$Q := \text{augment}(Q_{DC}, Q_{DW}, Q_{LL}, Q_{BR}, Q_{WA}, Q_{WS}, Q_{WL}, Q_{EQ})$

$$(\eta\gamma R)_z := \left\| \begin{array}{l} \text{for } i \in 1..n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1..n_j \\ \quad \left\| \eta\gamma R_{z_{i,j}} \leftarrow \eta_i \cdot \gamma_{i,j} \cdot Q_{i,j} \right\| \end{array} \right\| \\ \eta\gamma R_z \end{array} \right\|$$

$$\Sigma\eta\gamma R_z := \left\| \begin{array}{l} \text{for } i \in 1..n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1..n_j \\ \quad \left\| \Sigma\eta\gamma R_{z_i} \leftarrow \sum_{j=1}^{j} (\eta\gamma R)_{z_{i,j}} \right\| \end{array} \right\| \\ \Sigma\eta\gamma R_z \end{array} \right\|$$

FACTORED LATERAL LOADS: TRANSVERSE DIRECTION (KLF)									
limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	0.576	-	-	-	-	0.576	-	-	-
strength III	0.779	-	-	-	-	0.576	0.203	-	-
strength V	0.761	-	-	-	-	0.576	0.085	0.100	-
extreme event I	5.568	-	-	-	-	0.576	-	-	4.992
service I	0.741	-	-	-	-	0.576	0.065	0.100	-

$(R_{strength})_z := /f/controlling\left(\Sigma\eta\gamma R_z, \text{“strength”}\right) = 0.8\ klf$ *(controlling total factored strength transverse lateral load)*

$(R_{extreme})_z := /f/controlling\left(\Sigma\eta\gamma R_z, \text{“extreme event”}\right) = 5.6\ klf$ *(controlling total factored extreme event transverse lateral load)*

$(R_{serv})_z := /f/controlling\left(\Sigma\eta\gamma R_z, \text{“service”}\right) = 0.7\ klf$ *(controlling total factored service transverse lateral load)*

$(R_u)_z := \max\left((R_{strength})_z, (R_{extreme})_z\right) = 5.6\ klf$ *(transverse lateral design load)*

$(V_u)_{z.total} := (R_u)_z \cdot (b_t)_i$ *(transverse lateral design lateral loads at each support)*

$(V_u)_z := \dfrac{(V_u)_{z.total}}{N_{shaft}}$ *(transverse lateral design lateral load at each caisson)*

TRANSVERSE DESIGN LATERAL LOADS AT ABUMENTS & PIERS			
location	type	$(V_u)_{z.total}$ (kip)	$(V_u)_z$ (kip)
abutment 1	drilled pier	167.0	33.4
bent 1	drilled pier	334.1	66.8
bent 2	drilled pier	334.1	66.8
abutment 2	drilled pier	167.0	33.4

$Q_{WA} := /f/conform\left(n/a, limit.state\right)$

$$Q_{WS} := \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} w_{WS_i} \leftarrow \left\| \begin{array}{l} \text{try} \\ \quad \text{vlookup}\left(limit.state_i, table.wind, 3\right)_1 \\ \text{on error} \\ \quad n/a \end{array} \right. \end{array} \right. \\ w_{WS} \end{array} \right.$$

$Q_{WL} := /f/conform\left((w_{WL})_x, limit.state\right)$

$Q_{EQ} := /f/conform\left((w_{EQ})_x, limit.state\right)$

$Q := \text{augment}\left(Q_{DC}, Q_{DW}, Q_{LL}, Q_{BR}, Q_{WA}, Q_{WS}, Q_{WL}, Q_{EQ}\right)$

$$(\eta\gamma R)_x := \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. n_j \\ \quad \left\| \eta\gamma R_{x_{i,j}} \leftarrow \eta_i \cdot \gamma_{i,j} \cdot Q_{i,j} \right. \end{array} \right. \\ \eta\gamma R_x \end{array} \right.$$

$$\Sigma\eta\gamma R_x := \left\| \begin{array}{l} \text{for } i \in 1 .. n_i \\ \quad \left\| \begin{array}{l} \text{for } j \in 1 .. n_j \\ \quad \left\| \Sigma\eta\gamma R_{x_i} \leftarrow \sum_{j=1}^{j} (\eta\gamma R)_{x_{i,j}} \right. \end{array} \right. \\ \Sigma\eta\gamma R_x \end{array} \right.$$

FACTORED LATERAL LOADS: LONGITUDINAL TRANSVERSE DIRECTION (KLF)

limit state	Σ	DC	DW	LL	BR	WA	WS	WL	EQ
strength I	-	-	-	-	-	-	-	-	-
strength III	0.05	-	-	-	-	-	0.051	-	-
strength V	0.06	-	-	-	-	-	0.021	0.040	-
extreme event I	4.99	-	-	-	-	-	-	-	4.99
service I	0.06	-	-	-	-	-	0.016	0.040	-

$(R_{strength})_x := /f/controlling\left(\Sigma\eta\gamma R_x, \text{“strength”}\right) = 0.1\ klf$ *(controlling total factored strength transverse lateral load)*

$(R_{extreme})_x := /f/controlling\left(\Sigma\eta\gamma R_x, \text{“extreme event”}\right) = 5\ klf$ *(controlling total factored extreme event transverse lateral load)*

$(R_{serv})_x := /f/controlling\left(\Sigma\eta\gamma R_x, \text{“service”}\right) = 0.1\ klf$ *(controlling total factored service transverse lateral load)*

$(R_u)_x := \max\left((R_{strength})_x, (R_{extreme})_x\right) = 5\ klf$ *(transverse lateral design load)*

$(V_u)_{x.total} := (R_u)_x \cdot (b_t)_i$ *(transverse lateral design lateral loads at each support)*

$(V_u)_x := \frac{(V_u)_{x.total}}{N_{shaft}}$ *(transverse lateral design lateral load at each caisson)*

LONGITUDINAL DESIGN LATERAL LOADS AT ABUMENTS & PIERS			
location	type	$(V_u)_{x.total}$ (kip)	$(V_u)_x$ (kip)
abutment 1	drilled pier	149.8	30.0
bent 1	drilled pier	299.5	59.9
bent 2	drilled pier	299.5	59.9
abutment 2	drilled pier	149.8	30.0

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Merriam Drive Bridge Design

UPPER TURKEY CREEK

MERRIAM, KS

GENERAL

National Bridge Inventory (NBI) Data

condition.rating := "9 – excellent condition" *(condition rating of superstructure; SI&A Data, NBI Item 59)*

Deck Information

concrete := "normal weight" *(type of concrete weight)*

type := "grade 60 – 60 ksi" *(type of non-prestressed reinforcing steel)*

concrete.strength := "5.0 ksi" *(compressive strength of concrete)*

wearing.surface := "bituminous" *(type of wearing surface material)*

Material Properties

Reinforcement Strengths:

MATERIAL STRENGTH OF REINFORCING STEEL		
type of reinforcing steel	f_y (ksi)	f_u (ksi)
structural grade - 33 ksi	33	55
intermediate grade - 40 ksi	40	70
rail/hard grade - 50 ksi	50	80
grade 60 - 60 ksi	***60***	***90***
not provided - constructed prior to 1954	33	55
not provided - constructed during or after 1954	40	70

$f_y = 60\ ksi$ *(yield strength of reinforcing steel; MBE Table 6A.5.2.2-1)*

$f_u = 90\ ksi$ *(ultimate strength of reinforcing steel; CRSI)*

Concrete Strengths:

$f'_c = 5\ ksi$ *(compressive strength of concrete; MBE Table 6A.5.2.1-1)*

f_{ct} := "not specified" *(splitting tensile strength; enter "not specified" if not available)*

Unit Weight of Materials:

$$(\gamma_c)_{nw} := \begin{cases} \text{if } f'_c \le 5.0\ ksi \\ \quad 145\ pcf \\ \text{else if } 5.0\ ksi < f'_c \le 15.0\ ksi \\ \quad 140\ pcf + (0.001\ kcf) \cdot \left(f'_c \cdot ksi^{-1}\right) \end{cases}$$

(unit weight of normal weight concrete; LRFD Table 3.5.1-1)

$(\gamma_c)_{nw} = 145\ pcf$

UNIT WEIGHT OF CONCRETE	
type	$(\gamma_c)_y$ (pcf)
light weight	110
sand-light weight	120
normal weight	***145***

$(\gamma_c)_p = 145\ pcf$ *(unit weight of plain concrete; LRFD Table 3.5.1-1)*

$\gamma_c := (\gamma_c)_p + 5\ pcf = 150\ pcf$ *(unit weight of reinforced concrete; LRFD Table 3.5.1-1 & C3.5.1)*

UNIT WEIGHT OF WEARING SURFACE	
type	γ_{ws} (pcf)
bituminous	***140***
concrete	145
timber - hard wood	60
timber - soft wood	50
gravel	140
none	-

$\gamma_{ws} = 140\ pcf$ *(unit weight of wearing surface; LRFD Table 3.5.1-1)*

Modulus of Elasticity:

$E_s := 29000\ ksi$ *(modulus of elasticity of steel; LRFD 5.4.3.2)*

DESIGN FACTORS

Load and Resistance Factors

Strength I Load Factors:

$\gamma_{DC} := 1.25$ *(dead load factor, DC for strength I; MBE Table 6A.4.2.2-1 & LRFR Table 3.4.1-2)*

$\gamma_{DW} := 1.50$ *(maximum dead load factor, DW for strength I; MBE Table 6A.4.2.2-1 & LRFR Table 3.4.1-2)*

$(\gamma_L)_{inv} := 1.75$ *(live load factor for strength I, inventory limit state; MBE Table 6A.4.2.2-1)*

$(\gamma_L)_{op} := 1.35$ *(live load factor for strength I, operating limit state; MBE Table 6A.4.2.2-1)*

Resistance Factors:

$(\varphi_f)_t := 0.90$ *(flexural resistance factor for tension controlled sections; LRFD 5.5.4.2)*

$(\varphi_f)_c := 0.75$ *(flexural resistance factor for compression controlled sections; LRFD 5.5.4.2)*

$\varphi_c :=$ if $condition.rating \geq 6$ → 1.0; else if $condition.rating = 5$ → 0.95; else → 0.85 *(condition factor; MBE Table 6A.4.2.3-1)*

$\varphi_c = 1.00$

$\varphi_s := 1.00$ *(system factor, positive flexure; TCX Guidance)*

Other Factors

$IM := 1.33$ *(impact factor; MBE 6A.4.4.3)*

$\lambda :=$ if $f_{ct} =$ "not specified" → $\max\left(0.75, min\left(7.5 \cdot (\gamma_c)_p \cdot kcf^{-1}, 1.0\right)\right)$; else → $min\left(4.7 \cdot \frac{f_{ct}}{\sqrt{f'_c}}, 1.0\right)$ *(concrete density modification factor; MBE 6A.4.4.3)*

$\lambda = 1.0$

Member Dimensions

$L := \frac{180\ ft}{3} = 60\ ft$ *(length of span)*

$W := 70\ ft$ *(edge-to-edge width)*

$S := \frac{180\ ft}{3}$ *(spacing of bent caps)*

$b_{clr} := 70\ ft - 2\ ft - 10\ ft - 2\ ft = 56\ ft$ *(clear distance between curbs)*

$b := 12\ in$ *(unit width of concrete deck in direction of traffic)*

$(t_{slab})_{min} := 1\ ft + 6\ in$ *(thickness of concrete slab at midspan)*

$(t_{slab})_{max} := 2\ ft + 0\ in$ *(thickness of concrete slab at bents)*

$t := (t_{slab})_{min} = 18\ in$ *(thickness of concrete deck at midspan)*

$t_{ws} := 0\ in$ *(thickness of asphalt overlay)*

$cover_t := 2\ in$ *(concrete cover over top reinforcing bars in slab)*

$cover_b := 1\ in$ *(concrete cover over bottom reinforcing bars in slab)*

$cover := cover_t - t_{ws} = 2\ in$

$S_x := \frac{b \cdot t^2}{6}$ *(section modulus)*

$x_1 := 1\ ft + 5\ ft + 1\ ft = 7\ ft$ *(distance between the edge of the deck and the inside face of the downstream barrier at monolith 3)*

Design Lanes:

$w := min\left(b_{clr}\right) = 56\ ft$ *(clear roadway between curbs; LRFD 3.6.1.1.1)*

$N_L := \mathbf{if}\left(w \geq 18\ ft, \max\left(2, \mathrm{trunc}\left(\frac{w}{12\ ft}\right)\right), 1\right)$ *(number of design lanes; LRFD 3.6.1.1.1 & MBE 6A.2.3.2)*

$N_L = 4$

$b_L := \mathbf{if}\left(w \geq 18\ ft, min\left(\frac{w}{2}, 12\ ft\right), min\left(w, 12\ ft\right)\right)$ *(width of design lanes; LRFD 3.6.1.1.1 & MBE 6A.2.3.2)*

$b_L = 12\ ft$

Reinforcement Properties

$bar_b :=$ "#10" *(size of bottom bars)*

$bar_t :=$ "#10" *(size of top bars)*

$bar := bar_b$ *(size of flexural bars)*

$d_b = 1.27\ in$ *(diameter of bottom bars)*

$d_t = 1.27\ in$ *(diameter of top bars)*

$d = 1.27\ in$ *(diameter of flexural bars)*

$a_s := \frac{\pi \cdot d^2}{4} = 1.267\ in^2$ *(cross-sectional area of bar)*

$s_s := 4.75\ in$ *(spacing of reinforcing bars)*

$n := 1$ *(number of bars in section)*

$A_s := n \cdot \left(\frac{a_s}{s_s}\right) \cdot b = 3.2\ in^2$ *(total cross-sectional area of positive flexural reinforcing steel)*

$\bar{y}_s := cover + \frac{d}{2} = 2.64\ in$ *(location of centroid of flexural reinforcing steel)*

$d_s := t - \bar{y}_s = 15.37\ in$ *(distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement)*

LIVE LOAD DISTRIBUTION

$L_1 := min(L, 60\ ft) = 60\ ft$ *(modified span length)*

$W_{1.s} := min(W, 30\ ft) = 30\ ft$ *(modified edge-to-edge width for single-lane loading)*

$W_{1.m} := min(W, 60\ ft) = 60\ ft$ *(modified edge-to-edge width for multilane loading)*

Effective Strip Width (Interior Strip)

$E_s := \left(10 + 5 \cdot \sqrt{L_1 \cdot W_{1.s} \cdot ft^{-2}}\right) \cdot in = 222\ in$ *(equivalent width for single-lane loading; LRFD Eq. 4.6.2.3-1)*

$E_m := min\left(\left(84 + 1.44 \cdot \sqrt{L_1 \cdot W_{1.m} \cdot ft^{-2}}\right) \cdot in, \frac{12 \cdot W}{N_L}\right) = 170\ in$ *(equivalent width for multilane loading; LRFD Eq. 4.6.2.3-2)*

$E_1 := min(E_s, E_m) = 170\ in$ *(effective strip width for interior strips)*

Effective Strip Width (Edge Strip)

$E_2 := min(x_1 + 0.25 \cdot E_s + 12\ in, 0.5 \cdot E_s, 72\ in)$

$E_2 = 72\ in$ *(effective edge strip width - LRFD Eq. 4.6.2.1.4b)*

Lane Load Reduction for Edge Strip

LRFD Design Article 4.6.2.1.4b states that "edge beams shall be assumed to support one line of wheels and, where appropriate, a tributary portion of the design lane load." For this reason, the live load effects due to design truck loading will be reduced by 50% when determining the total live load effects for the edge strip in the calculations below. The live load effects due to design lane loading will be reduced in proportion to the width of lane load tributary to the edge strip.

$x_{trib} := \max\left(min\left(E_2 - x_1, 10\ ft\right), 0\ ft\right) = 0\ ft$ *(portion of design lane load tributary to the edge strip)*

$R_2 := \frac{x_{trib}}{10\ ft} = 0$ *(reduction factor for lane load tributary to edge strip)*

$(b_{eq})_{pos} := \left(26 + 6.6 \cdot S \cdot ft^{-1}\right) \cdot in = 35.17\ ft$ *(width of primary strip for positive flexure; LRFD Table 4.6.2.1.3-1)*

$(b_{eq})_{neg} := \left(48 + 3.0 \cdot S \cdot ft^{-1}\right) \cdot in = 19\ ft$ *(width of primary strip for negative flexure; LRFD Table 4.6.2.1.3-1)*

$b_{eq} := (b_{eq})_{pos} = 35.17\ ft$ *(controlling width of primary strip)*

LOAD EFFECTS

$(M_{DC})_{total} := 3708\ kip \cdot ft$ *(moment due to dead load, total cross-section)*

$M_{DC} := \left(\frac{(M_{DC})_{total}}{W}\right) \cdot b = 53\ kip \cdot ft$ *(moment due to dead load, effective width)*

$M_{DW} := 0\ kip \cdot ft$

$M_{LLL} := 212\ kip \cdot ft$

$M_{TLL} := 585\ kip \cdot ft$

$(M_{LL})_1 := \frac{\left(M_{TLL} \cdot IM + M_{LLL}\right) \cdot b}{E_1} = 69.7\ kip \cdot ft$ *(total live load moment at cross-section for interior strips)*

$(M_{LL})_2 := \frac{\left(50\% \cdot M_{TLL} \cdot IM + R_2 \cdot M_{LLL}\right) \cdot b}{E_2} = 64.8\ kip \cdot ft$ *(total live load moment at cross-section for edge strips)*

$M_{LL} := \max\left((M_{LL})_1, (M_{LL})_2\right) = 69.7\ kip \cdot ft$ *(controlling total live load moment at cross-section)*

$M_u := (\gamma_L)_{inv} \cdot M_{LL} + \gamma_{DC} \cdot M_{DC} + \gamma_{DW} \cdot M_{DW} = 188.2\ kip \cdot ft$ *(ultimate moment at cross-section)*

NOMINAL FLEXURAL RESISTANCE

Flexural Resistance

Stress Block Factors:

$$\beta_1 := \begin{cases} \text{if } f'_c < 4\ ksi \\ \quad 0.85 \\ \text{else} \\ \quad \max\left(0.85 - 0.05 \cdot \frac{\langle f'_c - 4\ ksi \rangle}{ksi}, 0.65\right) \end{cases}$$

(stress block factor; LRFD 5.6.2.2)

$\beta_1 = 0.80$

$$\alpha_1 := \begin{cases} \text{if } f'_c < 10\ ksi \\ \quad 0.85 \\ \text{else} \\ \quad \max\left(0.85 - 0.02 \cdot \frac{\langle f'_c - 10\ ksi \rangle}{ksi}, 0.75\right) \end{cases}$$

(stress block factor; LRFD 5.6.2.2)

$\alpha_1 = 0.85$

$c := \frac{A_s \cdot f_y}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} = 4.7\ in$

(distance between the neutral axis and the compressive face for rectangular behavior with bonded strands - LRFD Eq. 5.7.3.1.1-4)

$a := \beta_1 \cdot c = 3.8\ in$

(depth of equivalent stress block for rectangular section behavior with bonded strands)

$M_n := A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) = 215.7\ kip \cdot ft$

(nominal flexural resistance; LRFD 5.7.3.2.2 & 5.7.3.2.3)

Resistance Factor

$\varepsilon_t := \left(\frac{0.003}{c}\right) \cdot \langle d_s - c \rangle = 0.0068$

(net tensile strain; LRFD 5.6.2.1)

$\varepsilon_{cl} := 0.002$

(compression-controlled strain limit; LRFD 5.6.2.1)

$\varepsilon_{tl} := 0.005$

(tension-controlled strain limit; LRFD 5.6.2.1)

$\varphi_f := \max\left(min\left((\varphi_f)_c + \frac{0.15 \cdot \langle \varepsilon_t - \varepsilon_{cl} \rangle}{\langle \varepsilon_{tl} - \varepsilon_{cl} \rangle}, (\varphi_f)_t\right), (\varphi_f)_c\right)$

(flexural resistance factor prestressed members; LRFD Eq. 5.5.4.2-1)

$\varphi_f = 0.9$

$M_r := \varphi_f \cdot M_n = 194\ kip \cdot ft$

(flexural capacity)

$IC_f := \frac{M_u}{M_r} = 0.97$

$check_f := \begin{cases} \text{if } IC_f \le 1.0 \\ \quad \text{"okay"} \\ \text{else} \\ \quad \text{"no good"} \end{cases}$

$check_f = \text{"okay"}$

Minimum Reinforcement

$M_{cr} = \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right)$ *(cracking moment; LRFD Eq. 5.6.3.3.2-1)*

$S_c = S_{nc}$ ***therefore:*** $M_{dnc}(S_c/S_{nc} - 1) := 0 \; kip \cdot ft$

Minimum Reinforcement

$M_{cr} = \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1 \right) \right)$ *(cracking moment; LRFD Eq. 5.6.3.3.2-1)*

$S_c = S_{nc}$ ***therefore:*** $M_{dnc}(S_c/S_{nc} - 1) := 0 \; kip \cdot ft$

$S_c := S_x$

$f_r := 0.24 \cdot \sqrt{f'_c \cdot ksi} = 0.5 \; ksi$ *(modulus of rupture; LRFD 5.4.2.6)*

$f_{cpe} := 0 \; ksi$ *(compressive stress in concrete due to effective prestress force)*

$\gamma_1 := 1.6$ *(flexural cracking variability factor)*

$\gamma_2 := 1.0$ *(prestress variability factor)*

$\gamma_3 := \frac{f_y}{f_u}$ *(ratio of specified minimum yield strength to ultimate strength)*

$M_{cr} := \overrightarrow{\gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) \cdot S_c - M_{dnc}(S_c/S_{nc} - 1) \right)}$ *(cracking moment; LRFD Eq. 5.6.3.3.2-1)*

$check := \begin{cases} \text{if } M_r \ge min\left(1.33 \cdot M_u, M_{cr}\right) \\ \quad \text{"okay"} \\ \text{else} \\ \quad \text{"reduce Mn"} \end{cases}$

$check = \text{"okay"}$

$$K := \left\| \begin{array}{l} \text{if } M_r \geq min\left(1.33 \cdot M_u, M_{cr}\right) \\ \quad \| \ 1.0 \\ \text{else} \\ \quad \| \ min\left(1.0, \frac{M_r}{min\left(1.33 \cdot M_u, M_{cr}\right)}\right) \end{array} \right\|$$

(minimum reinforcement reduction factor - MBE 6A.5.6)

$K = 1.00$

RATING FACTORS: FLEXURE

$\gamma_{DC} = 1.25$ *(dead load factor for components and attachments)*

$\gamma_{DW} = 1.50$ *(dead load factor for wearing surface)*

$R_n := M_n$ *(nominal member resistance)*

$C := \varphi_c \cdot \varphi_s \cdot \varphi_f \cdot R_n = 194 \ kip \cdot ft$ *(capacity, MBE Eqs. 6A.4.2.1-2 & 6A.4.2.1-3)*

$DC := M_{DC} = 53 \ kip \cdot ft$ *(dead load effect due to structural components and attachments)*

$DW := M_{DW} = 0 \ kip \cdot ft$ *(dead load effect due to wearing surface and utilities)*

$LL := M_{LL} = 69.7 \ kip \cdot ft$ *(live load effect)*

$$RF_{inv} := \frac{C - \gamma_{DC} \cdot DC - \gamma_{DW} \cdot DW}{(\gamma_L)_{inv} \cdot LL}$$

(inventory design rating factor for positive flexure relative to the compression flange; MBE Eq. 6A.4.2.1-1)

$RF_{inv} = 1.05$

$$RF_{op} := \frac{(\gamma_L)_{inv}}{(\gamma_L)_{op}} \cdot RF_{inv}$$

(operating design rating factor for positive flexure)

$RF_{op} = 1.36$

Job Information

	Engineer	Checked	Approved
Name:			
Date:			

Project ID	
Project Name	

Structure Type	PLANE FRAME

Number of Nodes	4	Highest Node	103
Number of Elements	3	Highest Beam	103

Number of Basic Load Cases	1056
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Generation	1	LOAD GENERATION, LOAD #1, (1 of 208
Primary	10001	LANE LOAD - ALL SPANS
Primary	10002	LANE LOAD - SPAN 1 ONLY
Primary	10003	LANE LOAD - SPAN 2 ONLY
Primary	10004	LANE LOAD - SPAN 3 ONLY
Primary	10005	LANE LOAD - SPAN 1 & 2
Primary	10006	LANE LOAD - SPAN 2 & 3
Primary	10007	LANE LOAD - SPAN 1 & 3
Primary	20001	DEAD LOAD DC

Nodes

Node	X (ft)	Y (ft)	Z (ft)
100	0	0	0
101	54.000	0	0
102	122.500	0	0
103	176.500	0	0

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
101	100	101	54.000	1	0

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
100	Fixed	Fixed	Fixed	-	-	-
101	Fixed	Fixed	Fixed	-	-	-
102	Fixed	Fixed	Fixed	-	-	-
103	Fixed	Fixed	Fixed	-	-	-

Primary Load Cases

Number	Name	Type
10001	LANE LOAD - ALL SPANS	None
10002	LANE LOAD - SPAN 1 ONLY	None
10003	LANE LOAD - SPAN 2 ONLY	None
10004	LANE LOAD - SPAN 3 ONLY	None
10005	LANE LOAD - SPAN 1 & 2	None
10006	LANE LOAD - SPAN 2 & 3	None
10007	LANE LOAD - SPAN 1 & 3	None
20001	DEAD LOAD DC	None

Moving Load Definition : Type 1

Width (ft)
-

Force (kip)	Distance (ft)
8.000	-
32.000	14.000
32.000	14.000

Moving Load Definition : Type 2

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	14.000
8.000	14.000

Moving Load Definition : Type 3

Width (ft)
-

Force (kip)	Distance (ft)
8.000	-
32.000	14.000
32.000	30.000

Moving Load Definition : Type 4

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	30.000
8.000	14.000

Moving Load Definition : Type 5

Width (ft)
-

Force (kip)	Distance (ft)
25.000	-
25.000	4.000

Moving Loads: Loads 1 to 208

	Initial Position			Increment			
Type	X (ft)	Y (ft)	Z (ft)	X (ft)	Y (ft)	Z (ft)	Range (ft)
1	-28.000	0	0	1.000	-	-	-

10001 LANE LOAD - ALL SPANS : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-
102	UNI kips/ft	GY	-0.640	-	-	-	-
103	UNI kips/ft	GY	-0.640	-	-	-	-

10002 LANE LOAD - SPAN 1 ONLY : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-

10003 LANE LOAD - SPAN 2 ONLY : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
102	UNI kips/ft	GY	-0.640	-	-	-	-

10004 LANE LOAD - SPAN 3 ONLY : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
103	UNI kips/ft	GY	-0.640	-	-	-	-

10005 LANE LOAD - SPAN 1 & 2 : Beam Loads

Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-
102	UNI kips/ft	GY	-0.640	-	-	-	-

10006 LANE LOAD - SPAN 2 & 3 : Beam Loads

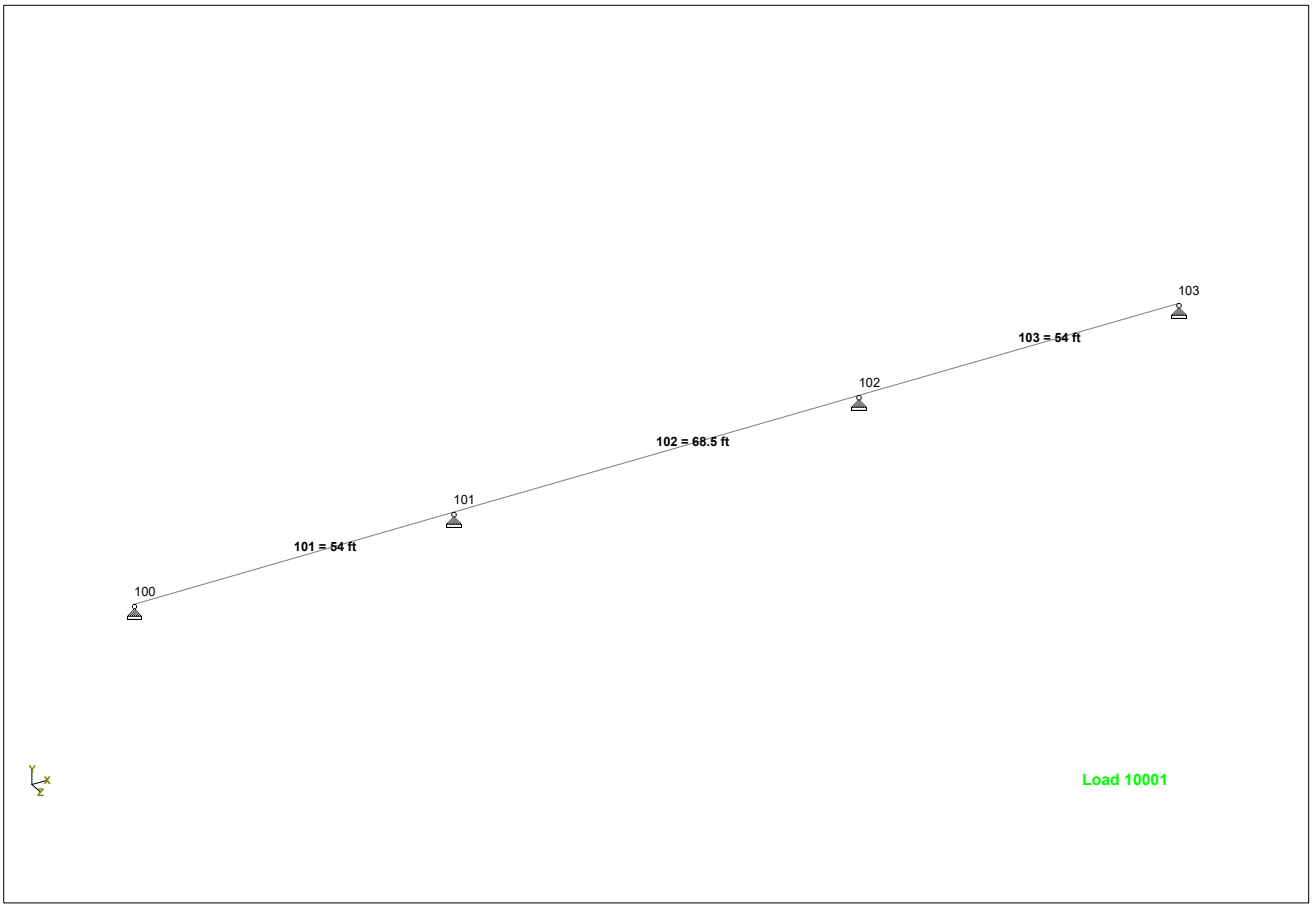
Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
102	UNI kips/ft	GY	-0.640	-	-	-	-
103	UNI kips/ft	GY	-0.640	-	-	-	-

10007 LANE LOAD - SPAN 1 & 3 : Beam Loads

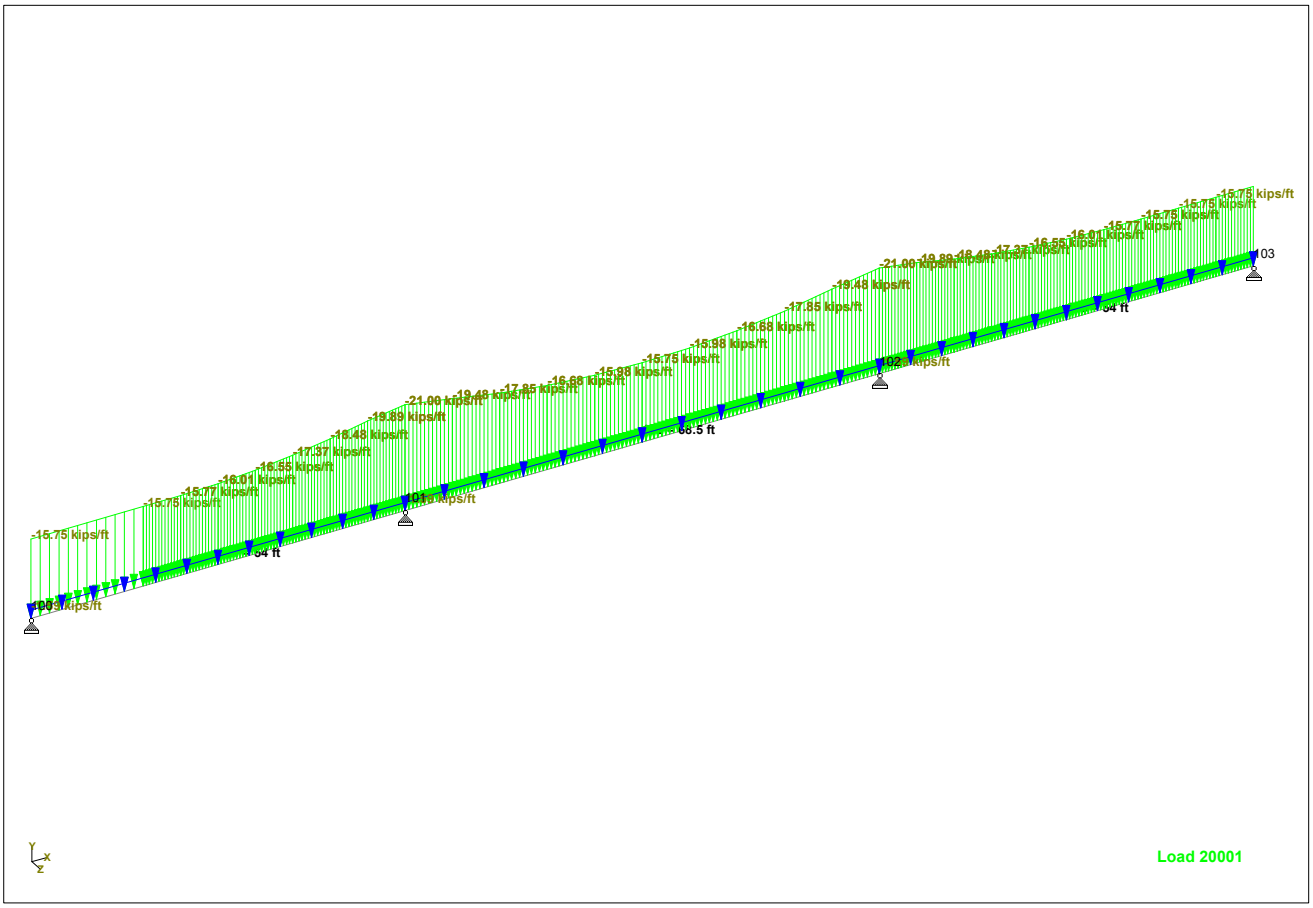
Beam	Type	Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI kips/ft	GY	-0.640	-	-	-	-
103	UNI kips/ft	GY	-0.640	-	-	-	-

20001 DEAD LOAD DC : Beam Loads

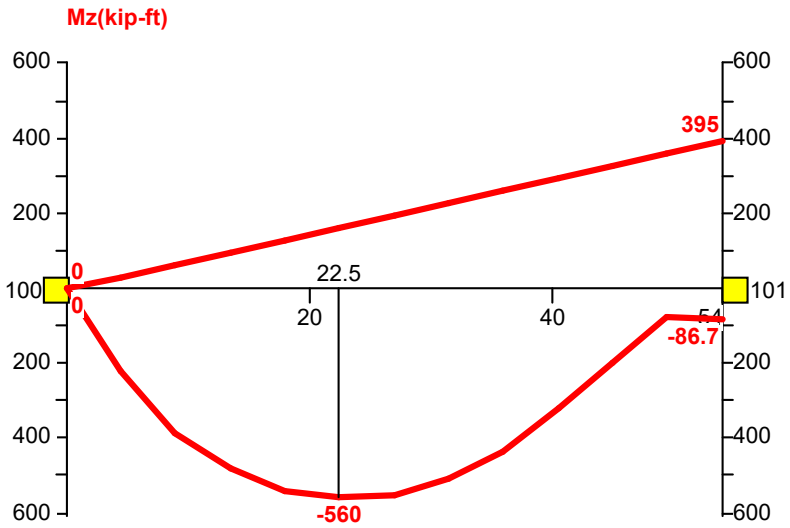
Beam	Type		Direction	Fa	Da (ft)	Fb	Db	Ecc. (ft)
101	UNI	kips/ft	GY	-1.594	-	-	-	-
	UNI	kips/ft	GY	-15.750	0	-	16.203	-
	TRAP	kips/ft	GY	-15.750	16.203	-15.767	21.604	-
	TRAP	kips/ft	GY	-15.767	21.604	-16.010	27.005	-
	TRAP	kips/ft	GY	-16.010	27.005	-16.546	32.406	-
	TRAP	kips/ft	GY	-16.546	32.406	-17.370	37.807	-
	TRAP	kips/ft	GY	-17.370	37.807	-18.484	43.208	-
	TRAP	kips/ft	GY	-18.484	43.208	-19.886	48.609	-
	TRAP	kips/ft	GY	-19.886	48.609	-21.000	54.000	-
102	UNI	kips/ft	GY	-1.594	-	-	-	-
	TRAP	kips/ft	GY	-21.000	0	-19.482	6.849	-
	TRAP	kips/ft	GY	-19.482	6.849	-17.849	13.698	-
	TRAP	kips/ft	GY	-17.849	13.698	-16.683	20.547	-
	TRAP	kips/ft	GY	-16.683	20.547	-15.982	27.396	-
	TRAP	kips/ft	GY	-15.982	27.396	-15.750	34.245	-
	TRAP	kips/ft	GY	-15.750	34.245	-15.982	41.094	-
	TRAP	kips/ft	GY	-15.982	41.094	-16.683	47.943	-
	TRAP	kips/ft	GY	-16.683	47.943	-17.849	54.792	-
	TRAP	kips/ft	GY	-17.849	54.792	-19.482	61.641	-
	TRAP	kips/ft	GY	-19.482	61.641	-21.000	68.490	-
103	UNI	kips/ft	GY	-1.594	-	-	-	-
	TRAP	kips/ft	GY	-21.000	0	-19.886	5.401	-
	TRAP	kips/ft	GY	-19.886	5.401	-18.484	10.802	-
	TRAP	kips/ft	GY	-18.484	10.802	-17.370	16.203	-
	TRAP	kips/ft	GY	-17.370	16.203	-16.546	21.604	-
	TRAP	kips/ft	GY	-16.546	21.604	-16.010	27.005	-
	TRAP	kips/ft	GY	-16.010	27.005	-15.767	32.406	-
	TRAP	kips/ft	GY	-15.767	32.406	-15.750	37.807	-
	TRAP	kips/ft	GY	-15.750	37.807	-15.750	43.208	-
	TRAP	kips/ft	GY	-15.750	43.208	-15.750	48.609	-
	TRAP	kips/ft	GY	-15.750	48.609	-15.750	54.010	-



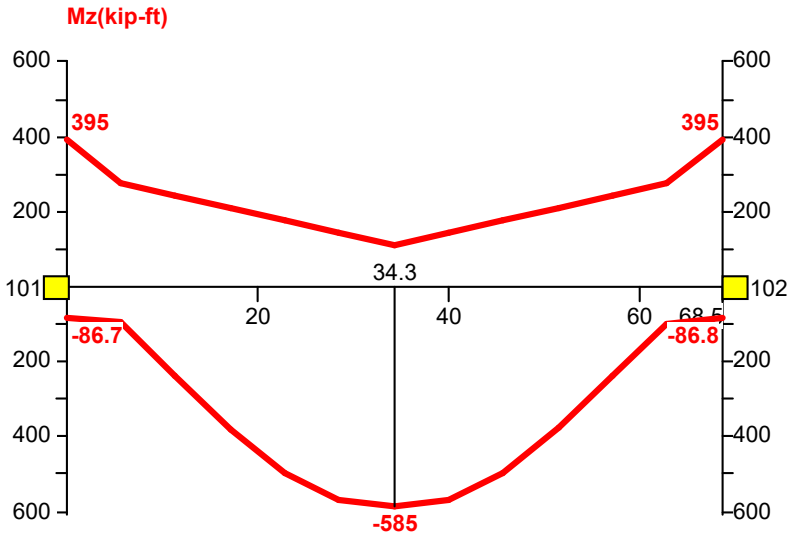
Dimensions



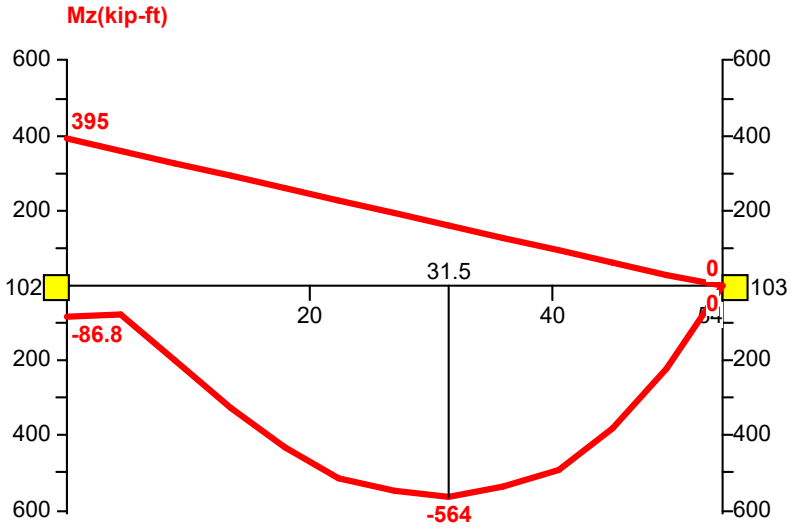
Dead Load



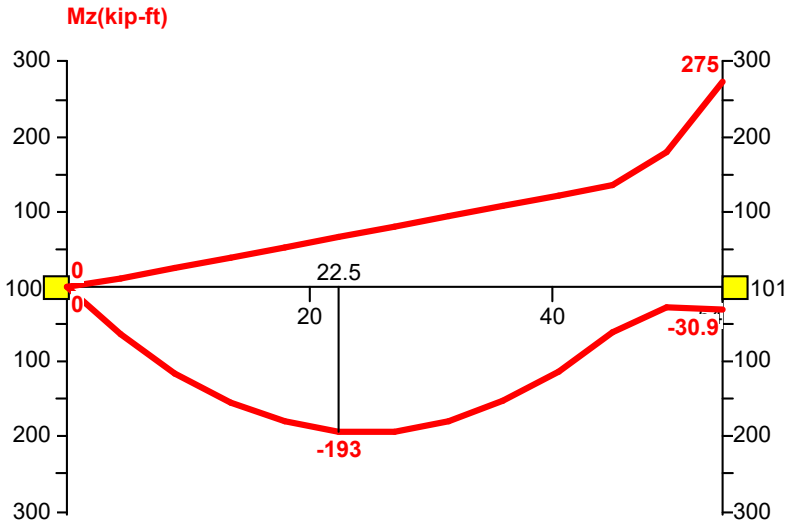
Span 1 Moment HL93 Truck/Tandem



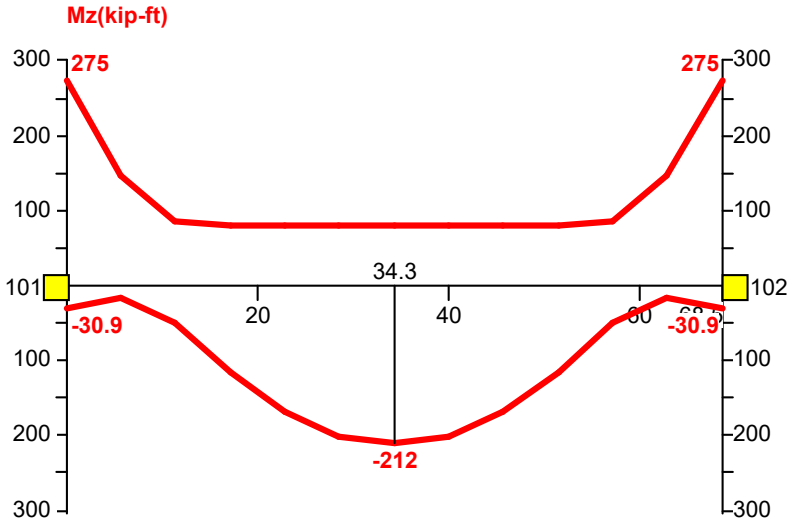
Span 2 Moment HL93 Truck/Tandem



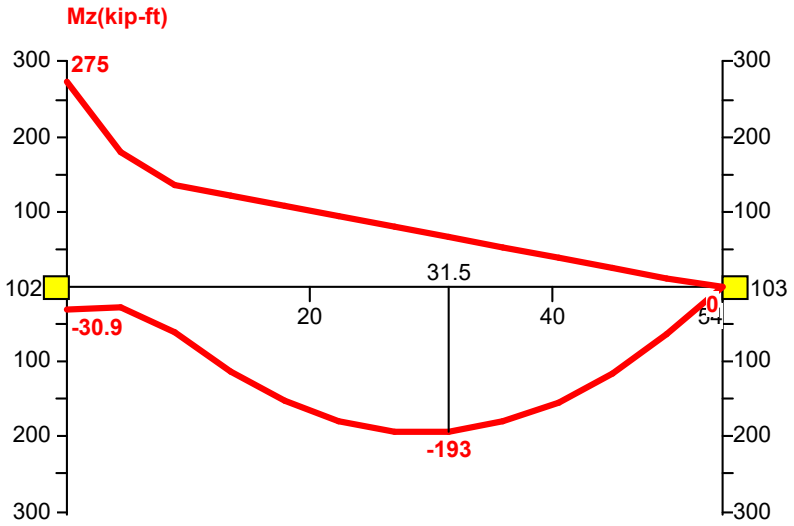
Span 3 Moment HL93 Truck/Tandem



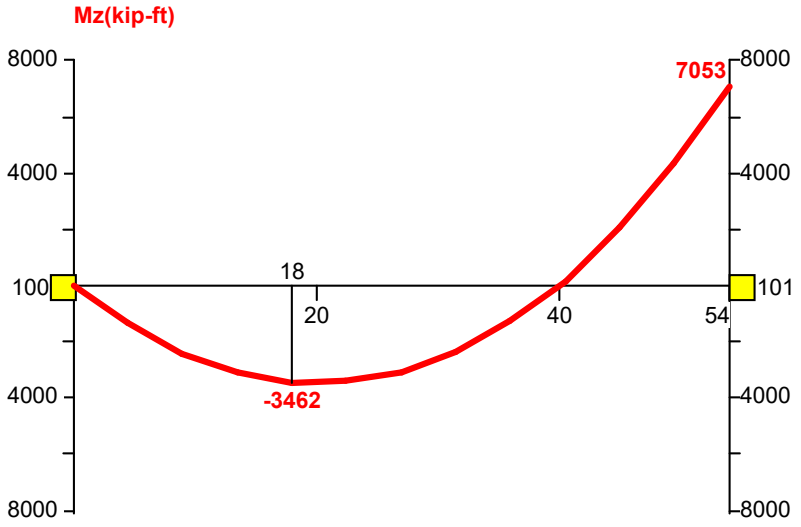
Span 1 Moment HL93 Lane Load



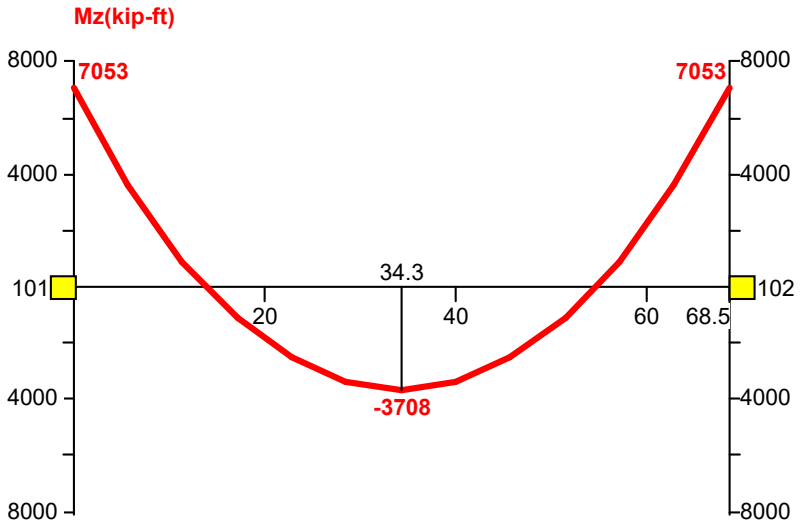
Span 2 Moment HL93 Lane Load



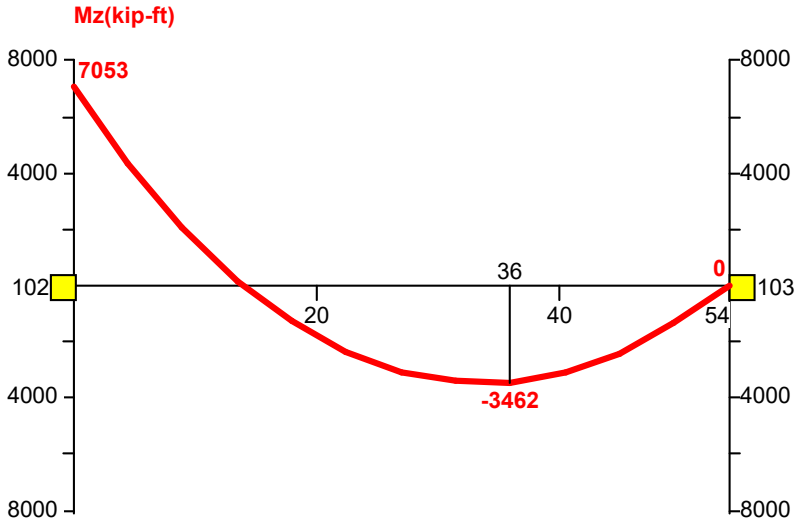
Span 3 Moment HL93 Lane Load



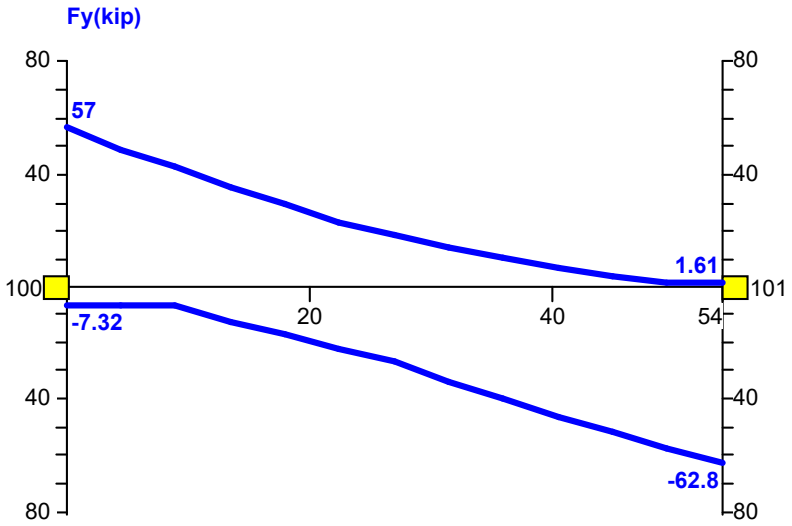
Span 1 Moment Dead Load DC



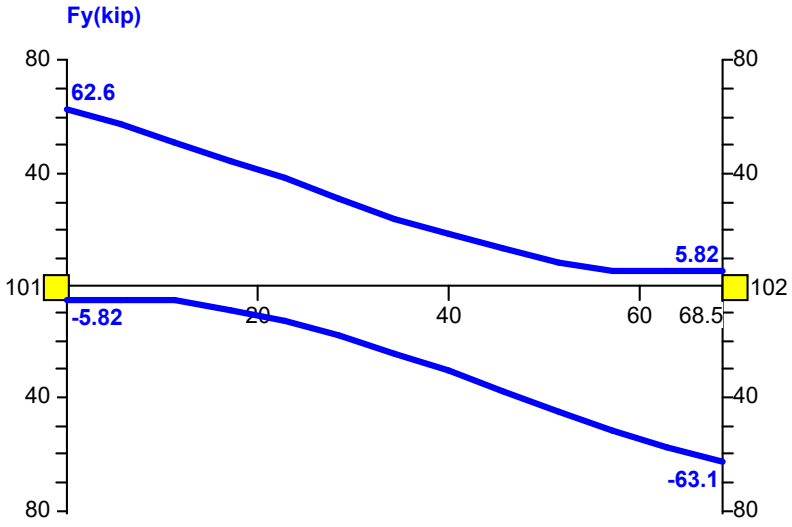
Span 2 Moment Dead Load DC



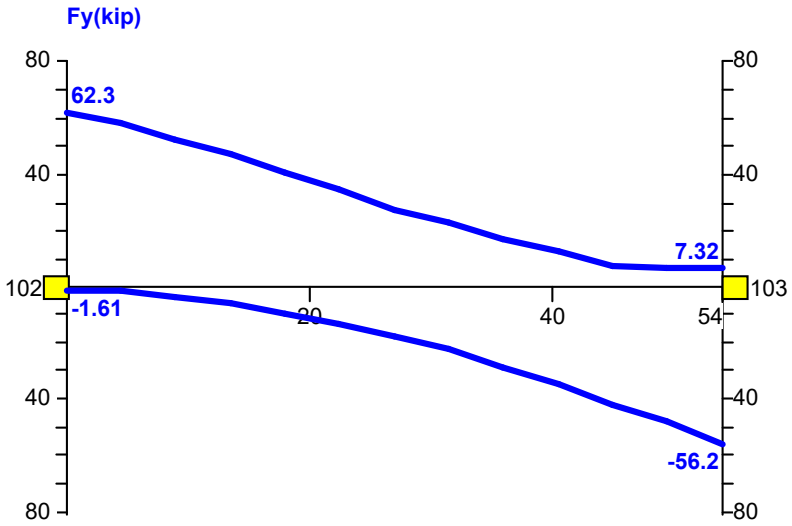
Span 3 Moment Dead Load DC



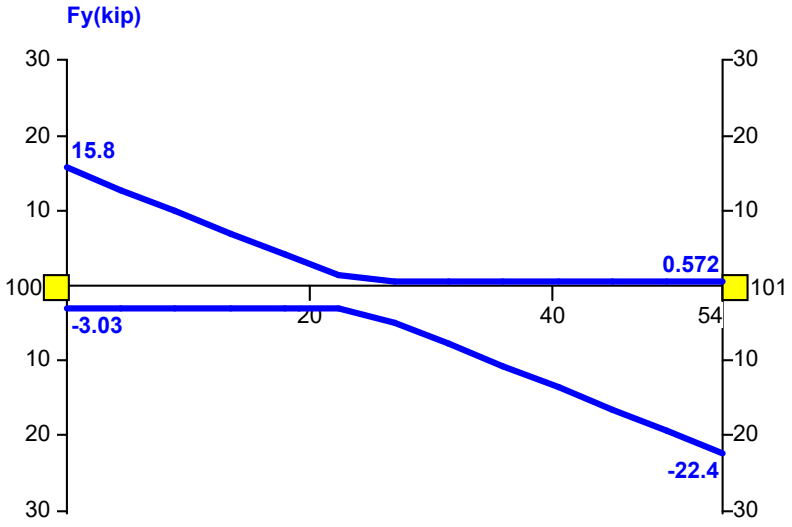
Span 1 Shear Truck/Tandem



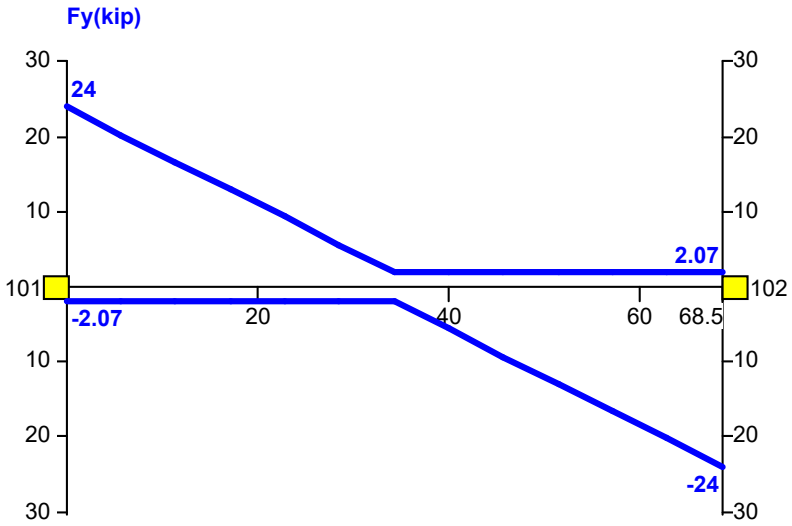
Span 2 Shear Truck/Tandem



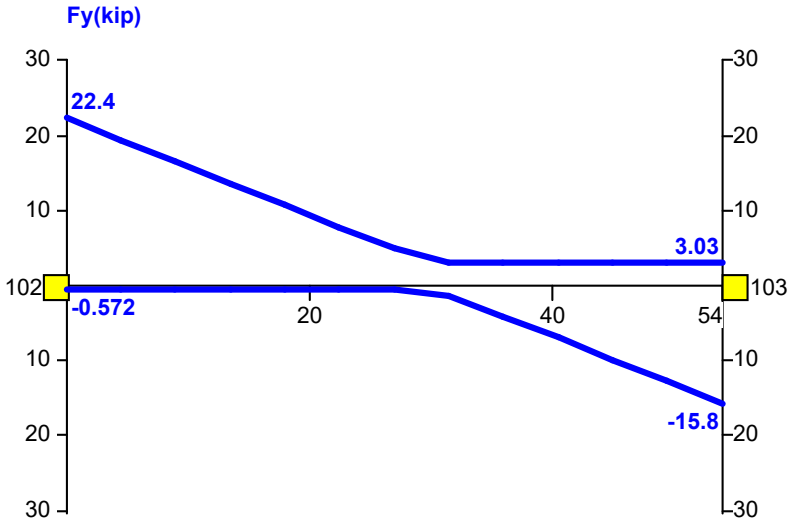
Span 3 Shear Truck/Tandem



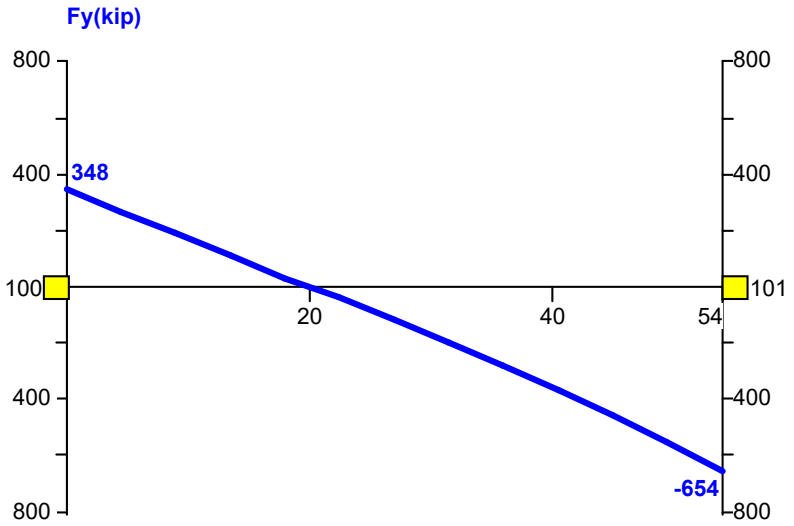
Span 1 Shear HL93 Lane Load



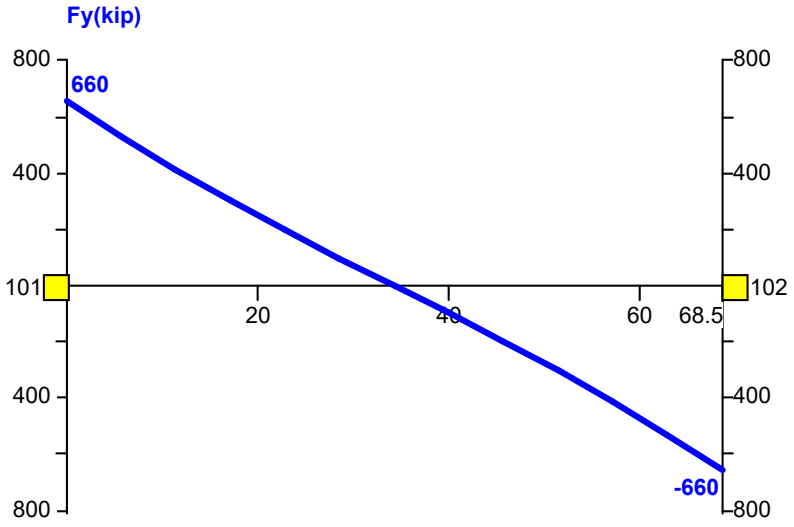
Span 2 Shear HL93 Lane Load



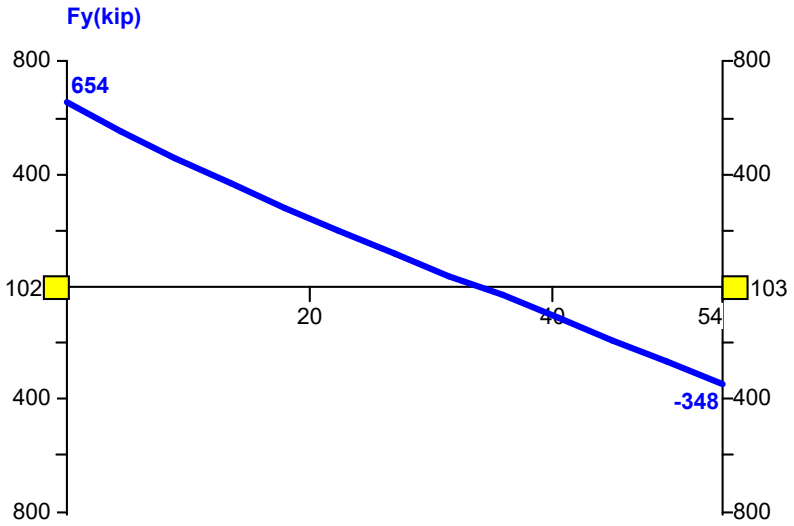
Span 3 Shear HL93 Lane Load



Span 1 Shear Dead Load DC



Span 2 Shear Dead Load DC



Span 3 Shear Dead Load DC

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
100	+ve	0	56.989	0	0	0	0
100	+ve	-	Load: 237	-	-	-	-
100	-ve	0	-7.321	0	0	0	0
100	-ve	-	Load: 309	-	-	-	-
101	+ve	0	69.234	0	0	0	0
101	+ve	-	Load: 283	-	-	-	-
101	-ve	0	-7.429	0	0	0	0
101	-ve	-	Load: 374	-	-	-	-
102	+ve	0	69.231	0	0	0	0
102	+ve	-	Load: 132	-	-	-	-
102	-ve	0	-7.431	0	0	0	0
102	-ve	-	Load: 40	-	-	-	-
103	+ve	0	56.197	0	0	0	0
103	+ve	-	Load: 177	-	-	-	-
103	-ve	0	-7.321	0	0	0	0
103	-ve	-	Load: 105	-	-	-	-

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
100	+ve	0	15.793	0	0	0	0
100	+ve	-	Load: 10007	-	-	-	-
100	-ve	0	-3.035	0	0	0	0
100	-ve	-	Load: 10003	-	-	-	-
101	+ve	0	46.367	0	0	0	0
101	+ve	-	Load: 10005	-	-	-	-
101	-ve	0	-2.646	0	0	0	0
101	-ve	-	Load: 10004	-	-	-	-
102	+ve	0	46.367	0	0	0	0
102	+ve	-	Load: 10006	-	-	-	-
102	-ve	0	-2.646	0	0	0	0
102	-ve	-	Load: 10002	-	-	-	-
103	+ve	0	15.793	0	0	0	0
103	+ve	-	Load: 10007	-	-	-	-
103	-ve	0	-3.035	0	0	0	0
103	-ve	-	Load: 10003	-	-	-	-

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
100	+ve	0	348.430	0	0	0	0
100	+ve	-	Load: 20001	-	-	-	-
100	-ve	0	0	0	0	0	0
100	-ve	-	-	-	-	-	-
101	+ve	0	1.31E+3	0	0	0	0
101	+ve	-	Load: 20001	-	-	-	-
101	-ve	0	0	0	0	0	0
101	-ve	-	-	-	-	-	-
102	+ve	0	1.31E+3	0	0	0	0
102	+ve	-	Load: 20001	-	-	-	-
102	-ve	0	0	0	0	0	0
102	-ve	-	-	-	-	-	-
103	+ve	0	348.437	0	0	0	0
103	+ve	-	Load: 20001	-	-	-	-
103	-ve	0	0	0	0	0	0
103	-ve	-	-	-	-	-	-

APPENDIX E – H&H

E.1 Supplemental H&H Report
E.2 Flow Input Data
E.3 HEC-FDA H&H Input Data
E.4 Riprap Sizing H&H Input Data
E.5 Bridge Scour H&H Input Data
E.6 Hydraulic Model Outputs
E.7 Stantec Hydraulic Model Report
E.8 Bridge Drawings

This page was intentionally left blank for duplex printing.

APPENDIX E.1 – SUPPLEMENTAL H&H REPORT

Introduction

Flood History

Hydrological Development from Feasibility Study

Hydraulic Model Development

Levee Profile Development

Sensitivity Analysis

E.1 SUPPLEMENTAL H&H REPORT

E.1.1. Introduction

The purpose of this supplemental report is to provide detailed methodology behind the hydrologic and hydraulic model development and analyses, along with background information of the project area.

E.1.2. Flood History

Major floods on Turkey Creek are usually caused by short-duration, high-intensity storms following an extended period of general rainfall over the basin. The rainfall preceding the heavy rains saturates the soil, preventing further infiltration and producing runoff into the Turkey Creek channel, creating higher than average flow. Since the Turkey Creek channel is so narrow and urbanized, it is usually subject to flash flood events, which are difficult to forecast and prepare for. Earlier flood events on Turkey Creek are known to have happened in 1844, 1951, 1958, 1961, and 1968. Very little information was documented for these specific events. Recent flood events that have available records and documentation include 1977, 1983, 1986, 1993, 1995, 1996, 1998, and 2017. However, the focus will be on the four largest flood events, which were 1977, 1993, 1998, and 2017. The first three floods' rainfall data were recorded by the same rain gage at Shawnee, Kansas. Precipitation for the 2017 events were determined using both Johnson County Stormwatch rain gage data and National Weather Service (NWS) radar observations. A basin average is provided for 21–22 August 2017. Table E.6 summarizes the observed precipitation events. Subsections E.1.2.1 through E.1.2.9 discuss details pertaining to historic flood events.

Table E.1. Comparison of Rainfall Totals for Major Turkey Creek Floods

Flood Event Date	Peak Hour Rainfall (inches)	Total Rainfall (inches)	Recorded Dates
1977	1.60	10.90	12–13 September 1977
1993	1.50	6.00	10 July 1993
1998	3.81	4.44	04 October 1998
2017	1.05	5.71	21–22 August 2017
2025	3.83	5.5	16-17 July 2025

E.1.2.1. Flood of 1935

The greatest known flood at Merriam, KS at the time occurred in June 1935 with an estimated Turkey Creek discharge of 13,650 cfs. It should be noted that a specific location of estimated flow was not given in the historic documentation, but it is assumed that flow was estimated at Johnson Dr. in Merriam, KS. This flood inundated the city area to depths of 7 feet above what was considered flood stage at the time. Development in the area was not extensive in 1935 and no estimated damage caused by this flood is available.

E.1.2.2. Flood of 1944

The April 1944 flood at Merriam, KS had an estimated Turkey Creek discharge of 11,300 cfs. It inundated the city to depths of about 6 feet above what was considered flood stage at the time. 58 residences and businesses were affected, and the damage was estimated at $118,700 in 1944 price levels.

E.1.2.3. Flood of 1958

In July 1958, a flood with an estimated discharge of 4,400 cfs inundated a total of 69 residences and business establishments and resulted in damages estimated at $155,000 in 1958 price levels.

E.1.2.4. Flood of 1961

The September 1961 flood at Merriam had an estimated discharge of 4,700 cfs and inundated a total of 82 residences and business establishments. An exhaustive damage survey, conducted shortly after the flood, established a value of $257,000 (1961 price levels) in damages in the city, of which $240,000 occurred along the main stem of Turkey Creek. This survey included all flood damages within the corporate city limits. The previous flood damage estimates may have included a smaller area, but the primary reason for the increasing damage values is the rapid development of the flood plain at the time.

E.1.2.5. Flood of 1977

Locally known as the Plaza Flood, this was a flash flood event that occurred 12–13 September 1977, throughout the Kansas City metropolitan area. It was caused by a high intensity storm following a long period of light rain. Approximately 16 inches of rain fell over the Kansas City, Missouri, area over the course of the event.

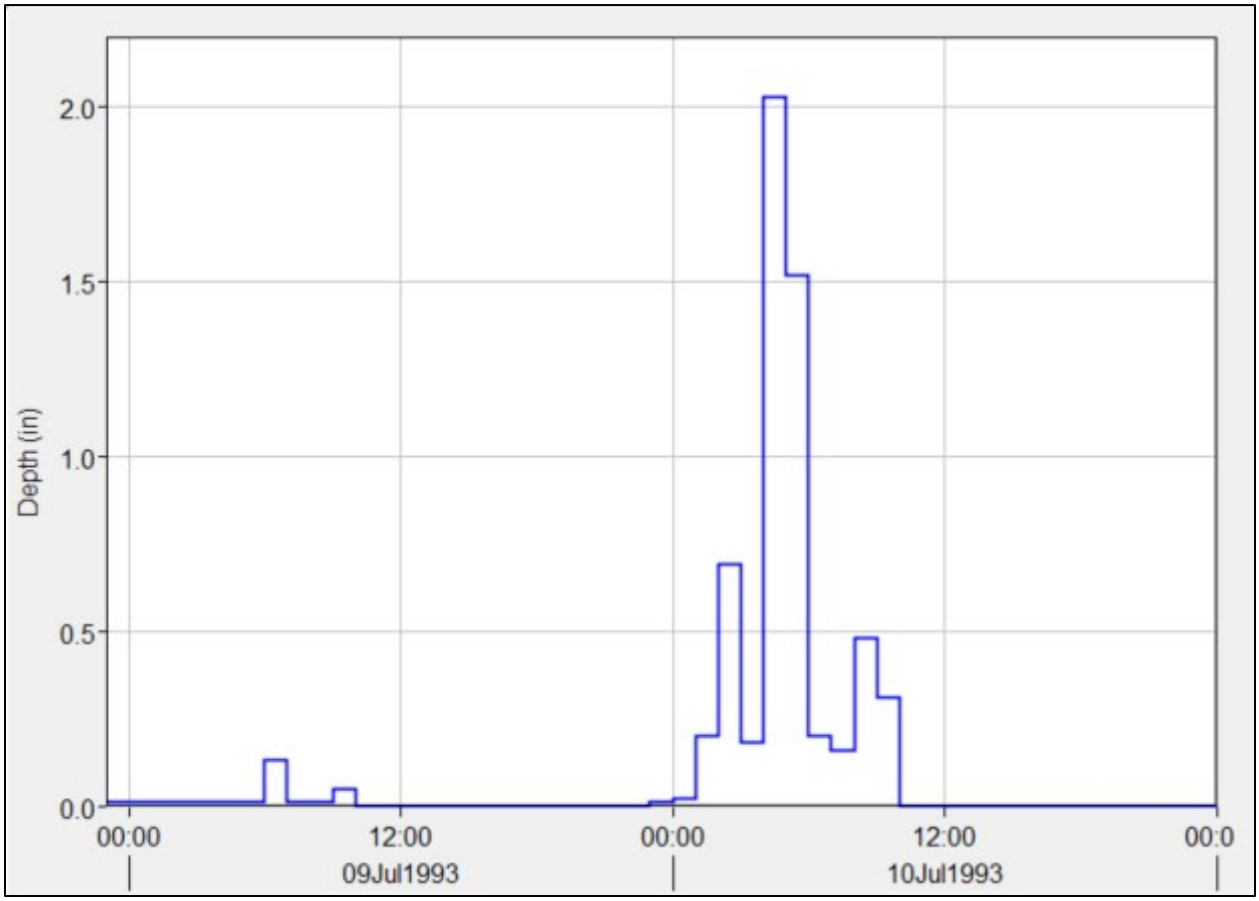
Most of the focus was on Brush Creek, which travels through the Plaza and discharges into the Blue River. During this event, there were 25 fatalities and 1,000 automobiles totaled, along with millions of dollars in damages to the commercial and residential areas surrounding Brush Creek.

In the Turkey Creek area, $8.1 million (1977 price levels) in damages occurred from flooding. Although damages were extensive, the urban study conducted after the flood determined implementing a program for managing flood events along Turkey Creek was not feasible.

E.1.2.6. Flood of 1993

Extensive flooding occurred on 10 July 1993. Although this year was also a major flood year for the Missouri and Kansas Rivers in Kansas City, this was an unrelated flash flood event caused by high intensity storms on that date. When Turkey Creek flooded in 1993, it overtopped I-35 at five different locations. The flooding occurring during this event caused one fatality. Damages from the 1993 event were estimate at $3.4 million in Merriam, Kansas, and $20 million in the lower basin in 1993 price levels. A rainfall gage located at the Overland Park City Hall building (approximately 3.3 miles south of Merriam, Kansas) recorded the July 1993 storm event. Figure E.1 provides a rainfall hyetograph of the event. According to this observed data, approximately 5.8 inches of rain fell over the upper portion of the Turkey Creek watershed within a 12-hour period.

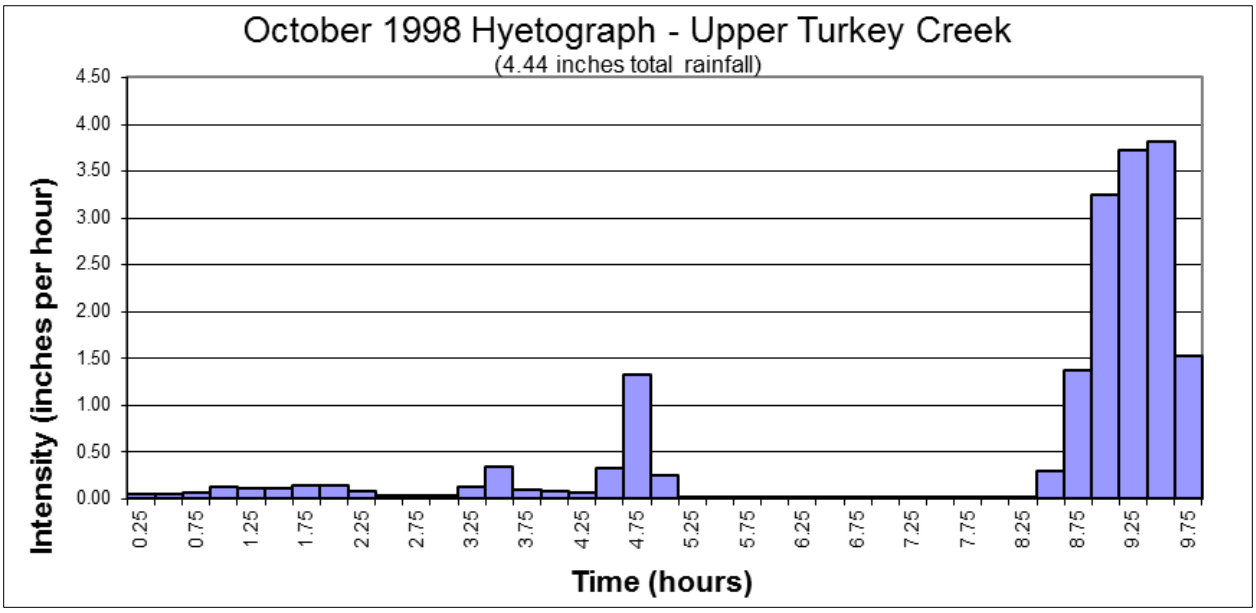
Figure E.1. 10 July 1993 Rainfall Hyetograph for Overland Park, Kansas City Hall Rain Gage.



E.1.2.7. Flood of 1998

The maximum flood of record for Turkey Creek occurred in October 1998 and resulted from the combined effects of saturated soil due to rain on 6 out of 7 days in one week and a high intensity storm that started 4 October 1998. The storm covered the Kansas City metropolitan area with rain for the entire day with afternoon rainfall totals of 0.25 to 1.00 inches. Around 7:00 p.m., another round of storms produced an additional 3 to 5 inches of rain in a 2-hour period. The rainfall produced by this event exceeded the rainfall amount defined as the 1% AEP (1/100-year event) at the time. Although this flood had the least amount of total rainfall when compared to the 1977 and 1993 floods, it produced the highest amount of rainfall in the peak hour. Figure E.2 illustrates the 10 hours of highest rainfall intensity of the 1998 rainfall event hyetograph emphasizing the rapidness of this event. The worst damages caused by this flood occurred at Merriam, Kansas, near the midpoint of the Upper Turkey Creek reach. Multiple bridges and roadways were flooded during this event including I-35 and the BNSF railroad tracks south of 75th Street. The total cost of damages was estimated at $12 million in Merriam, Kansas, and $20 million in the lower basin area, in 1998 price levels.

Figure E.2. Hyetograph of 4 October 1998 Storm Event

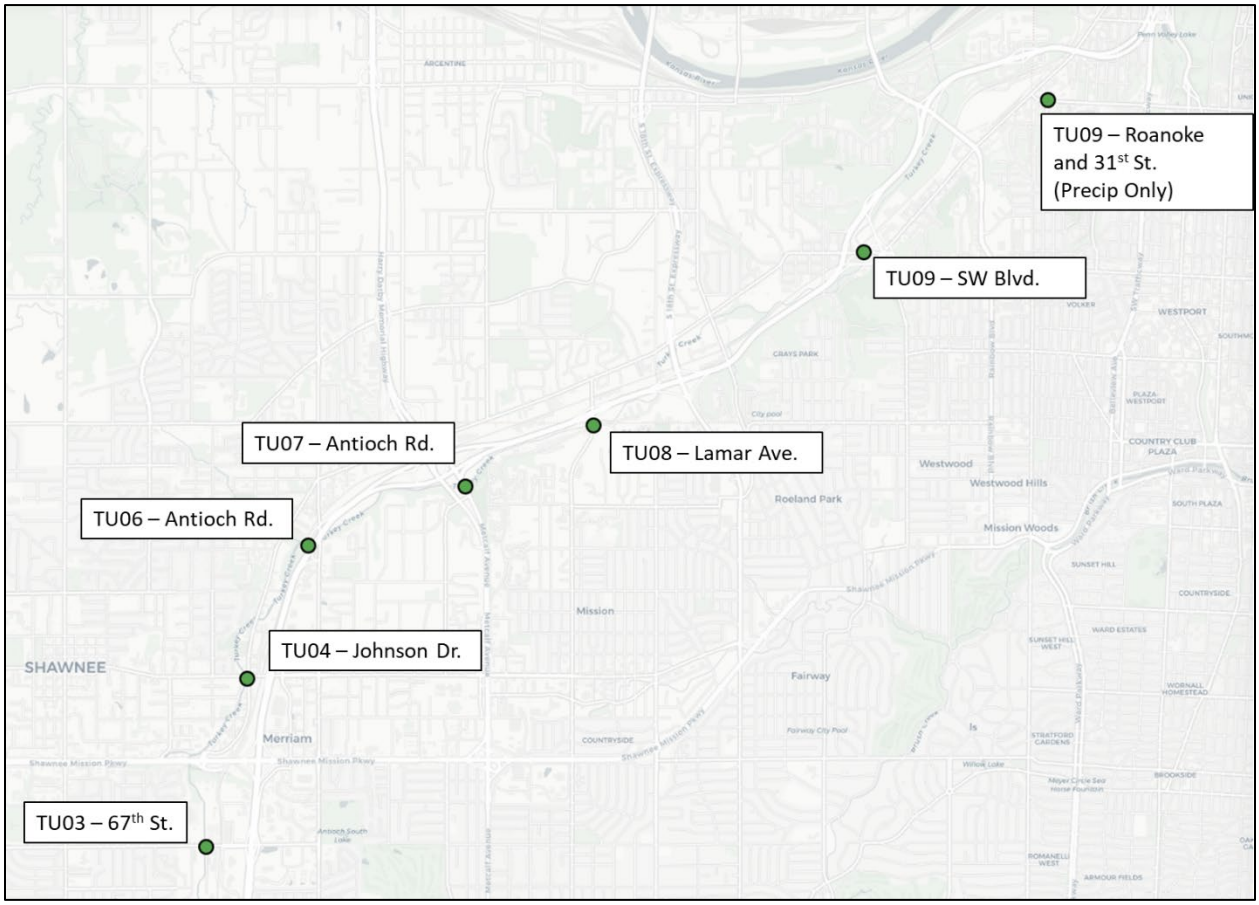


E.1.2.8. Floods of 2017

Three significant rainfall events occurred in the Turkey Creek watershed during the summer of 2017 on 26–27 July, 5–6 August, and 21–22 August. Storm total precipitation was highest on 21–22 August, but the intensity was greater for 26–27 July and 5–6 August. Rainfall from the events produced some of the highest stages along Turkey Creek since the flood of record on 4 October 1998. Of the 2017 events, 26–27 July resulted in the record stage of 16.45 feet at the Johnson Drive gage.

Precipitation, flow rate, and stream stage from the 2017 events were compared to historical records and to published data. Precipitation totals for the 2017 events were determined using both Johnson County Stormwatch rain gage data and NWS radar observations. The rain gages were location along Turkey Creek at Johnson Drive, Southwest Boulevard, and Lamar Avenue. Average basin precipitation totals were estimated from the NWS radar data using the Meteorological Visualization Utility Engine (MetVUE) software. Figure E.3 provides locations of the Stormwatch gages along Turkey Creek. All gages record both river stage and rainfall, with the exception of TU09, which records rainfall only.

Figure E.3. Turkey Creek Stormwatch Gages



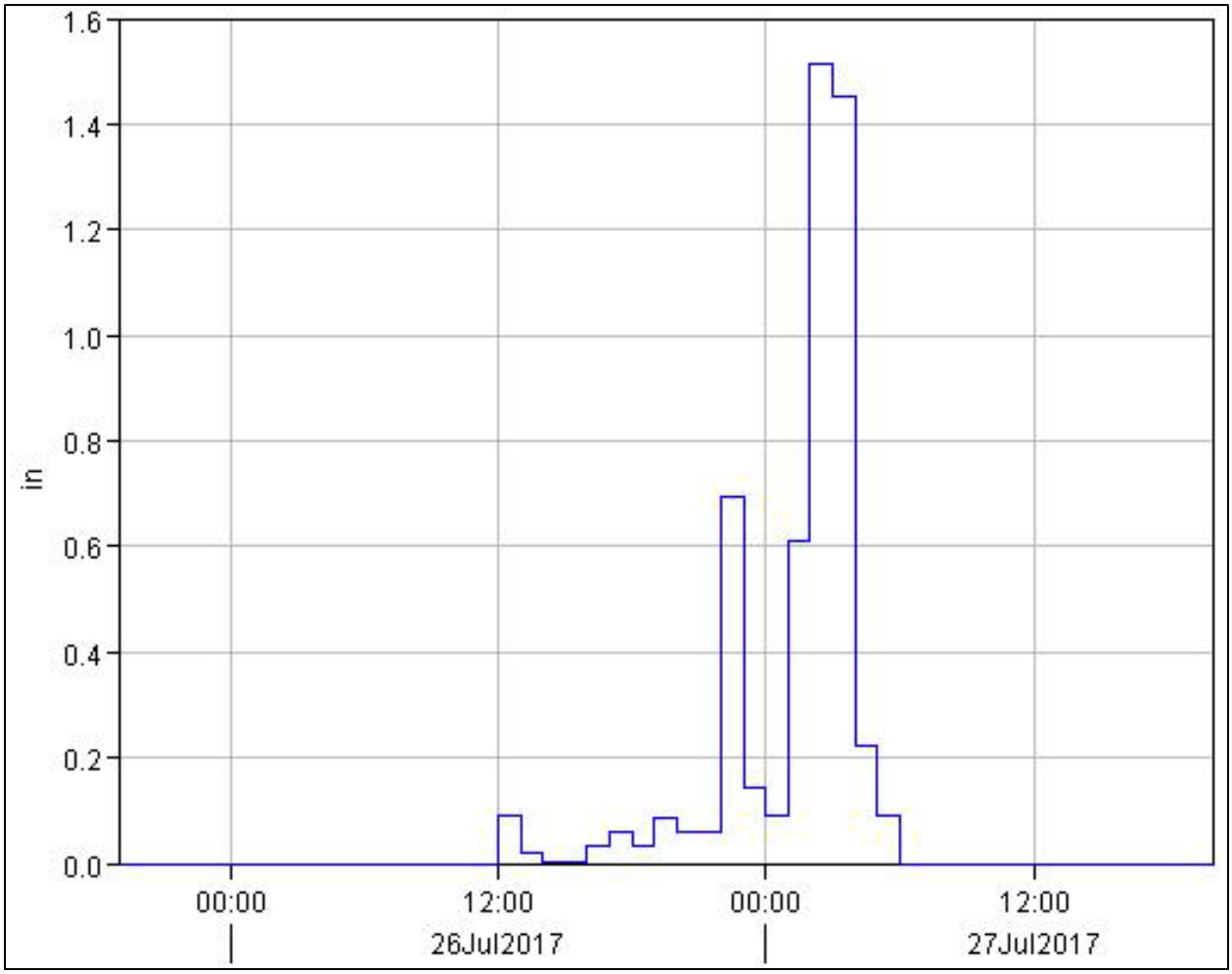
E.1.2.1.1 26–27 July 2017

An average of 5.27 inches of rain fell over the Turkey Creek Basin between 12:00 p.m. on 26 July 2017, and 6:00 a.m. on 27 July 2017. Approximately 68 percent of this total fell in a three-hour period between 1:00 and 4:00 a.m. on the 27th; while the maximum hourly total was 1.52 inches, occurring between 2:00 and 3:00 p.m. on the 27th. Table E.2 shows precipitation totals for the gages and the basin. All the rain gages recorded higher totals and more intense rainfall than the basin average. Figure E.4 shows the basin average hyetograph.

Table E.2. Precipitation Totals for 26–27 July 2017

Location	Total (inches)	Maximum 1-hour (inches)	Maximum 3-hour (inches)	Maximum 6-hour (inches)
Southwest Boulevard	5.92	1.80	4.12	5.32
Lamar Avenue	5.24	1.76	3.96	4.80
Johnson Drive	5.56	2.16	4.28	5.20
Basin Average	5.27	1.52	3.58	4.51

Figure E.4. Turkey Creek Basin Average Hyetograph for 26–27 July 2017



E.1.2.1.1 5–6 August 2017

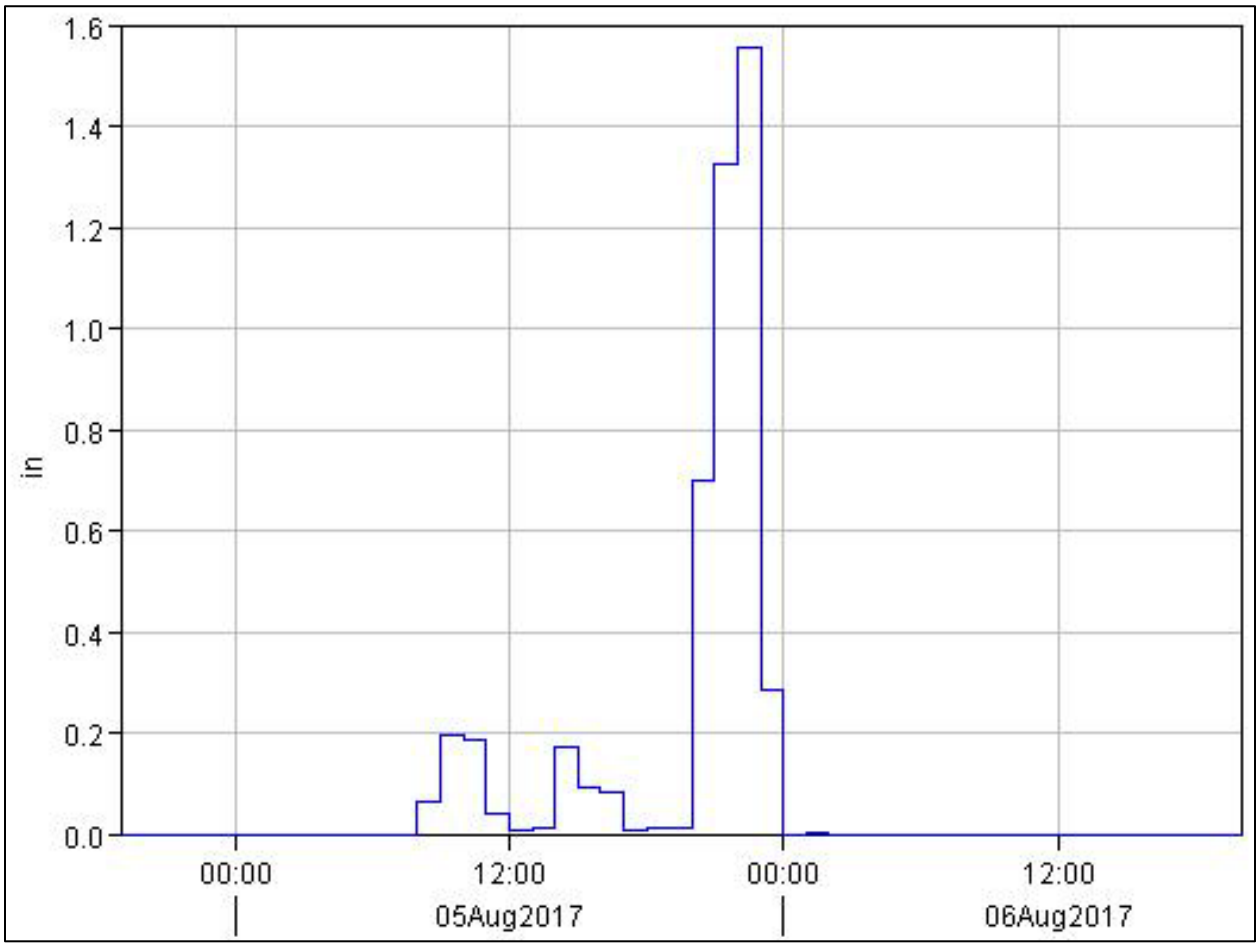
A total of 4.75 inches of rain fell over the Turkey Creek Basin between 8:00 a.m. 5 August 2017, and 2:00 a.m. 6 August 2017. Approximately 75 percent of the total precipitation fell in a three-hour period between 8:00 p.m. and 11:00 p.m. on the 5th; while the maximum 1-hour intensity was 1.55 inches between 10:00 p.m. and 11:00 p.m. Table E.3 shows storm precipitation totals for the gages and the entire basin. The rainfall totals were significantly higher at the Lamar Avenue and Johnson Drive gages than the basin average and the Southwest Boulevard gage. Peak discharge was estimated using the USACE lower Turkey Creek design model at Johnson Drive and Southwest Boulevard in the Turkey Creek 2017 flood report. Figure E.5 shows the basin average hyetograph.

Table E.3. Precipitation Totals for 5–6 August 2017

Location	Total (inches)	Maximum 1-hour (inches)	Maximum 3-hour (inches)	Maximum 6-hour (inches)	Peak Discharge (cfs)*
Southwest Boulevard	4.72	1.48	3.64	4.04	7,130
Lamar Avenue	6.32	2.24	4.48	5.20	—
Johnson Drive	6.12	2.68	5.20	5.40	13,400
Basin Average	4.75	1.55	3.58	3.89	—

*Note: Peak discharges estimated using RAS rating curves from lower Turkey Creek design model

Figure E.5. Turkey Creek Basin Average Hyetograph for 5–6 August 2017



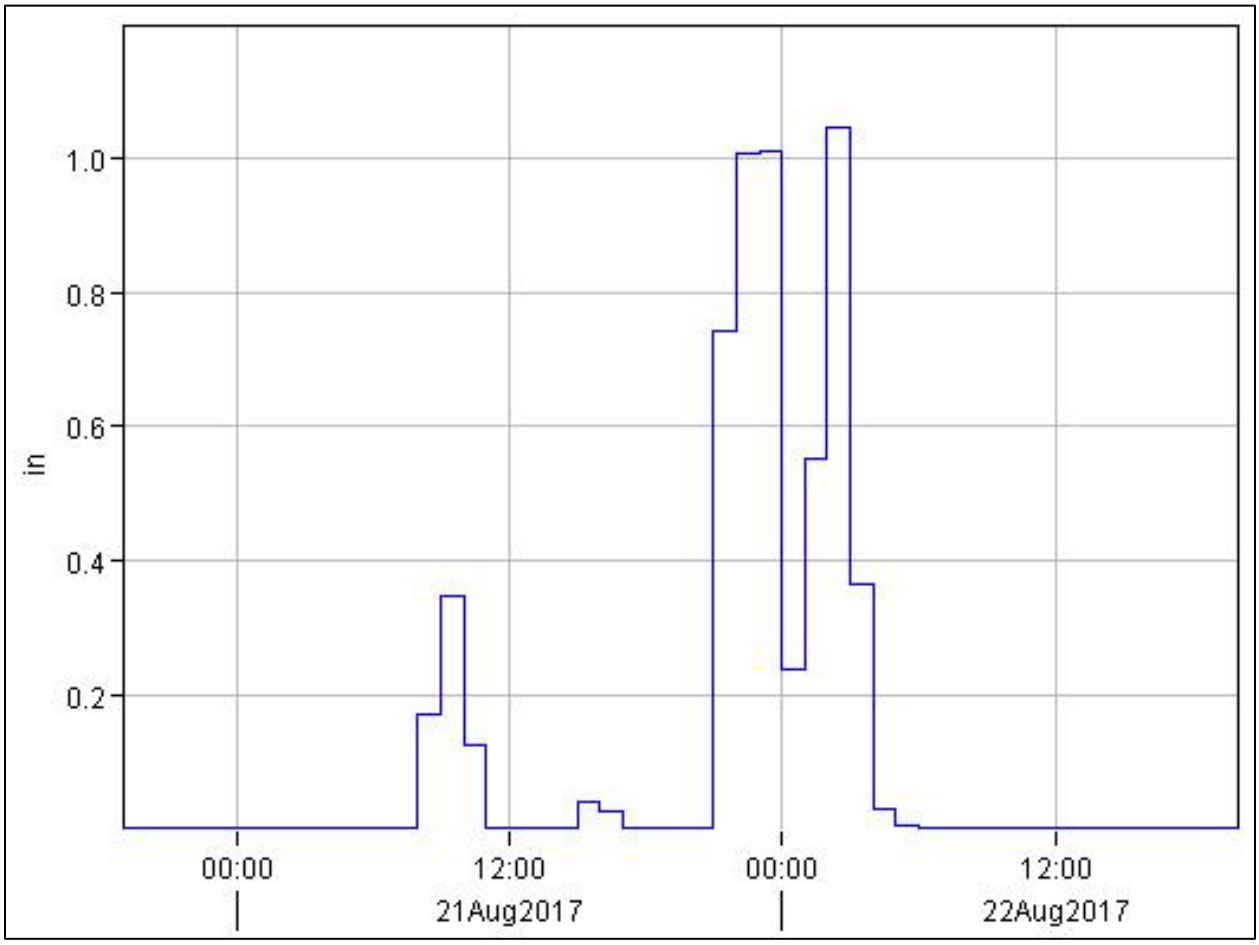
E.1.2.1.1 21–22 August 2017

The total basin precipitation for 21–22 August 2017, was 5.71 inches, and occurred between 8:00 a.m. on the 21st and 6:00 a.m. on the 22nd. The maximum hourly total was 1.05 inches between 2:00 and 3:00 a.m. on the 21st, and the maximum three-hour total was 2.76 inches between 1:00 and 4:00 a.m. on the 21st. Table E.4 contains the gage and basin rainfall totals for the event and indicates that the Southwest Boulevard gage recorded the greatest amounts. Figure E.6 shows the hyetograph for the basin.

Table E.4. Precipitation Totals for 21–22 August 2017

Location	Total (inches)	Maximum 1-hour (inches)	Maximum 3-hour (inches)	Maximum 6-hour (inches)
Southwest Boulevard	6.24	1.40	3.16	5.32
Lamar Avenue	5.24	1.08	2.72	4.64
Johnson Drive	4.96	1.08	2.84	4.56
Basin Average	5.71	1.05	2.76	4.60

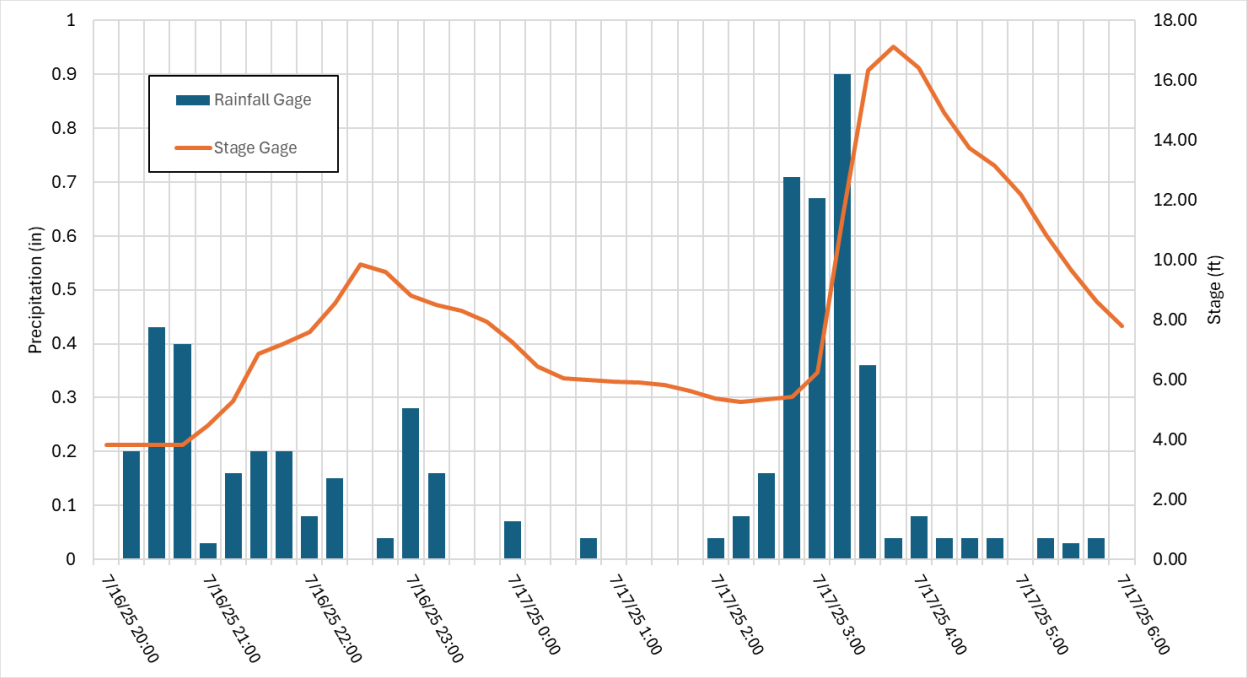
Figure E.6. Turkey Creek Basin Average Hyetograph for 21–22 August 2017



E.1.2.9. Flood of 2025

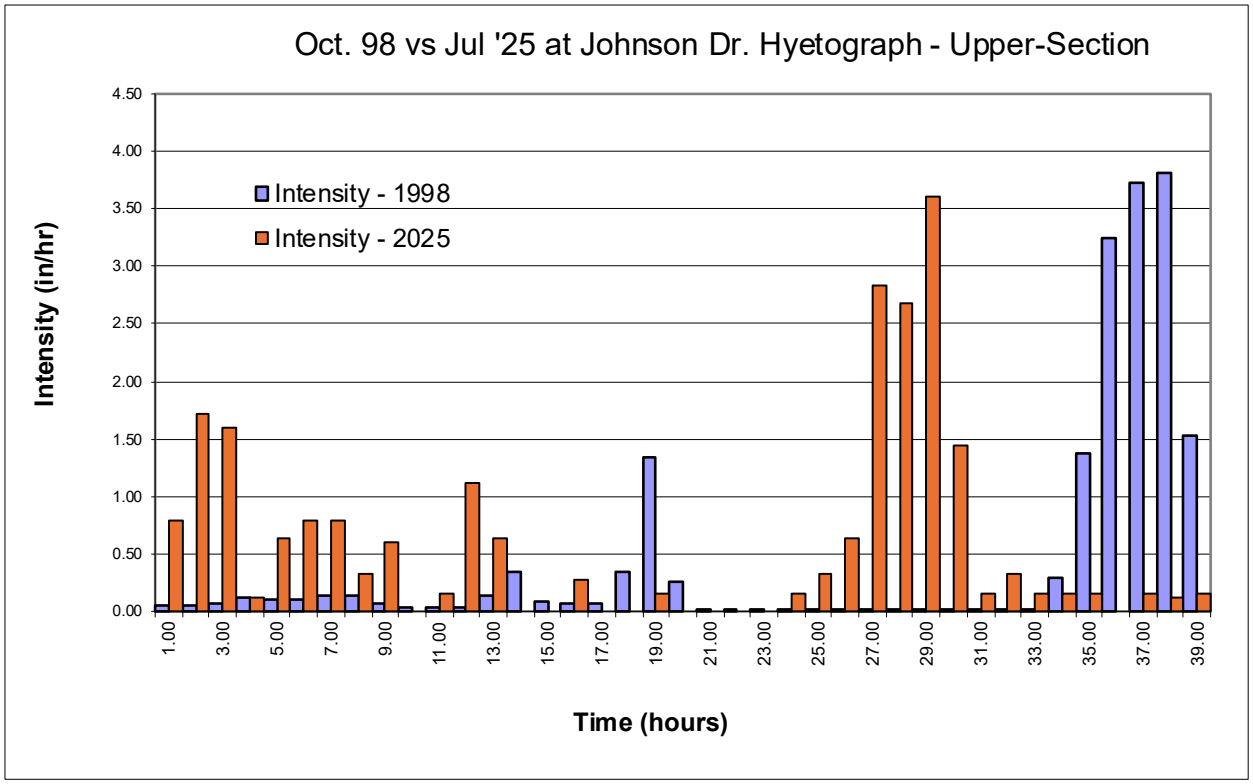
During the overnight hours of 16-17 July 2025, a storm event occurred in the Kansas City metro, including the Turkey Creek watershed. This event produced an average total rainfall of approximately 5.5 inches on the basin upstream of the project area in about 10 hours. Figure E.7 below overlays the rainfall hyetograph (at Johnson Drive StormWatch Gage TU04) with the Turkey Creek stage at the same location. This storm event produced a record gage stage of 17.24 feet at Johnson Drive within the gage's period of record (Sep1999-Present) and is second to only the 1998 flood event which occurred prior to the gage's installation.

Figure E.7. July 2025 Rainfall Hyetograph and Stage Hydrograph at Johnson Drive StormWatch Gage (TU04).



This event was quite similar to the 1998 storm in terms of total rainfall and total storm duration. Both events had lighter rainfall to start saturating the ground followed by an intense 1-2 hour burst of rainfall that drove the flooding. This is readily apparent in the previous figure which shows the very short lag time between peak rainfall intensity and peak stage. Figure E.8 provides a comparison of the rainfall hyetographs between the 1998 and 2025 storm events. Both events produced a maximum 15-min rainfall intensity of 3.8 in/hr, though the 1998 event had slightly more volume in this peak period. Based on Atlas 14 precipitation-frequency-duration tables, the 2025 event plots between a 4% AEP (1/25-year) and 2% AEP (1/50-year) storm event.

Figure E.8. Rainfall Hyetograph Comparison – 1998 vs. 2025



Despite the 1998 event producing 1.1 inches less of total rainfall, highwater marks showed a peak stage approximately 7 feet higher than the 2025 event. Figure E.9 provides a summary of the 2025 event surveyed highwater marks and their locations These were collected the following week on 24 July 2026. Information regarding the collection of the high water marks can be found in the high water mark document referenced in Section 4. The substantial difference in stage could be due to poor channel conditions in 1998 and potentially a lower conveyance capacity from the previous Johnson Drive bridge. The bridge was replaced around 2013-2014, by the Kansas Department of Transportation. Details of the previous bridge were not found, so there is uncertainty of the previous bridge played a role in producing higher stages during the 1998 event. Cursory review of aerial imagery from the late 1990s and early 2000s revealed that Turkey Creek channel was significantly overgrown. The channel had since been cleared of overgrown vegetation which notably improves channel capacity.

Figure E.9. July 2025 Surveyed Highwater Marks.

HWM #	Location	Elev (feet, NAVD88)	Notes
1	Quail Creek Park	956.75	Tributary to Turkey Creek
2	Johnson Drive Right Bank	910.59	0.35 ft above Johnson Drive gage reading
3	Johnson Drive Left Bank	910.6	0.36 ft above Johnson Drive gage reading
4	Merriam Marketplace Right Bank	907.4	Water reached corner fence line on property immediately north of pavilion
5	Merriam Marketplace Left Bank	907.36	Groundhogs angry about their homes being flooded
6	Turkey Creek Trail Tree Line	904.97	Short section of trail was underwater just north of marketplace ped bridge.
7	Waterfall Park	891.22	Pretty significant flooding of industrial area next to park due to being a low spot
8	Lamar Avenue Offramp (NB I-35)	846.42	I-35 near Lamar Avenue was between 2-4 feet underwater



E.1.3. Hydrological Development from Feasibility Study

The hydrology of Turkey Creek upstream of the Johnson and Wyandotte County line is dominated by groundwater sources during baseflow conditions and surface water runoff during precipitation events. The amount of impervious cover associated with urban development (approximately 30 percent) had increased the rate of stormwater runoff reaching the creek, thereby causing an increase in the peak discharge volumes of Turkey Creek following storm events. The amount of impervious cover within the Turkey Creek watershed has likely reduced groundwater recharge, thus reducing the baseflow in portions of Turkey Creek. Baseflow conditions of Turkey Creek at 67th Street, just upstream of the study area, were recorded at approximately 1 cubic foot per second (cfs) as part of a field study conducted by the U.S. Geological Survey (USGS) in November 2000, and July 2003 (Lee, et al. 2005). Baseflow conditions downstream of the Johnson and Wyandotte County line are dominated by discharges from the Myron Nelson Wastewater Treatment Plant Complex and typically average around 25 cfs. No active USGS gage stations exist on Turkey Creek. Stormwatch gages record stages only.

This study's existing conditions associated with land use, imperviousness, rainfall runoff, and storm drainage systems are captured in a HEC-1 model and a HEC-RAS model. Hydrologic modeling of existing conditions began on 31 May 2005, when the Larkin Group Inc. began a study titled, The Northeast Johnson County Watershed Study for Johnson County, Kansas to assist the county in FEMA Flood Insurance Rate Map (FIRM) revisions for three or four of the smaller and well-developed watersheds of the county (Larkin Group Inc. 2005). Larkin Group Inc. produced the study for the Storm Management Program of Johnson County, Kansas, Public Works. As part of the study, Larkin Group Inc. developed a hydrologic model using the HEC-1 software. Many of the existing enclosed storm drainage systems needed specific assumptions and analytical processes, specifically for setting up an open channel flow tool, to best characterize the watersheds in northeastern Johnson County, which is the portion closest to the center of the Kansas City metropolitan area.

The HEC-1 model produced by Larkin Group Inc. included the complete watershed contributing to Upper Turkey Creek. During the early stage of the Feasibility Study (FS), planners and hydraulic engineers in the Kansas City District decided to use Larkin Group Inc.'s HEC-1 model as the official existing conditions hydrologic model for the Johnson County, Kansas, study.

Gages help to establish existing conditions by tying possible flows to statistical records. One USGS stream gage lies within the northeastern watershed. However, this gage, identified as Gage 06893557, is on Brush Creek located near Ward Parkway in Kansas City, Missouri. The gage monitors Brush Creek downstream of the northeastern watershed study area. USGS has reported data for this gage since 1999, and the average annual mean stream flow for this gage is 10.31 cfs. Based on conversations with Don Wilkinson of USGS in 2002, the 100-year flow at this location has been estimated at approximately 16,000 cfs based on variety of source data, but not enough data has been collected from the gage to verify this estimate. Although this gage is not directly related to Turkey Creek, it was an important calibration tool for Larkin Group Inc. during the preparation of the Johnson County, Kansas, HEC-1 model. Johnson County Public Works maintains a stream gage network, which is part of the Stormwatch system (City of Overland Park, 2012) and available on the internet, consisting of five gage stations within the area included in the northeast Johnson County, Kansas, watershed study area. Two of these stream gage stations are within the Turkey Creek watershed (Table E.5). These gage stations are not set up for streamflow measurement; they collect precipitation and water level data.

Table E.5. Turkey Creek Watershed Gages in the Stormwatch Gage Network

Gage	Location	Oldest Available Record
3010	Johnson Drive	14 September 1999
3020	65th Street	23 March 2000

The large majority of the northeast Johnson County watershed study area is fully developed and urbanized, including the Turkey Creek watershed. A quantitative analysis of existing land use was not required for the study because the required hydrologic data was acquired from a detailed study of impervious surfaces. However, the study area can be roughly characterized as follows:

- About 36 square miles of the northeast portion of the Johnson County study area are urbanized and fully developed. About 0.5 square miles of undeveloped land planned for future development are present in the Lake Quivira watershed in western Shawnee, Kansas. This undeveloped land is either pasture or woodlands. Open areas are also present in the community of Lake Quivira, but the planned use for those areas is to remain wooded open space.
- About 60 to 70 percent of the urban area is residential usage (subdivisions of single-family homes, multi-family dwellings, and apartment complexes).
- About 20 to 25 percent of the land is used for commercial, business, or institutional uses, including retail shopping, restaurants, entertainment, offices, hospitals, schools, churches, and synagogues.
- Less than 5 percent of the urban land is used for light industries.
- The remainder of the urban land is open space (such as floodplain areas), parks, golf courses, or transportation corridors.

- The average percent imperviousness for the Feasibility Study area was approximately 38 percent.
- According to the 2021 National Land Cover Database (NLCD) impervious layer dataset, Turkey Creek watershed is approximately 42.2% impervious surface.

The basin experienced significant development in the 1980s and 1990s and used up most remaining developable land. This contributed to increased Turkey Creek stormwater flows; however, no flows are expected to increase in the future due to additional impermeable surfaces.

E.1.4. Hydraulic Model Development

The Stantec 2D model was adopted for use in the Upper Turkey Creek design project. The hydraulic model was adopted because it had undergone some level of calibration and validation with historic rainfall events. This 2D model was developed on behalf of FEMA and Johnson County, Kansas, with expectations to become the new effective FEMA model for the Turkey Creek watershed. Using the same base model would be ideal to maintain consistency with FEMA for possible future levee accreditation. The Stantec model was developed to perform both hydrologic and hydraulic computations, providing rainfall runoff flood mapping for interior drainage analysis as well as the hydraulic analysis of the Turkey Creek channel from local inflows.

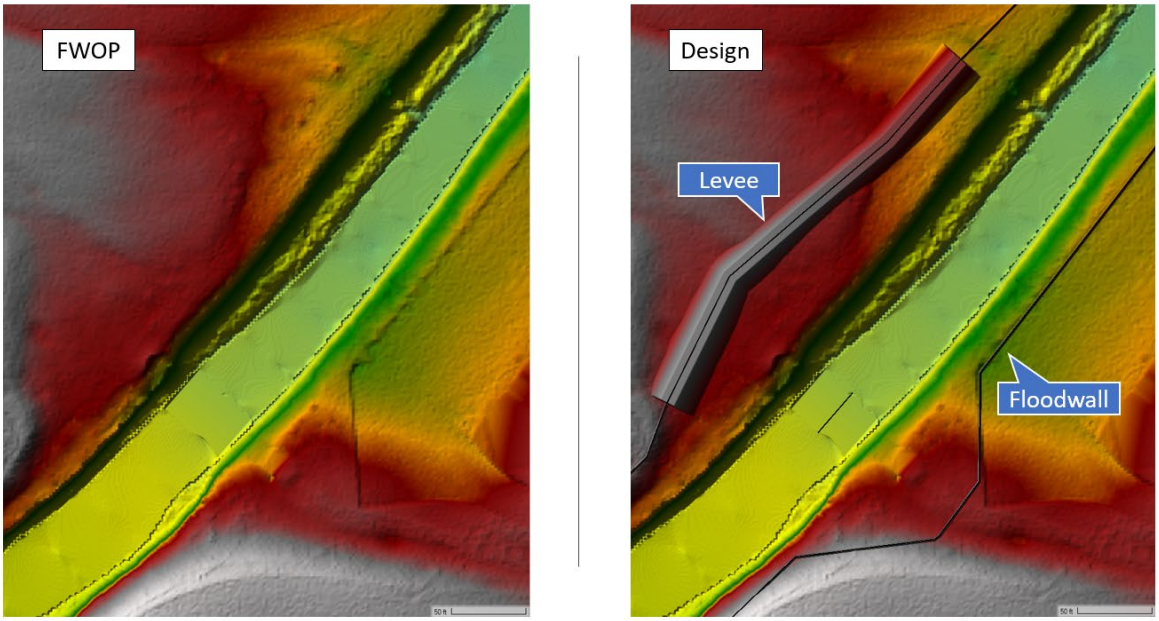
E.1.4.1. Terrain Modifications

Terrain modifications were used to represent proposed levees and floodwalls within the project area as well as modifications to the channel to account for bridge widening. With these modifications, two copies of the terrain were utilized. One copy represented the future without project conditions (FWOP), and one represented the 50% design. Table E.6 provides names and descriptions of the terrains used in the hydraulic model. An example of the FWOP vs. 50% design conditions is shown in Figure E.10.

Table E.6. Stantec 2D Model Terrain Descriptions.

Terrain Name	Description
Turkey	Stantec model existing conditions watershed terrain
Turkey_50%_Design	Stantec model watershed terrain with proposed levee/floodwall

Figure E.10. Terrain Modifications – FWOP vs. Design.



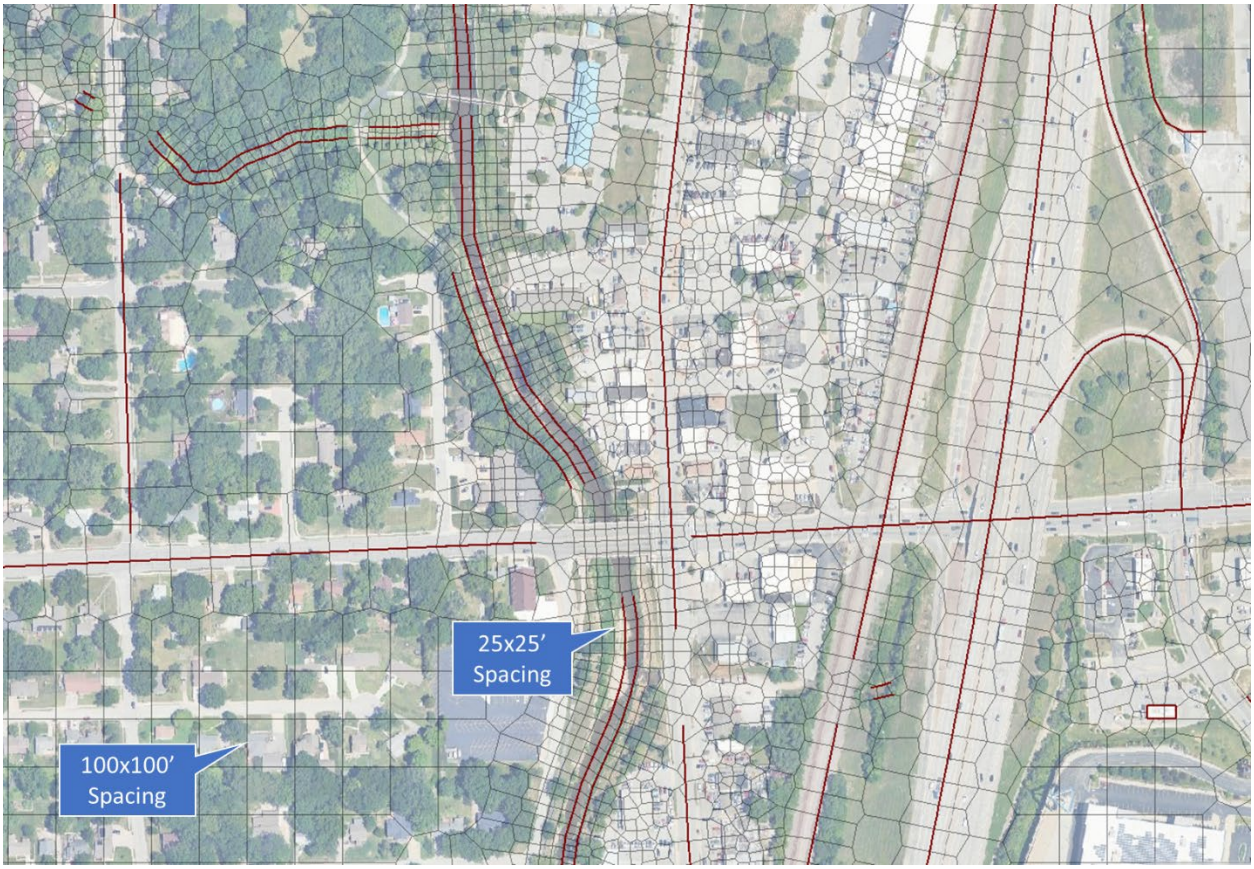
E.1.4.2. 2D Mesh

As mentioned previously, the hydraulic model is configured as fully 2D. The project area is represented by 2D cells of varying sizes along with refinement areas and breaklines to enforce instances of high ground (levees, floodwalls, roadways, etc.) and flow direction. Cell sizes outside of the channel and floodplain area generally used a 100x100 foot spacing. Cell sizes in the channel generally targeted a 25x25 foot spacing so the velocity variance, superelevation, and flow direction could be better represented. Cell spacing in the floodplain varied between 25 feet to 100 feet to represent roadways, buildings, and other features of the interior area. Figure E.11 provides a sample of the 2D mesh within the project area.

Breaklines were utilized in the Turkey Creek channel and other minor tributaries to better align cell faces perpendicular to flow direction. Breaklines were placed along major roadways which typically act as minor high ground. While not shown in the figure, breaklines were also placed on top of the floodwall and levee crest alignments to enforce the proposed levee heights.

Cells were generally adjusted to encompass building or structure footprints so that representative roughness values could be assigned to these locations. This is discussed in further detail in Section E.1.4.3.

Figure E.11. Example of 2D Mesh in Project Area.

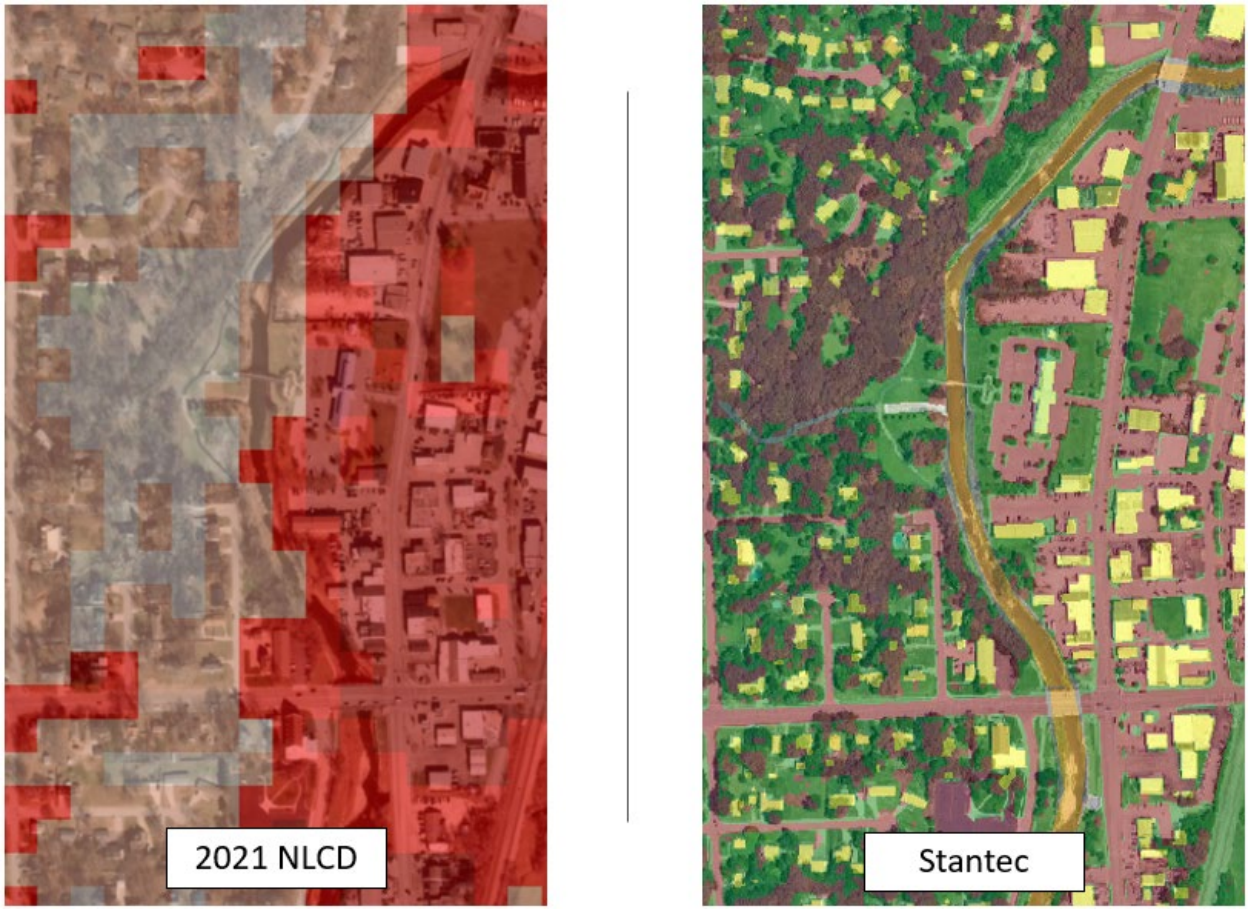


E.1.4.3. Land Cover Data

Land cover data was developed by Stantec as part of the 2D hydraulic model using recent aerial imagery provided by Johnson County's Automated Information Mapping System (AIMS) website. The NLCD land cover layer is typically used for 2D hydraulic modeling. The NLCD land cover layer uses a 30-meter resolution which is too coarse for the relatively smaller project area of Upper Turkey Creek and lacks the necessary detail required of an extensively urbanized watershed.

Figure E.12 below provides a side-by-side comparison of the Stantec developed layer and the NLCD layer. Stantec's land cover layer uses very detailed classification boundaries to acknowledge roadways/parking lots, buildings, channels, trees, and grass cover. The NLCD layer is too coarse to recognize Turkey Creek as open water and occasionally represents the channel as high intensity developed area. While edits could be made to the NLCD layer, the Stantec land cover layer was deemed more suitable for use. This is because of the greater detail in land cover type boundaries and usability improvements to make sweeping changes to roughness coefficients for any given land cover type.

Figure E.12. Land Cover Layer Comparison.



Land Cover types were broken up into the following categories to further discretize from the typical NLCD land cover types and assigned Manning's n values as shown in Table E.7.

Table E.7. Manning's N Values.

Land Cover Type	Manning's N Value
Barren Land	0.025
Buildings	0.5
Grass	0.03
Open Water	0.035
Pavement	0.015
Trees	0.06
Natural Channel	0.035
Concrete Channel	0.015
Riprap	0.045
Turkey Creel Channel1	0.032-0.035

1Calibration region to capture full channel widths of creek. Uses same value as Natural Channel

E.1.4.4. Infiltration Layer

An infiltration layer was also developed by Stantec in their initial modeling development. The Soil Conservation Service (SCS) Curve Number (CN) method was utilized. Development of the infiltration layer used aerial imagery and the Soil Survey Geographic Database (SSURGO) Soil Data layer. For more information, the Stantec modeling report can be viewed in Appendix E.7. It should be noted that infiltration layer is only used when running 2D rain-on-grid simulations. Given that the levee design hydraulic analysis did not use 2D rain-on-grid, the associated infiltration layer had no impact on the hydraulic results.

E.1.4.5. Boundary Conditions

The most notable modification to the Stantec 2D model was switching from the rain-on-grid boundary condition and importing the channel inflow boundary conditions used in the USACE 2D RAS model. This was necessary to maintain consistency with previous methodology of hydraulically routing the frequency flows and ensure flow volume remained the same as before. Using 2D rain-on-grid has notable limitations for purpose of developing channel water surface profiles for flood protection designs, especially in such urbanized areas with expansive stormwater networks. Stormwater pipes can significantly reduce time of concentration compared to overland flow which the 2D RAS model is limited to. Additionally, storm networks can route flow to different channel inflows points than what would occur naturally.

Boundary conditions include the upstream inflow location for the Turkey Creek channel on the upstream side of 67th Street bridge, a downstream boundary located just east of Roe Lane, and lateral inflow locations as dictated by the HEC-1 output locations. The downstream boundary used the normal depth condition with a friction slope of 0.00561 based on the channel bed slope in this location.

Boundary conditions were added within the 2D mesh to represent local inflows from tributaries and notable stormwater outlets into Turkey Creek according to output locations set in the HEC-1 model. Boundary condition names correspond to HEC-1 output hydrograph names for consistency. Table E.8 summarizes the boundary conditions ordered from upstream to downstream by river mile as well as general location.

Table E.8. Inflow Boundary Conditions for USACE 2D Model.

Boundary Condition	River Mile	Location
*TC-M80 XS 4.234 TCC23	4.234	67th Street bridge
TC94	3.990	1350ft downstream of 67th Street
TCT71	3.726	Tributary on upstream side of SMP bridge
TC-T59	3.670	Tributary on downstream side of SMP bridge
TC66	3.670	Tributary on downstream side of SMP bridge
TC79	3.153	400ft upstream of Johnson Drive
TCT50-TC64	3.112	220ft upstream of Johnson Drive
TC-T40	2.883	Upstream side of Merriam Marketplace bridge
TC57	2.836	160ft downstream of Merriam Marketplace bridge

Boundary Condition	River Mile	Location
TC41	2.502	640ft downstream of Merriam bridge
TC48	2.336	1500ft downstream of Merriam bridge
TC38	2.185	600ft upstream of Hwy I-35 bridge
TC19	2.090	Tributary on upstream side of Hwy I-35 bridge
TC-T30	2.090	Tributary on upstream side of Hwy I-35 bridge

*Note: Upstream boundary condition

E.1.4.6. Bridges

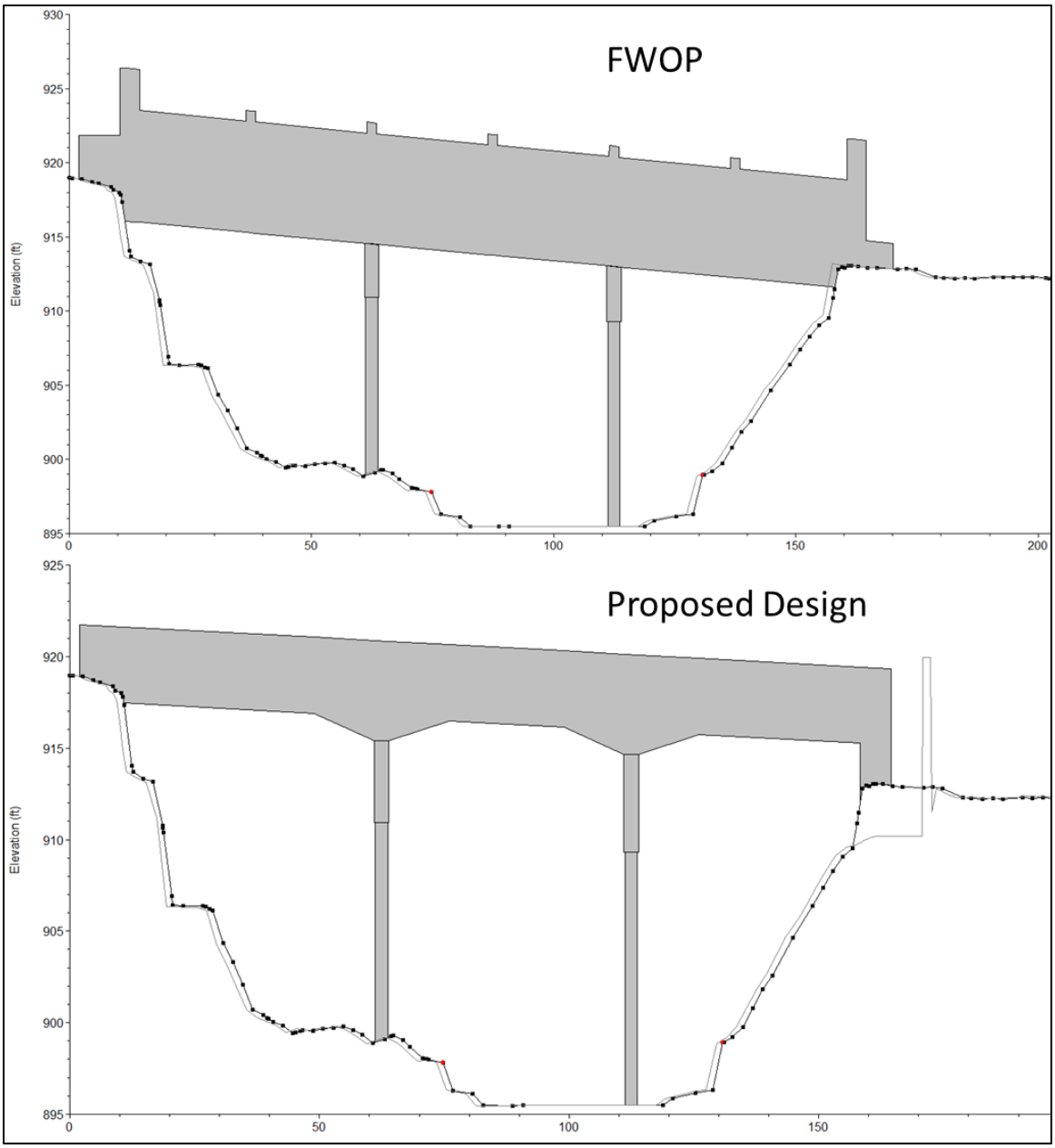
Bridges were added into the 2D model as 2D connections. Within the modeling extents, ten bridges spanning the Turkey Creek channel were identified. Bridges consist of vehicular, pedestrian, and railroad bridges. The location and description of bridges are summarized in Table E.9. Dimensions and heights for most bridges were based on drawings provided from Johnson County. The Merriam Marketplace bridge and DS Bus Line bridge geometries were based on USACE surveys. The 2D Equation Domain was selected for the overflow computation method for all bridges. Additionally, all bridges used a weir coefficient value of 2.

Table E.9. Bridges within Stantec FEMA 2D Model Extents.

Bridge (Model Ref)	Description	Approx. River Mile
67th Street Bridge (TUR_2600)	Vehicular bridge along 67th Street in Merriam, KS	4.25
Shawnee Mission Parkway Bridge (TUR_2300)	Vehicular bridge along SMP/63rd Street	3.73
Shawnee Mission Parkway Pedestrian Bridge (TUR_2200)	Pedestrian Bridge approx. 600ft downstream of SMP	3.57
Johnson Drive Bridge (TUR_1900)	Vehicular bridge along Johnson Drive	3.07
Merriam Marketplace Pedestrian Bridge (TUR_1700)	Pedestrian bridge connects TurkeyCr Streamway Trail (left bank) to Merriam Mrkt/Commercial area (right bank)	2.87
Merriam Drive Bridge (TUR_1500)	Vehicular Bridge along Merriam Drive	2.66
DS Bus Line Bridge (TUR_1400)	Small vehicular bridge connecting school bus parking area to alternate access to Merriam Drive	2.49
BNSF Trestle Bridge (TUR_1100)	Railroad bridge for BNSF adjacent to I-35	2.07
Highway I-35 southbound on-ramp (TUR_1000)	Vehicular bridge serving as onramp to SB I-35 from Antioch Road	2.02
Highway I-35 (TUR_900)	Vehicular bridge for Hwy I-35	2.00

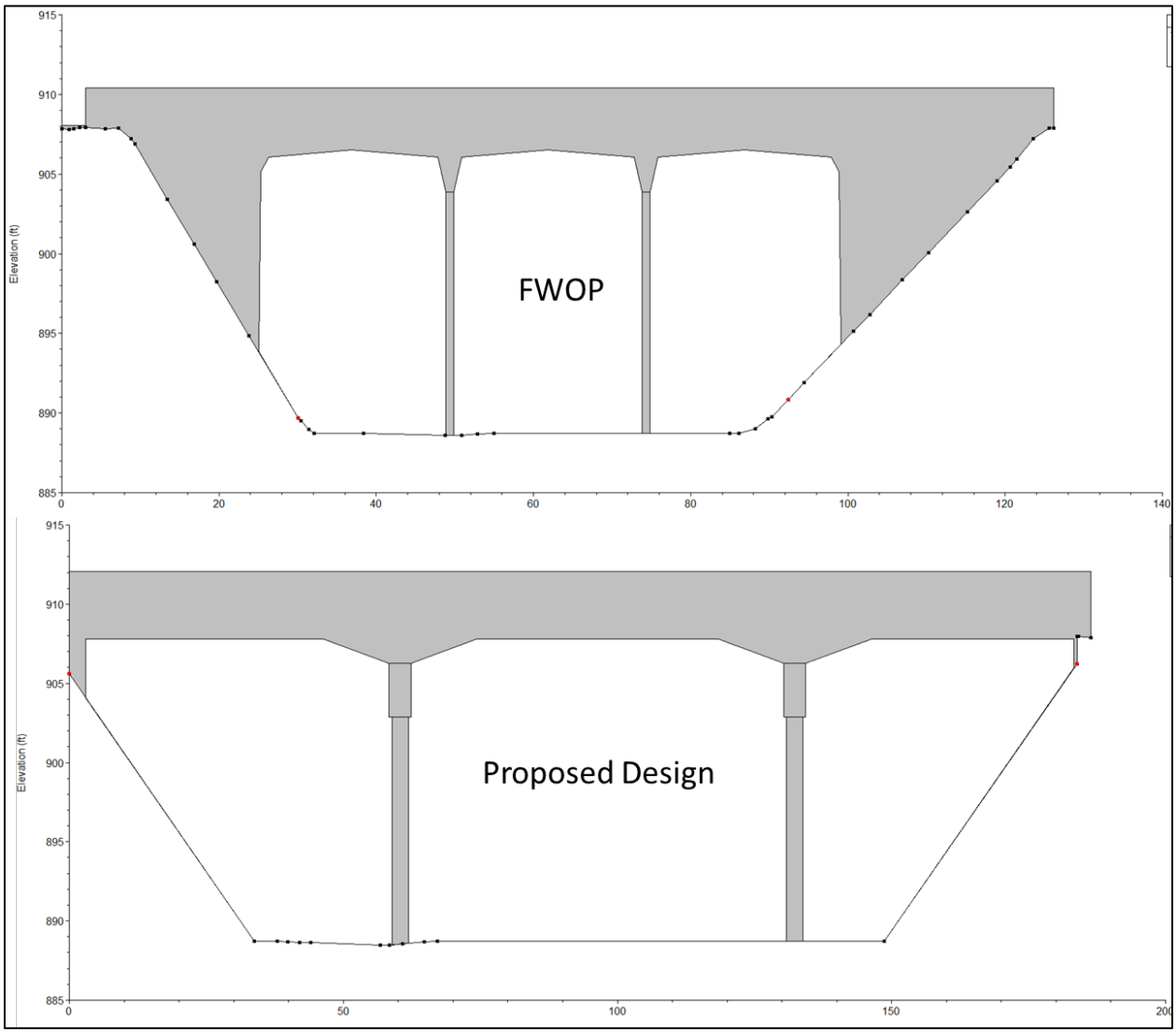
Part of the 50% design includes replacement of both the Johnson Drive bridge and Merriam Drive bridge. These two bridges were updated in the hydraulic model for the 50% design geometry. Structural calculations showed that the existing Johnson Drive bridge would experience flotation during the 1% AEP design event. The proposed replacement is a Kansas Department of Transportation (KDOT) standard slab bridge. Additionally, the bridge is being raised so that it acts as part of the flood protection system and eliminates the need for closure structures on the left and right sides of the bridge deck. The proposed Johnson Drive bridge replacement geometry is shown in Figure E.13.

Figure E.13. Johnson Drive Bridge Replacement.



The Merriam Bridge will be replaced due to the low conveyance capacity of the current bridge. The openings are not aligned with flow direction, and the bridge overtops during the 1% AEP event. Like Johnson Drive, a standard KDOT slab bridge is proposed to replace the existing bridge at Merriam Drive. It should be noted that the channel will be widened to accommodate a longer bridge span which helps further increase flow conveyance. Figure E.14 provides a comparison between FWOP and the 50% design.

Figure E.14. Merriam Drive bridge replacement.



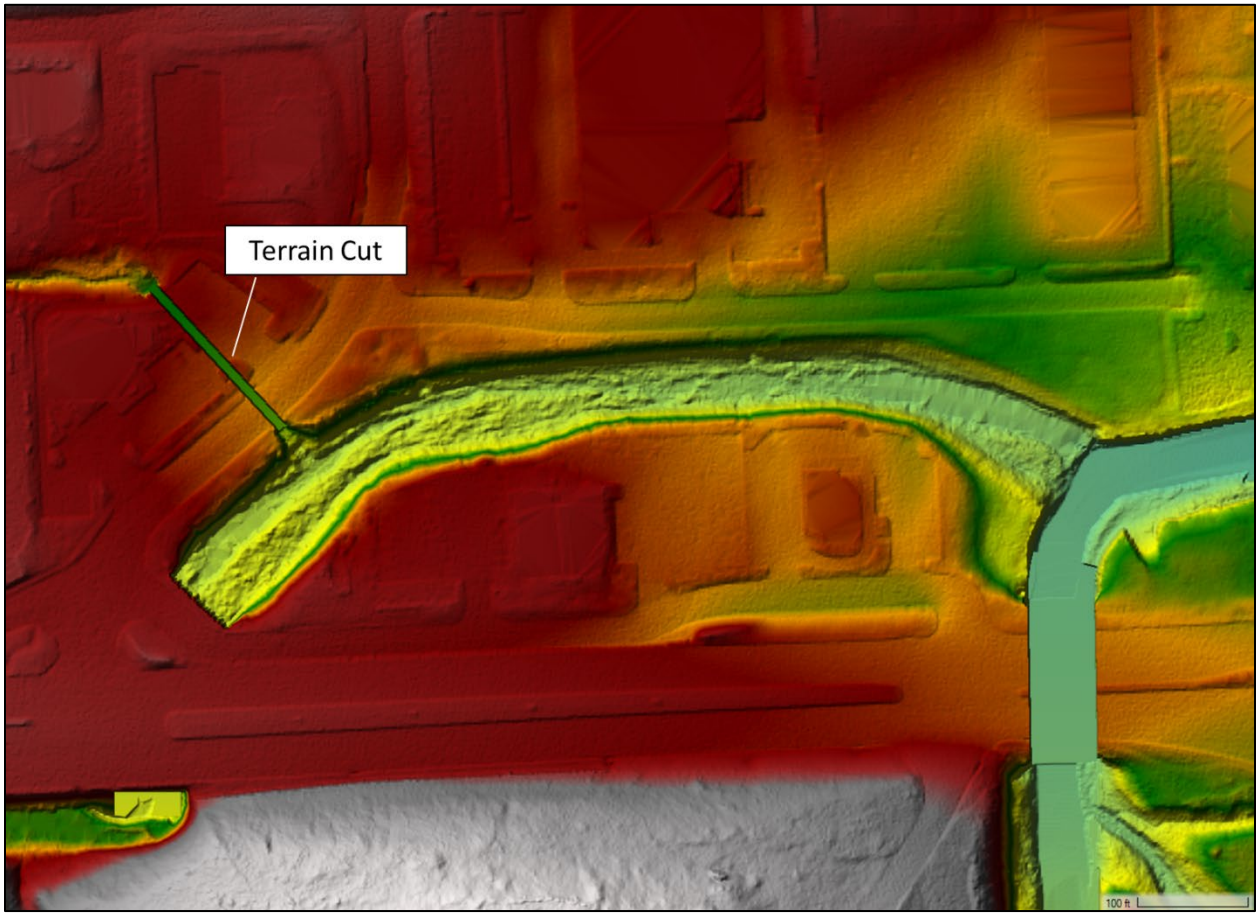
E.1.4.7. Miscellaneous Refinements

Other refinements to the Stantec 2D model include refinements to cell alignments both in the channel and floodplains. Cell alignment was adjusted to better represent flow direction in the channel. This improved model stability and runtime while providing more representative results.

Any terrain modifications used to assist with rain-on-grid modeling were removed. These included cuts into the terrain to allow rainfall runoff to route towards Turkey Creek in place of

modeling storm water networks. An example of this is shown below at the tributary just north of Shawnee Mission Parkway.

Figure E.15. Example of Removed Terrain Modification.



E.1.5. Levee Profile Development

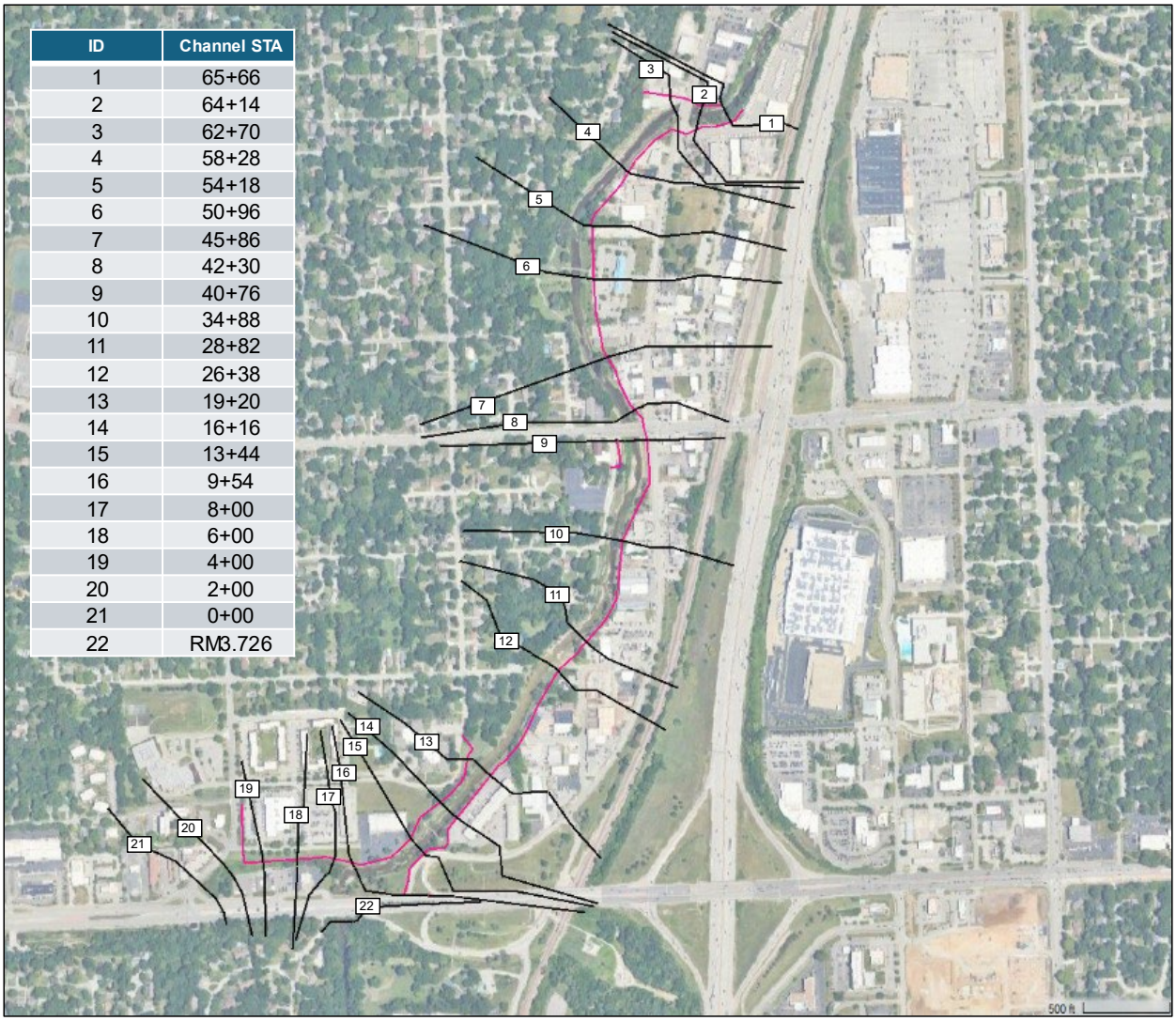
Development of the levee profile was a multi-stage effort. Levee height was determined through risk-based analysis outlined in EM 1110-2-1619. Hydrologic Engineering Center Flood Damage Reduction Analysis (HEC-FDA) software was used to determine levee height with a required assurance level based on HEC-RAS model outputs. This included the following data for both the FWOP and with project conditions at each index location.

- Unsteady HDF plan files
- Frequency Functions (Flow Frequency Curve)
- Stage Transform Functions (Rating Curve)
- Lateral Structures (Levee Alignment Shapefile)

E.1.5.1. Index Locations

Before outputting hydraulic data from HEC-RAS, index locations were identified. More index locations are typically needed for purposes of levee design, especially in highly dynamic systems such as Turkey Creek. More index locations allow the levee profile to capture inflection points in the channel water surface profile, particularly upstream and downstream of bridges, and at notable changes to the bed slope. The initial set of index locations used the cross-sections from the previously developed 1D hydraulic model. This allowed for easy points of comparison when looking at changes to levee height from previous iterations. Additional index locations were added along the tributary at the upstream portion of the project. Figure E.16 below provides a visual of the index locations used for HEC-FDA. Additionally, the levee alignment shapefile for both the left and right banks was provided.

Figure E.16. Index Locations.



E.1.5.2. Unsteady Plan Sets

HEC-FDA requires a set of frequency plans to develop stage-transform function (rating curves) and associated uncertainty bounds at each index location. This consists of the 50%, 20%, 10%, 5%, 2%, 1% 0.5%, and 0.2% AEP flows. The set of 8 flow frequency plans was needed for the FWOP and with design conditions. Additionally, 4 additional sets of plans were run to develop stage uncertainty associated with each frequency flow in FWOP and with design conditions. Generally, the Manning's n value for the FWOP and with design conditions was modified +/- 30% to represent uncertainty in the stages. These plans are discussed in detail in Section E.1.5.4**.** In total, 48 plans were necessary to perform the HEC-FDA analysis to develop the levee profile. Despite a stage uncertainty being developed for each frequency for both the FWOP and with design conditions, the maximum computed value was used for all frequencies when input into HEC-FDA to reduce the risk of underestimating stage variability. Especially for the 1% AEP design event. This was 0.460-feet and 0.468-feet for the FWOP and with design conditions, respectively.

E.1.5.3. HEC-RAS Model Outputs

Flows and water surface elevations were pulled to develop flow frequency curves and rating curves at each index location. It should be noted that flow was measured using the index cross-sections across the entire floodplain to properly capture all conveying flow. Conversely, the water surface elevation was only measured within the extent of the channel, even for floodplain wide flows. This was necessary to ensure that the computed levee height would have the proper level of assurance compared to the design water surface profile in the channel. To take superelevation into account, water surface elevations were manually pulled from the model outputs on the left and right banks. Figure E.17 provides an example at channel STA 64+14 which is immediately downstream of the Merriam bridge. At this location, the right bank (outside channel bend) experienced a superelevation of approximately 0.8-feet relative to the left bank and 0.4-feet relative to the center of channel. An example of the flow and stage data is provided in Table E.10 at the same location which illustrates the difference in water surface elevation between left and right bank due to superelevation.

Figure E.17. Example of Water Surface Superelevation in Channel.

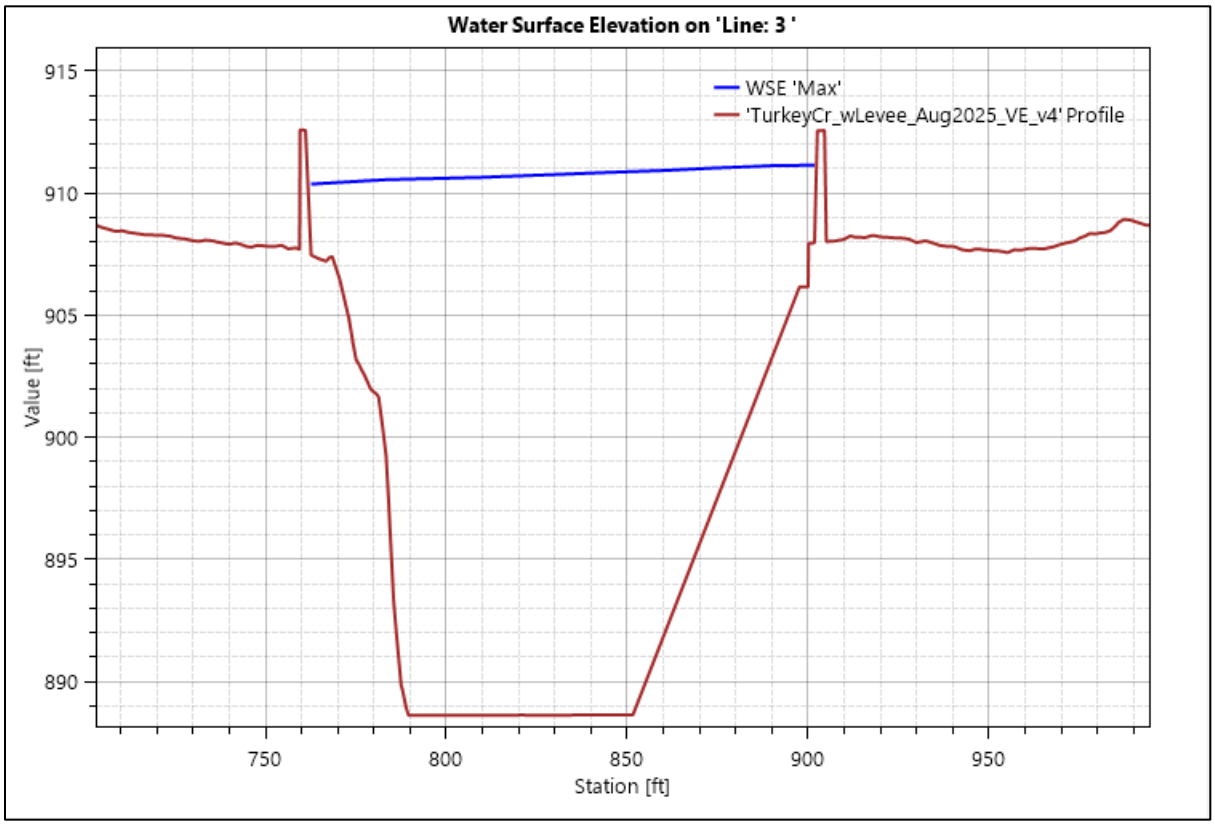


Table E.10. Example H&H Flow and WSE Input Data for HEC-FDA.

AEP	Flow	Water Surface Elevation (feet)		Δ WSE (ft)
		Left Bank	Right Bank	
50%	6823	901.94	902.42	0.48
20%	9185	904.12	904.76	0.64
10%	10930	905.57	906.35	0.78
5%	14268	907.88	908.37	0.49
2%	16502	909.20	909.71	0.51
1%	18674	910.38	911.15	0.77
0.5%	20405	911.09	912.29	1.20
0.2%	21879	911.48	912.60	1.12

The unsteady plan HDFs were needed to develop stage-damage functions for impacted structures in the FWOP and with design conditions. Stage-damage curves were not needed for computing levee height but were necessary for the economic analysis performed in HEC-FDA.

E.1.5.4. HEC-FDA Data Inputs

Stage Uncertainty

A sensitivity analysis was conducted during the levee profile design process to account for uncertainty in the hydraulic modeling. This process is outlined in EM 1110-2-1619, Section 5-7. In the absence of observed data, Manning's n values can be varied (typically +/- 30%) in the hydraulic model to develop and upper and lower confidence bound in frequency stages.

Manning's n values were varied +/- 30% for both the with and without project conditions. The difference in feet and standard deviation was computed to represent the stage uncertainty using methods outlined in EM 1110-2-1619. Table E.11 summarizes the average stage uncertainty across the project reach for each frequency. It should be noted that the results show stage uncertainty generally decreasing with smaller frequencies (larger events). This hydraulic behavior can be attributed to typical behavior of a stage-discharge rating curve where equal increases in flow result in smaller and smaller increases in stage as more of the floodplain is inundated. All HEC-FDA input data is available in Appendix E.3.

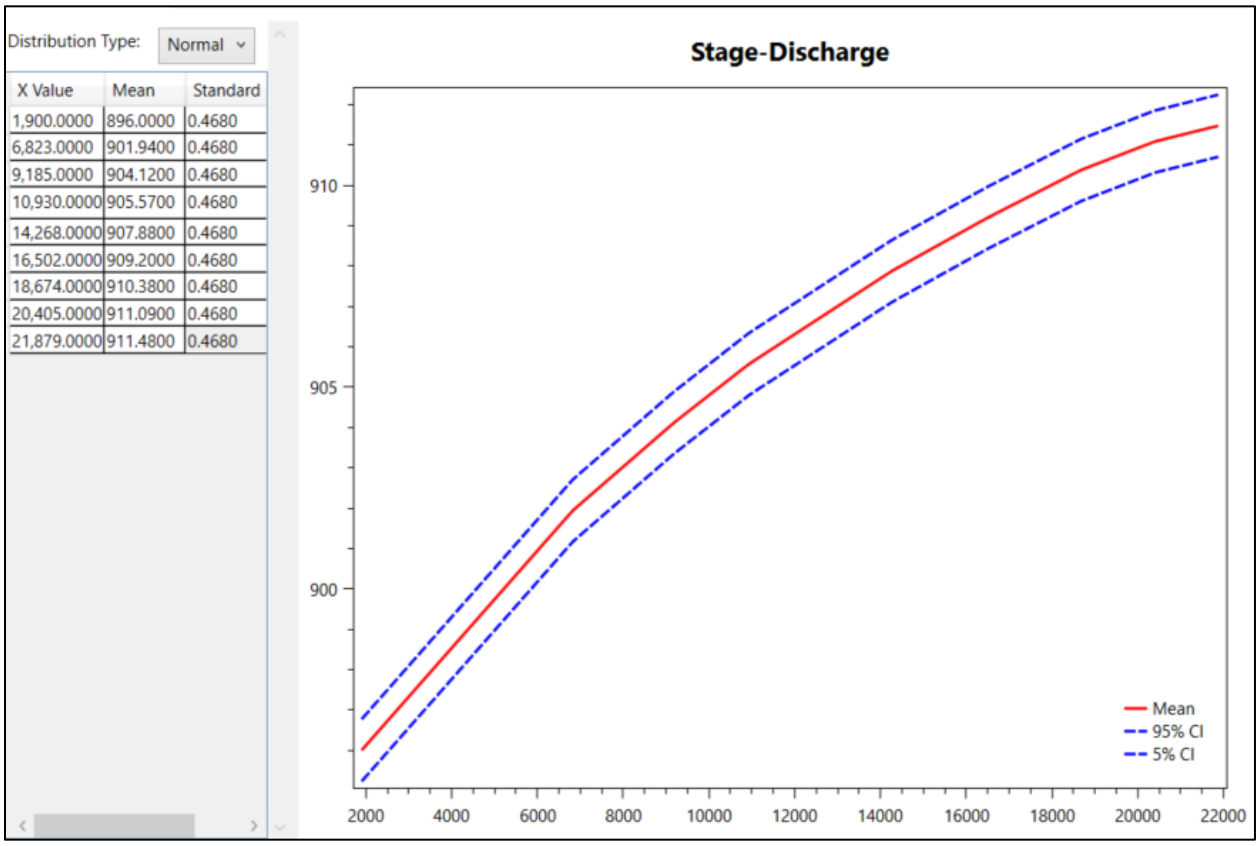
Table E.11. Manning's N Sensitivity Test for Stage Uncertainty.

AEP	FWOP		50% Design	
	Δ WSE (feet)	Standard Deviation (feet)	Δ WSE (feet)	Standard Deviation (feet)
50%	1.84	0.460	1.87	0.468
20%	1.46	0.364	1.67	0.417
10%	1.72	0.429	1.66	0.414
5%	1.19	0.298	1.59	0.397
2%	1.02	0.254	1.53	0.382
1%	0.85	0.213	1.32	0.329
0.5%	0.74	0.185	0.93	0.234
0.2%	0.62	0.154	0.54	0.136

Stage-Transform Relationship

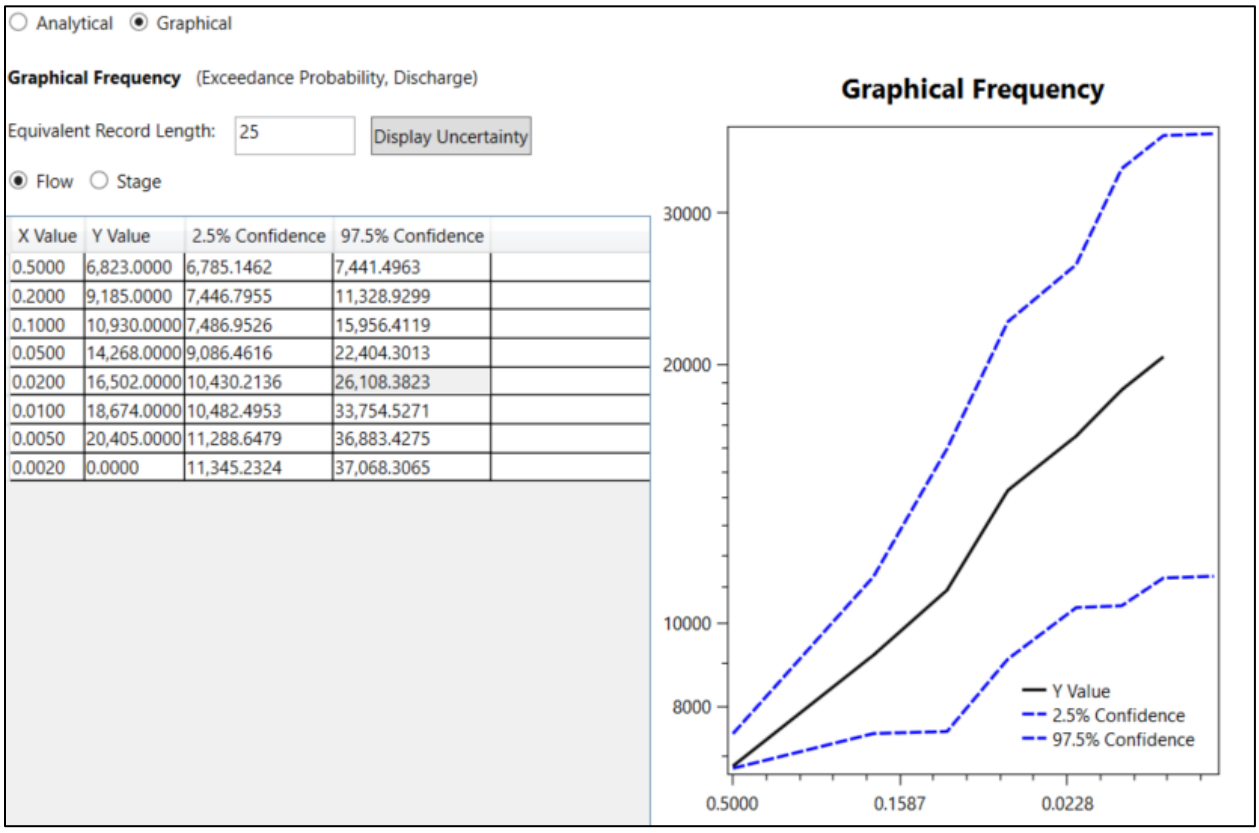
The stage-transform function (stage-discharge rating curve) uses a normal distribution type. This allows the user to input the stage-discharge for each frequency as well as the computed standard deviation. As mentioned previously, only the maximum respective value for the FWOP and with design conditions was used. The left bank index point at STA 64+14 is shown below as an example in Figure E.18. The data displayed is for the with design condition.

Figure E.18. STA 64+14 LB Stage-Discharge Curve.



The frequency function (flow-frequency curve) used the graphical method in the absence of observed data to develop a flow-frequency curve through HEC-SSP. The graphical method involves adding the peak flows measured at an index point for each corresponding frequency into a table. An equivalent record length must then be chosen for HEC-FDA to compute 2.5% and 97.5% confidence bounds. The equivalent record length was chosen to be 25 years to represent length of record of the Johnson Drive StormWatch stage gage. While this gage does not record flow, sub-daily observed stages at this location could be utilized to inform record length. Figure E.19 provides an example at index location STA 64+14.

Figure E.19. STA 64+14 Flow Frequency Curve.



E.1.6. Sensitivity Analysis

Sensitivity analyses were conducted to bracket uncertainty in the hydraulic modeling as well as understand the effect of parameter assumptions on the modeling results. In addition to the stage-uncertainty analysis discussed in Section E.1.5.4, a sensitivity analysis was also performed the downstream boundary condition of the hydraulic model.

E.1.6.1. Normal Depth Slope

The normal depth friction slope at the downstream boundary of the model was varied to evaluate any effect on water surface in the project area upstream. It should be noted that the selected friction slope for the normal depth boundary condition was estimated using the bed slope at the downstream portion of the model. The friction slope was factored up and down by 2. Table E.12 provides a summary of water surface elevations at the downstream boundary and at Merriam Bridge for the selected friction slope and the slopes used for the sensitivity test. Factoring the base friction slope up or down by 2 has a negligible effect on the water surface at the project location. Changes to water surface elevation were less than a tenth of a foot.

Table E.12. Normal Depth Sensitivity Analysis

Scenario	Friction Slope (ft/ft)	WSE at Downstream Boundary	WSE at Merriam Bridge (ft)
Base	0.00561	811.8	910.9
2x Slope	0.01122	811.8	910.9
0.5x Slope	0.00231	811.8	910.9

E.1.6.2. Bridge Modeling Approach

The high flow methods available in the HEC-RAS bridge modeling approach are Energy method and the Pressure and/or Weir method. The HEC-RAS hydraulic reference manual generally recommends selecting the Pressure and/or Weir method if flow is being constrained down to only the bridge opening without flanking or overtopping. For instances where flow conveyance is not limited to pressurized flow through the opening, the Energy method is recommended. These high flow method selections were made for each individual bridge based on the if the 1% AEP design event would likely flank or overtop the bridge deck. Pressure and/or Weir method was selected for the Johnson Drive (TUR_1900), Merriam Drive (TUR_1500), BNSF Railroad (TUR_1100), I-35 On-Ramp (TUR_1000), I-35 (TUR_900).

A sensitivity test was conducted by switching the high flow method for all bridges within the study area to Energy Method only. Analysis of results showed that high flow methods impacted the 1% AEP water surface profile around Johnson Drive and Merriam Drive bridges as shown in Figure E.20. Using only Energy for the high flow method resulted in a decrease in water surface elevation upstream of Johnson Drive bridge (from STA 43+50 to 65+50) and an increase in water surface elevation upstream of Merriam Drive bridge (from STA 65+50 to 86+00). Table E.13 below provides the maximum and average change in water surface elevation over these two stretches. All other portions of the water surface profile changed by less than 0.1 feet.

Figure E.20. 1% AEP Profile Comparison – 50% Design vs Energy Method Only

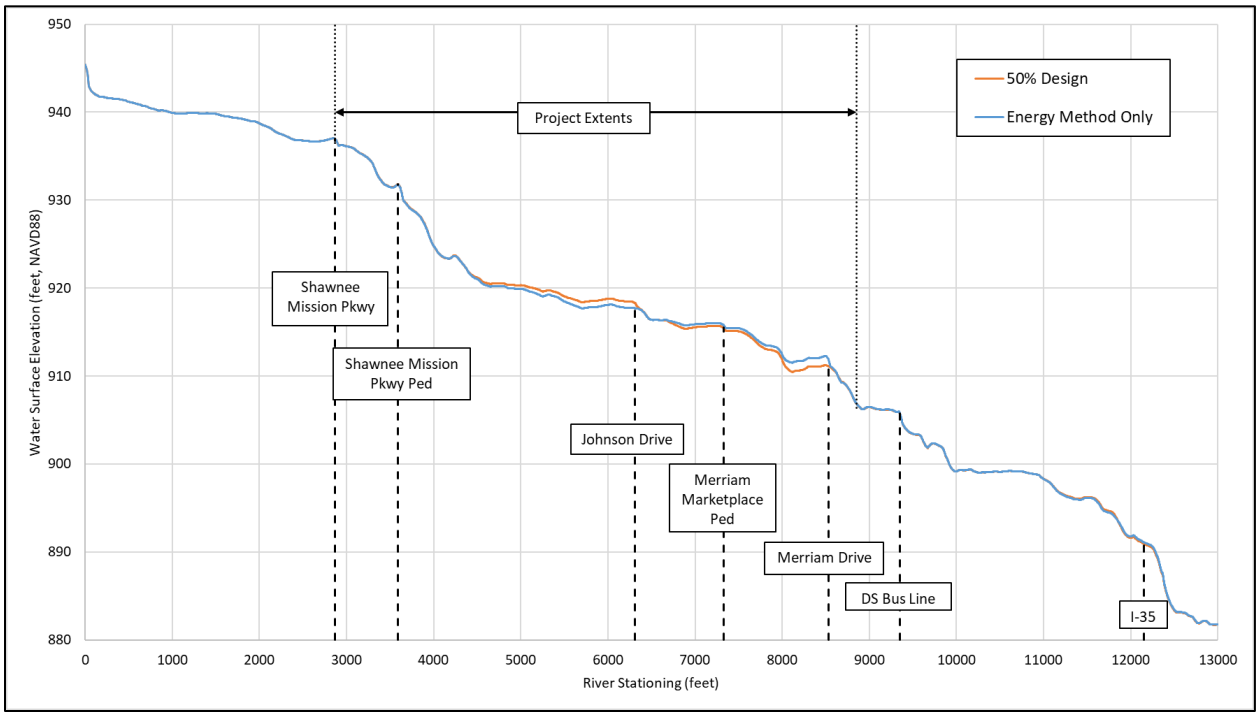


Table E.13. High Flow Bridge Modeling Approach Sensitivity Analysis

Location	Type	Change in WSE (ft)
Project Length	Max	1.07
	Average	0.00
Upstream Johnson Drive (STA 43+50-65+50)	Max	1.07
	Average	0.48
Upstream Merriam Drive (STA 65+50-86+00)	Max	-0.72
	Average	-0.44

Considerations should be given to variation in water surface profile with bridge modeling assumptions. But Pressure and/or Weir flow method is still deemed suitable for Johnson Drive and Merriam Drive bridges given that flow is fully constrained to flow through the bridge openings. Energy method is not applicable.

APPENDIX E.2 – FLOW INPUT DATA

HEC-1 Frequency Flow Hydrographs

E.2 FLOW INPUT DATA

Below are the flow frequency inflow hydrographs at each boundary location listed in the main report. These hydrographs were developed as part of the Upper Turkey Creek feasibility study using the HEC-1 hydrologic model. The boundary conditions are ordered from upstream to downstream.

E.2.1 TC-M8 XS 4.234 TCC23

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	2	0	0
01 Jan 2000, 00:05	1	1	1	1	2	2	2	2
01 Jan 2000, 00:10	2	3	3	4	4	5	5	6
01 Jan 2000, 00:15	3	4	4	6	6	7	8	9
01 Jan 2000, 00:20	5	6	7	9	10	11	12	14
01 Jan 2000, 00:25	7	8	9	12	14	17	18	19
01 Jan 2000, 00:30	9	11	13	17	19	22	24	26
01 Jan 2000, 00:35	11	14	16	22	25	29	32	33
01 Jan 2000, 00:40	13	17	21	27	32	37	41	42
01 Jan 2000, 00:45	16	21	26	34	40	46	50	51
01 Jan 2000, 00:50	19	25	31	41	48	56	61	62
01 Jan 2000, 00:55	23	30	37	48	57	66	72	73
01 Jan 2000, 01:00	26	34	43	55	65	76	83	85
01 Jan 2000, 01:05	30	39	48	63	74	86	94	98
01 Jan 2000, 01:10	34	44	54	70	82	96	105	110
01 Jan 2000, 01:15	37	49	60	77	91	105	116	123
01 Jan 2000, 01:20	41	54	65	83	98	114	126	136
01 Jan 2000, 01:25	44	58	70	89	105	122	135	149
01 Jan 2000, 01:30	47	62	75	94	112	129	144	162
01 Jan 2000, 01:35	50	66	80	99	118	136	151	173
01 Jan 2000, 01:40	53	70	85	104	123	142	159	184
01 Jan 2000, 01:45	55	74	89	108	128	148	166	194
01 Jan 2000, 01:50	58	77	93	111	133	154	172	203
01 Jan 2000, 01:55	60	80	96	115	138	159	178	210
01 Jan 2000, 02:00	62	83	99	118	141	163	184	216
01 Jan 2000, 02:05	64	85	102	121	145	167	188	220
01 Jan 2000, 02:10	66	87	104	124	147	170	191	223
01 Jan 2000, 02:15	67	89	106	127	150	172	194	226
01 Jan 2000, 02:20	69	90	108	129	152	174	196	228
01 Jan 2000, 02:25	70	91	109	131	153	175	197	229
01 Jan 2000, 02:30	71	92	110	133	154	176	198	229
01 Jan 2000, 02:35	72	93	110	134	155	177	199	230
01 Jan 2000, 02:40	72	93	111	135	155	177	200	230
01 Jan 2000, 02:45	73	94	111	135	155	178	200	231
01 Jan 2000, 02:50	73	94	111	136	156	178	200	231
01 Jan 2000, 02:55	74	94	112	136	156	178	200	231
01 Jan 2000, 03:00	74	94	112	137	156	178	201	231

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:05	74	94	112	137	156	178	201	231
01 Jan 2000, 03:10	74	94	112	137	156	178	201	231
01 Jan 2000, 03:15	74	94	112	137	156	178	201	231
01 Jan 2000, 03:20	74	94	112	137	156	178	201	231
01 Jan 2000, 03:25	74	94	112	137	156	178	201	231
01 Jan 2000, 03:30	74	94	112	137	156	178	201	231
01 Jan 2000, 03:35	74	94	112	137	156	178	201	231
01 Jan 2000, 03:40	74	94	112	137	156	178	201	231
01 Jan 2000, 03:45	74	94	112	137	156	178	201	231
01 Jan 2000, 03:50	75	95	113	138	157	179	202	232
01 Jan 2000, 03:55	75	96	113	139	158	181	204	234
01 Jan 2000, 04:00	76	96	114	140	159	182	205	236
01 Jan 2000, 04:05	76	97	115	141	161	184	207	238
01 Jan 2000, 04:10	77	98	116	143	163	186	209	241
01 Jan 2000, 04:15	78	99	118	145	165	189	212	244
01 Jan 2000, 04:20	79	100	119	147	168	192	215	248
01 Jan 2000, 04:25	80	102	121	149	170	195	219	252
01 Jan 2000, 04:30	81	103	123	152	174	199	223	257
01 Jan 2000, 04:35	82	105	125	154	177	203	227	262
01 Jan 2000, 04:40	83	107	127	157	180	206	232	268
01 Jan 2000, 04:45	85	108	129	160	183	210	236	273
01 Jan 2000, 04:50	86	110	131	162	186	213	240	279
01 Jan 2000, 04:55	87	112	133	165	189	216	244	284
01 Jan 2000, 05:00	89	113	135	167	191	219	247	289
01 Jan 2000, 05:05	90	115	137	168	193	222	251	295
01 Jan 2000, 05:10	91	116	138	170	196	224	254	300
01 Jan 2000, 05:15	92	118	140	172	198	227	257	305
01 Jan 2000, 05:20	93	119	142	173	199	229	260	310
01 Jan 2000, 05:25	93	120	143	175	201	231	263	314
01 Jan 2000, 05:30	94	121	144	176	203	233	266	318
01 Jan 2000, 05:35	95	122	145	177	204	235	268	322
01 Jan 2000, 05:40	96	123	146	178	205	237	271	326
01 Jan 2000, 05:45	96	124	147	179	206	238	273	330
01 Jan 2000, 05:50	97	124	148	180	207	239	275	334
01 Jan 2000, 05:55	97	125	148	181	207	240	277	338
01 Jan 2000, 06:00	98	125	148	181	208	241	279	342
01 Jan 2000, 06:05	98	126	149	182	209	243	282	347
01 Jan 2000, 06:10	99	127	150	184	211	246	286	354
01 Jan 2000, 06:15	100	127	151	185	213	249	290	360
01 Jan 2000, 06:20	101	128	152	187	215	252	295	367

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:25	102	129	153	189	218	256	301	375
01 Jan 2000, 06:30	102	130	155	191	221	260	307	384
01 Jan 2000, 06:35	103	132	157	193	224	265	314	394
01 Jan 2000, 06:40	105	133	159	196	228	271	322	404
01 Jan 2000, 06:45	106	135	161	199	232	277	331	415
01 Jan 2000, 06:50	107	137	163	202	236	284	340	426
01 Jan 2000, 06:55	108	138	165	205	241	291	348	438
01 Jan 2000, 07:00	110	140	167	209	245	297	357	449
01 Jan 2000, 07:05	111	142	169	212	250	304	366	460
01 Jan 2000, 07:10	112	144	171	215	254	310	374	471
01 Jan 2000, 07:15	114	145	173	217	258	317	382	481
01 Jan 2000, 07:20	115	147	175	220	263	323	390	491
01 Jan 2000, 07:25	116	148	177	222	267	329	397	500
01 Jan 2000, 07:30	117	150	179	225	271	335	404	509
01 Jan 2000, 07:35	118	151	180	227	276	341	411	517
01 Jan 2000, 07:40	118	152	182	230	280	347	418	524
01 Jan 2000, 07:45	119	153	183	232	284	352	424	530
01 Jan 2000, 07:50	120	154	185	235	288	357	429	536
01 Jan 2000, 07:55	121	155	186	237	291	361	434	542
01 Jan 2000, 08:00	121	156	187	240	295	365	439	547
01 Jan 2000, 08:05	122	157	188	243	299	371	445	555
01 Jan 2000, 08:10	123	158	190	247	304	377	452	563
01 Jan 2000, 08:15	124	159	192	250	309	382	459	570
01 Jan 2000, 08:20	125	160	194	255	315	390	468	581
01 Jan 2000, 08:25	127	162	197	261	323	399	478	594
01 Jan 2000, 08:30	128	164	200	267	331	409	490	608
01 Jan 2000, 08:35	130	167	204	274	340	421	503	625
01 Jan 2000, 08:40	132	170	208	282	350	433	518	644
01 Jan 2000, 08:45	134	173	213	291	361	447	535	665
01 Jan 2000, 08:50	136	176	219	300	373	462	552	688
01 Jan 2000, 08:55	139	180	225	310	386	477	571	711
01 Jan 2000, 09:00	142	184	231	320	398	492	589	734
01 Jan 2000, 09:05	145	189	238	331	412	509	609	760
01 Jan 2000, 09:10	148	194	246	342	426	526	629	785
01 Jan 2000, 09:15	151	199	253	353	439	542	649	810
01 Jan 2000, 09:20	154	204	261	364	452	559	669	834
01 Jan 2000, 09:25	157	209	269	375	466	575	689	859
01 Jan 2000, 09:30	161	215	277	386	480	593	710	884
01 Jan 2000, 09:35	164	221	287	399	496	612	732	911
01 Jan 2000, 09:40	168	227	297	412	512	631	755	938

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:45	172	234	307	425	528	650	778	965
01 Jan 2000, 09:50	176	241	316	439	544	670	800	991
01 Jan 2000, 09:55	180	247	326	453	560	689	822	1018
01 Jan 2000, 10:00	184	254	336	466	576	708	845	1045
01 Jan 2000, 10:05	189	262	347	481	594	729	870	1075
01 Jan 2000, 10:10	194	270	358	496	612	750	895	1106
01 Jan 2000, 10:15	198	278	369	510	629	772	921	1136
01 Jan 2000, 10:20	204	288	382	528	650	797	951	1172
01 Jan 2000, 10:25	210	298	396	547	673	824	984	1209
01 Jan 2000, 10:30	215	309	410	566	696	852	1016	1246
01 Jan 2000, 10:35	223	322	427	589	725	887	1058	1293
01 Jan 2000, 10:40	232	337	447	617	759	928	1107	1347
01 Jan 2000, 10:45	240	352	467	645	793	971	1158	1406
01 Jan 2000, 10:50	251	371	492	681	837	1024	1221	1480
01 Jan 2000, 10:55	264	392	521	722	887	1086	1289	1566
01 Jan 2000, 11:00	278	415	551	765	940	1152	1360	1657
01 Jan 2000, 11:05	294	441	587	816	1003	1227	1445	1764
01 Jan 2000, 11:10	313	471	627	873	1073	1308	1542	1886
01 Jan 2000, 11:15	333	503	670	934	1147	1392	1644	2014
01 Jan 2000, 11:20	357	541	723	1005	1232	1492	1766	2163
01 Jan 2000, 11:25	385	584	782	1086	1322	1603	1902	2321
01 Jan 2000, 11:30	413	629	842	1168	1414	1720	2043	2481
01 Jan 2000, 11:35	491	748	997	1374	1658	2017	2392	2887
01 Jan 2000, 11:40	595	912	1210	1653	1993	2425	2867	3453
01 Jan 2000, 11:45	702	1082	1427	1944	2352	2860	3376	4062
01 Jan 2000, 11:50	971	1469	1917	2622	3183	3859	4513	5366
01 Jan 2000, 11:55	1362	2042	2653	3630	4385	5247	6046	7110
01 Jan 2000, 12:00	1829	2696	3517	4776	5676	6638	7518	8609
01 Jan 2000, 12:05	2216	3285	4235	5569	6453	7355	8173	9164
01 Jan 2000, 12:10	2644	3847	4780	5994	6781	7638	8363	9222
01 Jan 2000, 12:15	3179	4386	5214	6349	7074	7892	8601	9470
01 Jan 2000, 12:20	3575	4678	5444	6470	7171	7960	8669	9672
01 Jan 2000, 12:25	3839	4833	5540	6502	7186	7972	8722	9968
01 Jan 2000, 12:30	3968	4924	5632	6571	7294	8158	9040	10525
01 Jan 2000, 12:35	3971	4986	5742	6704	7536	8556	9612	11262
01 Jan 2000, 12:40	3884	5032	5835	6906	7904	9074	10285	12012
01 Jan 2000, 12:45	3785	5009	5877	7156	8308	9584	10843	12559
01 Jan 2000, 12:50	3702	4895	5823	7387	8624	9920	11182	12846
01 Jan 2000, 12:55	3611	4745	5678	7505	8781	10047	11248	12835
01 Jan 2000, 13:00	3506	4600	5456	7455	8765	9976	11110	12628

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:05	3368	4424	5234	7179	8531	9724	10802	12242
01 Jan 2000, 13:10	3188	4200	5053	6726	8100	9347	10371	11756
01 Jan 2000, 13:15	2948	3955	4875	6169	7513	8863	9878	11215
01 Jan 2000, 13:20	2617	3712	4651	5673	6815	8244	9337	10631
01 Jan 2000, 13:25	2232	3485	4377	5304	6114	7557	8726	10031
01 Jan 2000, 13:30	1849	3253	4084	5014	5576	6835	8058	9441
01 Jan 2000, 13:35	1536	2966	3797	4742	5221	6115	7371	8828
01 Jan 2000, 13:40	1306	2597	3533	4457	4959	5548	6681	8184
01 Jan 2000, 13:45	1138	2180	3274	4162	4718	5199	5998	7557
01 Jan 2000, 13:50	1018	1786	2981	3873	4458	4967	5482	6936
01 Jan 2000, 13:55	931	1472	2611	3607	4177	4758	5173	6322
01 Jan 2000, 14:00	863	1255	2193	3350	3898	4531	4971	5753
01 Jan 2000, 14:05	808	1116	1798	3052	3636	4272	4794	5334
01 Jan 2000, 14:10	763	1019	1489	2677	3385	4002	4599	5102
01 Jan 2000, 14:15	724	952	1279	2257	3104	3744	4367	4937
01 Jan 2000, 14:20	688	898	1146	1867	2764	3499	4105	4784
01 Jan 2000, 14:25	656	853	1056	1572	2370	3233	3843	4610
01 Jan 2000, 14:30	628	815	994	1372	1987	2910	3608	4390
01 Jan 2000, 14:35	601	779	945	1246	1677	2517	3375	4138
01 Jan 2000, 14:40	576	747	903	1159	1463	2115	3104	3883
01 Jan 2000, 14:45	554	718	867	1096	1326	1787	2763	3646
01 Jan 2000, 14:50	533	692	834	1046	1239	1557	2373	3415
01 Jan 2000, 14:55	513	667	804	1004	1176	1415	2004	3153
01 Jan 2000, 15:00	495	644	776	968	1125	1326	1721	2829
01 Jan 2000, 15:05	478	622	751	936	1082	1265	1533	2464
01 Jan 2000, 15:10	462	602	727	907	1045	1219	1422	2111
01 Jan 2000, 15:15	448	585	706	882	1014	1181	1353	1827
01 Jan 2000, 15:20	434	568	686	857	986	1145	1305	1634
01 Jan 2000, 15:25	422	552	668	835	960	1112	1267	1520
01 Jan 2000, 15:30	411	538	652	816	938	1084	1236	1452
01 Jan 2000, 15:35	401	526	638	798	918	1060	1208	1408
01 Jan 2000, 15:40	391	514	624	782	899	1038	1182	1374
01 Jan 2000, 15:45	383	504	611	766	881	1017	1157	1345
01 Jan 2000, 15:50	375	494	599	751	864	998	1133	1319
01 Jan 2000, 15:55	368	484	587	737	848	980	1111	1295
01 Jan 2000, 16:00	361	476	577	724	834	963	1091	1273
01 Jan 2000, 16:05	354	468	567	712	821	948	1074	1253
01 Jan 2000, 16:10	348	460	559	702	809	935	1059	1236
01 Jan 2000, 16:15	343	454	551	693	799	924	1046	1220
01 Jan 2000, 16:20	338	448	545	686	791	914	1035	1207

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:25	334	443	539	679	783	905	1026	1195
01 Jan 2000, 16:30	331	439	535	674	777	898	1018	1184
01 Jan 2000, 16:35	327	435	529	667	769	889	1008	1172
01 Jan 2000, 16:40	323	430	523	660	761	880	998	1160
01 Jan 2000, 16:45	320	425	518	654	754	872	990	1149
01 Jan 2000, 16:50	317	421	513	647	747	864	980	1138
01 Jan 2000, 16:55	314	417	508	640	739	855	970	1126
01 Jan 2000, 17:00	310	413	503	633	730	844	958	1112
01 Jan 2000, 17:05	307	408	497	624	720	833	945	1096
01 Jan 2000, 17:10	303	403	490	616	710	821	931	1080
01 Jan 2000, 17:15	299	398	483	607	700	809	918	1064
01 Jan 2000, 17:20	296	393	477	599	690	798	905	1049
01 Jan 2000, 17:25	292	387	471	591	681	788	893	1035
01 Jan 2000, 17:30	288	383	465	584	673	778	882	1022
01 Jan 2000, 17:35	285	378	460	578	666	770	873	1011
01 Jan 2000, 17:40	282	375	455	572	660	763	864	1001
01 Jan 2000, 17:45	280	371	451	568	654	756	857	992
01 Jan 2000, 17:50	276	367	446	561	647	747	847	981
01 Jan 2000, 17:55	273	362	440	555	639	738	837	970
01 Jan 2000, 18:00	270	358	435	549	632	731	828	960
01 Jan 2000, 18:05	267	354	430	542	625	722	819	949
01 Jan 2000, 18:10	264	349	425	535	617	712	808	937
01 Jan 2000, 18:15	261	345	419	527	608	702	796	923
01 Jan 2000, 18:20	257	340	413	519	598	691	784	908
01 Jan 2000, 18:25	253	335	407	510	588	679	770	892
01 Jan 2000, 18:30	250	330	400	502	578	667	757	877
01 Jan 2000, 18:35	246	325	394	494	568	656	744	862
01 Jan 2000, 18:40	242	320	387	486	559	646	732	848
01 Jan 2000, 18:45	239	315	382	479	551	637	722	835
01 Jan 2000, 18:50	235	311	377	473	544	628	712	824
01 Jan 2000, 18:55	232	307	372	467	537	621	704	813
01 Jan 2000, 19:00	230	303	368	462	532	614	696	804
01 Jan 2000, 19:05	227	300	364	458	526	608	689	796
01 Jan 2000, 19:10	225	297	361	454	522	603	684	790
01 Jan 2000, 19:15	223	295	358	451	518	598	679	785
01 Jan 2000, 19:20	221	292	355	448	515	595	675	781
01 Jan 2000, 19:25	220	291	353	446	512	592	672	778
01 Jan 2000, 19:30	219	289	351	443	510	590	670	776
01 Jan 2000, 19:35	217	288	350	442	509	588	668	775
01 Jan 2000, 19:40	216	287	349	440	508	587	667	774

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:45	215	286	348	439	507	586	666	773
01 Jan 2000, 19:50	214	284	346	437	504	583	662	769
01 Jan 2000, 19:55	212	282	344	433	500	579	657	764
01 Jan 2000, 20:00	211	280	342	430	497	575	653	758
01 Jan 2000, 20:05	209	278	339	426	492	570	647	751
01 Jan 2000, 20:10	207	276	336	421	487	563	639	742
01 Jan 2000, 20:15	205	273	331	415	479	554	630	730
01 Jan 2000, 20:20	203	269	327	409	471	544	619	717
01 Jan 2000, 20:25	200	265	321	401	463	534	607	703
01 Jan 2000, 20:30	197	261	315	394	453	523	594	688
01 Jan 2000, 20:35	194	256	309	386	444	513	582	674
01 Jan 2000, 20:40	190	251	304	379	435	503	570	661
01 Jan 2000, 20:45	187	247	298	372	428	494	560	648
01 Jan 2000, 20:50	184	243	293	366	421	485	551	636
01 Jan 2000, 20:55	181	239	289	361	414	478	543	626
01 Jan 2000, 21:00	179	235	284	357	409	472	535	616
01 Jan 2000, 21:05	176	232	281	352	404	466	528	608
01 Jan 2000, 21:10	174	229	277	349	399	460	522	602
01 Jan 2000, 21:15	172	227	274	345	395	456	517	596
01 Jan 2000, 21:20	171	224	271	342	391	452	513	592
01 Jan 2000, 21:25	169	222	269	339	389	449	509	588
01 Jan 2000, 21:30	168	221	267	337	386	446	506	586
01 Jan 2000, 21:35	166	219	266	335	385	444	504	584
01 Jan 2000, 21:40	165	218	265	334	383	443	503	583
01 Jan 2000, 21:45	164	217	264	332	382	442	502	582
01 Jan 2000, 21:50	164	216	263	331	382	441	501	581
01 Jan 2000, 21:55	163	216	263	331	381	441	500	581
01 Jan 2000, 22:00	162	215	262	330	381	440	500	580
01 Jan 2000, 22:05	162	215	262	330	381	440	500	580
01 Jan 2000, 22:10	162	215	262	330	380	440	499	580
01 Jan 2000, 22:15	161	215	262	329	380	440	499	580
01 Jan 2000, 22:20	161	215	262	329	380	440	499	580
01 Jan 2000, 22:25	161	215	262	329	380	440	499	580
01 Jan 2000, 22:30	161	215	262	329	380	440	499	580
01 Jan 2000, 22:35	161	215	262	329	380	440	499	580
01 Jan 2000, 22:40	161	215	262	329	380	440	499	580
01 Jan 2000, 22:45	161	215	262	329	380	440	499	580
01 Jan 2000, 22:50	161	215	262	329	380	440	499	580
01 Jan 2000, 22:55	161	215	262	329	380	440	499	580
01 Jan 2000, 23:00	161	215	262	329	380	440	499	580

TC-M80 XS 4.234 TCC23								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:05	161	215	262	329	380	440	499	580
01 Jan 2000, 23:10	161	215	262	329	380	440	499	580
01 Jan 2000, 23:15	161	215	262	329	380	440	499	580
01 Jan 2000, 23:20	161	215	262	329	380	440	499	580
01 Jan 2000, 23:25	161	215	262	329	380	440	499	580
01 Jan 2000, 23:30	161	215	262	329	381	440	500	580
01 Jan 2000, 23:35	161	215	262	330	381	440	500	580
01 Jan 2000, 23:40	161	215	262	330	381	440	500	580
01 Jan 2000, 23:45	161	215	262	330	381	440	500	580
01 Jan 2000, 23:50	160	214	261	328	378	438	497	577
01 Jan 2000, 23:55	159	212	258	325	375	434	492	572
01 Jan 2000, 24:00	158	211	256	322	372	430	488	567

E.2.2 TC94

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	1	1	1	1	1
01 Jan 2000, 00:10	1	1	1	2	2	2	2	3
01 Jan 2000, 00:15	1	1	2	2	2	3	3	3
01 Jan 2000, 00:20	1	2	2	2	3	3	4	4
01 Jan 2000, 00:25	2	2	2	3	3	4	4	5
01 Jan 2000, 00:30	2	2	3	3	4	4	5	5
01 Jan 2000, 00:35	2	2	3	3	4	4	5	6
01 Jan 2000, 00:40	2	2	3	3	4	4	5	6
01 Jan 2000, 00:45	2	2	3	3	4	4	5	6
01 Jan 2000, 00:50	2	2	3	3	4	4	5	6
01 Jan 2000, 00:55	2	2	3	3	4	4	5	6
01 Jan 2000, 01:00	2	2	3	3	4	4	5	6
01 Jan 2000, 01:05	2	2	3	3	4	4	5	6
01 Jan 2000, 01:10	2	2	3	3	4	4	5	6
01 Jan 2000, 01:15	2	2	3	3	4	4	5	6
01 Jan 2000, 01:20	2	2	3	3	4	4	5	6
01 Jan 2000, 01:25	2	2	3	3	4	4	5	6
01 Jan 2000, 01:30	2	2	3	3	4	4	5	6
01 Jan 2000, 01:35	2	2	3	3	4	4	5	6
01 Jan 2000, 01:40	2	2	3	3	4	4	5	6
01 Jan 2000, 01:45	2	2	3	3	4	4	5	6
01 Jan 2000, 01:50	2	2	3	3	4	4	5	6
01 Jan 2000, 01:55	2	2	3	3	4	4	5	6
01 Jan 2000, 02:00	2	2	3	3	4	4	5	6
01 Jan 2000, 02:05	2	2	3	3	4	4	5	6
01 Jan 2000, 02:10	2	2	3	3	4	4	5	6
01 Jan 2000, 02:15	2	2	3	3	4	4	5	6
01 Jan 2000, 02:20	2	2	3	3	4	4	5	6
01 Jan 2000, 02:25	2	2	3	3	4	4	5	6
01 Jan 2000, 02:30	2	2	3	3	4	4	5	6
01 Jan 2000, 02:35	2	2	3	3	4	4	5	6
01 Jan 2000, 02:40	2	2	3	3	4	4	5	6
01 Jan 2000, 02:45	2	2	3	3	4	4	5	6
01 Jan 2000, 02:50	2	2	3	3	4	4	5	6
01 Jan 2000, 02:55	2	2	3	3	4	4	5	6
01 Jan 2000, 03:00	2	2	3	3	4	4	5	6

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:05	2	2	3	3	4	4	5	6
01 Jan 2000, 03:10	2	2	3	3	4	4	5	6
01 Jan 2000, 03:15	2	2	3	3	4	4	5	6
01 Jan 2000, 03:20	2	2	3	3	4	4	5	6
01 Jan 2000, 03:25	2	2	3	3	4	4	5	6
01 Jan 2000, 03:30	2	2	3	3	4	4	5	6
01 Jan 2000, 03:35	2	2	3	3	4	4	5	6
01 Jan 2000, 03:40	2	2	3	3	4	4	5	6
01 Jan 2000, 03:45	2	2	3	3	4	4	5	6
01 Jan 2000, 03:50	2	3	3	4	4	5	5	6
01 Jan 2000, 03:55	2	3	3	4	5	5	6	7
01 Jan 2000, 04:00	2	3	4	4	5	6	6	7
01 Jan 2000, 04:05	2	3	4	4	5	6	6	7
01 Jan 2000, 04:10	2	3	4	4	5	6	6	7
01 Jan 2000, 04:15	2	3	4	4	5	6	6	7
01 Jan 2000, 04:20	2	3	4	4	5	6	6	7
01 Jan 2000, 04:25	2	3	4	4	5	6	6	8
01 Jan 2000, 04:30	2	3	4	4	5	6	6	8
01 Jan 2000, 04:35	2	3	4	4	5	6	6	8
01 Jan 2000, 04:40	2	3	4	4	5	6	6	8
01 Jan 2000, 04:45	2	3	4	4	5	6	6	8
01 Jan 2000, 04:50	2	3	4	4	5	6	6	8
01 Jan 2000, 04:55	2	3	4	4	5	6	7	8
01 Jan 2000, 05:00	2	3	4	4	5	6	7	9
01 Jan 2000, 05:05	2	3	4	4	5	6	7	9
01 Jan 2000, 05:10	2	3	4	4	5	6	7	9
01 Jan 2000, 05:15	2	3	4	4	5	6	7	9
01 Jan 2000, 05:20	2	3	4	4	5	6	7	9
01 Jan 2000, 05:25	2	3	4	4	5	6	7	9
01 Jan 2000, 05:30	2	3	4	4	5	6	7	9
01 Jan 2000, 05:35	2	3	4	4	5	6	7	10
01 Jan 2000, 05:40	2	3	4	4	5	6	7	10
01 Jan 2000, 05:45	2	3	4	4	5	6	8	10
01 Jan 2000, 05:50	2	3	4	4	5	6	8	10
01 Jan 2000, 05:55	2	3	4	4	5	6	8	10
01 Jan 2000, 06:00	2	3	4	4	5	6	8	10
01 Jan 2000, 06:05	3	3	4	5	5	7	9	11
01 Jan 2000, 06:10	3	4	4	5	6	8	10	12
01 Jan 2000, 06:15	3	4	4	5	6	8	10	13
01 Jan 2000, 06:20	3	4	4	5	7	8	10	13

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:25	3	4	5	6	7	9	11	14
01 Jan 2000, 06:30	3	4	5	6	7	9	11	14
01 Jan 2000, 06:35	3	4	5	6	7	9	11	14
01 Jan 2000, 06:40	3	4	5	6	7	9	11	14
01 Jan 2000, 06:45	3	4	5	6	7	9	11	14
01 Jan 2000, 06:50	3	4	5	6	7	9	11	15
01 Jan 2000, 06:55	3	4	5	6	7	9	12	15
01 Jan 2000, 07:00	3	4	5	6	8	10	12	15
01 Jan 2000, 07:05	3	4	5	6	8	10	12	15
01 Jan 2000, 07:10	3	4	5	6	8	10	12	15
01 Jan 2000, 07:15	3	4	5	6	8	10	12	15
01 Jan 2000, 07:20	3	4	5	6	8	10	12	15
01 Jan 2000, 07:25	3	4	5	6	8	10	12	16
01 Jan 2000, 07:30	3	4	5	6	8	10	13	16
01 Jan 2000, 07:35	3	4	5	7	8	10	13	16
01 Jan 2000, 07:40	3	4	5	7	8	10	13	16
01 Jan 2000, 07:45	3	4	5	7	8	11	13	16
01 Jan 2000, 07:50	3	4	5	7	9	11	13	16
01 Jan 2000, 07:55	3	4	5	7	9	11	13	17
01 Jan 2000, 08:00	3	4	5	7	9	11	13	17
01 Jan 2000, 08:05	3	4	5	7	9	12	14	18
01 Jan 2000, 08:10	3	4	6	8	10	13	15	19
01 Jan 2000, 08:15	4	4	6	8	11	13	16	20
01 Jan 2000, 08:20	4	5	6	9	11	14	17	21
01 Jan 2000, 08:25	4	5	7	10	12	15	19	23
01 Jan 2000, 08:30	4	5	7	10	13	16	19	24
01 Jan 2000, 08:35	4	5	7	11	13	16	20	25
01 Jan 2000, 08:40	4	5	7	11	13	17	20	25
01 Jan 2000, 08:45	4	6	8	11	14	17	20	25
01 Jan 2000, 08:50	4	6	8	11	14	17	21	26
01 Jan 2000, 08:55	4	6	8	11	14	17	21	26
01 Jan 2000, 09:00	4	6	8	11	14	17	21	26
01 Jan 2000, 09:05	4	6	8	12	15	18	22	27
01 Jan 2000, 09:10	5	7	9	13	16	19	23	29
01 Jan 2000, 09:15	5	7	9	13	16	20	24	30
01 Jan 2000, 09:20	5	7	9	13	17	21	25	30
01 Jan 2000, 09:25	5	7	10	14	17	21	25	31
01 Jan 2000, 09:30	5	7	10	14	17	21	25	31
01 Jan 2000, 09:35	5	8	10	14	18	22	26	32
01 Jan 2000, 09:40	5	8	11	15	19	23	28	34

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:45	5	8	11	16	20	24	29	35
01 Jan 2000, 09:50	6	9	11	16	20	24	29	36
01 Jan 2000, 09:55	6	9	12	16	20	25	30	36
01 Jan 2000, 10:00	6	9	12	17	20	25	30	37
01 Jan 2000, 10:05	6	9	12	17	21	26	31	38
01 Jan 2000, 10:10	6	10	13	18	22	27	33	40
01 Jan 2000, 10:15	7	10	13	19	23	28	34	41
01 Jan 2000, 10:20	7	11	15	20	25	30	36	44
01 Jan 2000, 10:25	8	12	16	22	27	33	39	48
01 Jan 2000, 10:30	8	13	17	23	28	35	41	50
01 Jan 2000, 10:35	9	14	18	25	31	38	45	55
01 Jan 2000, 10:40	10	15	20	28	35	42	50	61
01 Jan 2000, 10:45	11	16	22	30	36	44	53	64
01 Jan 2000, 10:50	12	18	24	32	40	48	57	69
01 Jan 2000, 10:55	13	20	26	36	43	53	63	76
01 Jan 2000, 11:00	14	21	27	37	46	55	65	79
01 Jan 2000, 11:05	15	22	29	40	49	59	70	85
01 Jan 2000, 11:10	16	24	32	44	53	64	76	92
01 Jan 2000, 11:15	17	26	33	46	55	67	79	95
01 Jan 2000, 11:20	19	28	37	50	61	73	86	104
01 Jan 2000, 11:25	21	31	41	56	67	81	95	115
01 Jan 2000, 11:30	22	33	43	59	71	86	100	121
01 Jan 2000, 11:35	44	64	83	113	135	163	190	229
01 Jan 2000, 11:40	77	113	146	196	236	282	330	395
01 Jan 2000, 11:45	96	140	180	241	287	343	400	477
01 Jan 2000, 11:50	164	235	300	396	470	558	646	768
01 Jan 2000, 11:55	265	375	475	622	735	868	1002	1184
01 Jan 2000, 12:00	327	457	574	745	876	1030	1184	1394
01 Jan 2000, 12:05	273	379	474	612	718	842	965	1134
01 Jan 2000, 12:10	153	213	265	342	400	468	536	629
01 Jan 2000, 12:15	99	136	169	218	254	297	340	399
01 Jan 2000, 12:20	71	98	121	155	181	211	242	283
01 Jan 2000, 12:25	53	73	90	115	135	157	179	210
01 Jan 2000, 12:30	46	62	77	98	115	134	153	178
01 Jan 2000, 12:35	41	56	69	88	102	119	135	158
01 Jan 2000, 12:40	37	50	62	79	91	106	121	142
01 Jan 2000, 12:45	35	48	59	75	87	102	116	135
01 Jan 2000, 12:50	32	44	55	70	81	94	107	125
01 Jan 2000, 12:55	29	40	49	62	73	84	96	112
01 Jan 2000, 13:00	28	38	47	59	69	80	91	107

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:05	26	35	44	56	65	75	86	100
01 Jan 2000, 13:10	24	33	40	51	59	69	79	92
01 Jan 2000, 13:15	23	31	39	49	57	66	76	88
01 Jan 2000, 13:20	22	30	37	47	54	63	72	84
01 Jan 2000, 13:25	21	28	34	44	51	59	67	78
01 Jan 2000, 13:30	20	27	33	43	49	57	65	76
01 Jan 2000, 13:35	19	26	32	40	47	54	62	72
01 Jan 2000, 13:40	18	24	29	37	43	50	57	67
01 Jan 2000, 13:45	17	23	28	36	42	49	56	65
01 Jan 2000, 13:50	16	22	27	35	40	47	53	62
01 Jan 2000, 13:55	16	21	26	33	39	45	51	60
01 Jan 2000, 14:00	15	21	26	33	38	44	50	58
01 Jan 2000, 14:05	15	20	25	31	37	42	48	56
01 Jan 2000, 14:10	14	19	24	30	35	41	46	54
01 Jan 2000, 14:15	14	19	23	29	34	40	45	53
01 Jan 2000, 14:20	13	18	22	28	33	38	43	51
01 Jan 2000, 14:25	13	17	21	27	31	36	41	48
01 Jan 2000, 14:30	12	17	21	26	30	35	40	47
01 Jan 2000, 14:35	12	16	20	25	29	34	38	45
01 Jan 2000, 14:40	11	15	19	24	27	32	36	42
01 Jan 2000, 14:45	11	15	18	23	27	31	35	41
01 Jan 2000, 14:50	11	15	18	23	26	31	35	41
01 Jan 2000, 14:55	11	15	18	23	26	31	35	41
01 Jan 2000, 15:00	11	15	18	23	26	31	35	40
01 Jan 2000, 15:05	11	15	18	23	26	31	35	40
01 Jan 2000, 15:10	11	15	18	23	26	31	35	40
01 Jan 2000, 15:15	11	15	18	23	26	31	35	40
01 Jan 2000, 15:20	10	14	17	22	25	29	33	39
01 Jan 2000, 15:25	10	13	16	20	24	28	31	36
01 Jan 2000, 15:30	9	13	16	20	23	27	30	35
01 Jan 2000, 15:35	9	13	16	20	23	26	30	35
01 Jan 2000, 15:40	9	13	15	20	23	26	30	35
01 Jan 2000, 15:45	9	13	15	20	23	26	30	35
01 Jan 2000, 15:50	9	13	15	20	23	26	30	35
01 Jan 2000, 15:55	9	13	15	20	23	26	30	35
01 Jan 2000, 16:00	9	13	15	20	23	26	30	35
01 Jan 2000, 16:05	9	13	15	20	23	26	30	35
01 Jan 2000, 16:10	9	13	15	20	23	26	30	35
01 Jan 2000, 16:15	9	13	15	20	23	26	30	35
01 Jan 2000, 16:20	9	13	15	20	23	26	30	35

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:25	9	13	15	20	23	26	30	35
01 Jan 2000, 16:30	9	13	15	20	23	26	30	35
01 Jan 2000, 16:35	9	12	15	19	22	25	29	33
01 Jan 2000, 16:40	8	11	14	17	20	23	26	31
01 Jan 2000, 16:45	8	11	13	17	19	22	25	30
01 Jan 2000, 16:50	8	11	13	16	19	22	25	29
01 Jan 2000, 16:55	8	11	13	16	19	22	25	29
01 Jan 2000, 17:00	8	11	13	16	19	22	25	29
01 Jan 2000, 17:05	8	11	13	16	19	22	25	29
01 Jan 2000, 17:10	8	11	13	16	19	22	25	29
01 Jan 2000, 17:15	8	11	13	16	19	22	25	29
01 Jan 2000, 17:20	8	11	13	16	19	22	25	29
01 Jan 2000, 17:25	8	11	13	16	19	22	25	29
01 Jan 2000, 17:30	8	11	13	16	19	22	25	29
01 Jan 2000, 17:35	8	11	13	16	19	22	25	29
01 Jan 2000, 17:40	8	11	13	16	19	22	25	29
01 Jan 2000, 17:45	8	11	13	16	19	22	25	29
01 Jan 2000, 17:50	7	10	12	15	18	21	24	27
01 Jan 2000, 17:55	7	9	11	14	16	19	22	25
01 Jan 2000, 18:00	6	9	11	13	16	18	21	24
01 Jan 2000, 18:05	6	9	10	13	15	18	20	23
01 Jan 2000, 18:10	6	9	10	13	15	18	20	23
01 Jan 2000, 18:15	6	8	10	13	15	18	20	23
01 Jan 2000, 18:20	6	8	10	13	15	18	20	23
01 Jan 2000, 18:25	6	8	10	13	15	18	20	23
01 Jan 2000, 18:30	6	8	10	13	15	18	20	23
01 Jan 2000, 18:35	6	8	10	13	15	18	20	23
01 Jan 2000, 18:40	6	8	10	13	15	18	20	23
01 Jan 2000, 18:45	6	8	10	13	15	18	20	23
01 Jan 2000, 18:50	6	9	10	13	15	18	20	23
01 Jan 2000, 18:55	6	9	10	13	15	18	20	23
01 Jan 2000, 19:00	6	9	10	13	15	18	20	23
01 Jan 2000, 19:05	6	9	10	13	15	18	20	23
01 Jan 2000, 19:10	6	9	10	13	15	18	20	23
01 Jan 2000, 19:15	6	9	10	13	15	18	20	23
01 Jan 2000, 19:20	6	9	10	13	15	18	20	23
01 Jan 2000, 19:25	6	9	10	13	15	18	20	23
01 Jan 2000, 19:30	6	9	10	13	15	18	20	23
01 Jan 2000, 19:35	6	9	10	13	15	18	20	23
01 Jan 2000, 19:40	6	9	10	13	15	18	20	23

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:45	6	9	10	13	15	18	20	23
01 Jan 2000, 19:50	6	8	10	12	14	16	19	22
01 Jan 2000, 19:55	5	7	9	11	13	15	17	19
01 Jan 2000, 20:00	5	7	8	10	12	14	16	18
01 Jan 2000, 20:05	5	7	8	10	12	13	15	18
01 Jan 2000, 20:10	5	6	8	10	11	13	15	18
01 Jan 2000, 20:15	5	6	8	10	11	13	15	17
01 Jan 2000, 20:20	5	6	8	10	11	13	15	17
01 Jan 2000, 20:25	5	6	8	10	11	13	15	17
01 Jan 2000, 20:30	5	6	8	10	11	13	15	17
01 Jan 2000, 20:35	5	6	8	10	11	13	15	17
01 Jan 2000, 20:40	5	6	8	10	11	13	15	17
01 Jan 2000, 20:45	5	6	8	10	11	13	15	17
01 Jan 2000, 20:50	5	6	8	10	11	13	15	17
01 Jan 2000, 20:55	5	6	8	10	11	13	15	17
01 Jan 2000, 21:00	5	6	8	10	11	13	15	17
01 Jan 2000, 21:05	5	6	8	10	11	13	15	17
01 Jan 2000, 21:10	5	6	8	10	11	13	15	17
01 Jan 2000, 21:15	5	6	8	10	11	13	15	17
01 Jan 2000, 21:20	5	6	8	10	11	13	15	17
01 Jan 2000, 21:25	5	6	8	10	11	13	15	17
01 Jan 2000, 21:30	5	6	8	10	11	13	15	17
01 Jan 2000, 21:35	5	6	8	10	11	13	15	17
01 Jan 2000, 21:40	5	6	8	10	11	13	15	17
01 Jan 2000, 21:45	5	6	8	10	11	13	15	17
01 Jan 2000, 21:50	5	6	8	10	11	13	15	17
01 Jan 2000, 21:55	5	6	8	10	11	13	15	17
01 Jan 2000, 22:00	5	6	8	10	11	13	15	17
01 Jan 2000, 22:05	5	6	8	10	11	13	15	17
01 Jan 2000, 22:10	5	6	8	10	11	13	15	17
01 Jan 2000, 22:15	5	6	8	10	11	13	15	17
01 Jan 2000, 22:20	5	6	8	10	11	13	15	17
01 Jan 2000, 22:25	5	6	8	10	11	13	15	17
01 Jan 2000, 22:30	5	6	8	10	11	13	15	17
01 Jan 2000, 22:35	5	6	8	10	11	13	15	17
01 Jan 2000, 22:40	5	6	8	10	11	13	15	17
01 Jan 2000, 22:45	5	6	8	10	11	13	15	17
01 Jan 2000, 22:50	5	6	8	10	11	13	15	17
01 Jan 2000, 22:55	5	6	8	10	11	13	15	17
01 Jan 2000, 23:00	5	6	8	10	11	13	15	17

TC94								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:05	5	6	8	10	11	13	15	17
01 Jan 2000, 23:10	5	6	8	10	11	13	15	17
01 Jan 2000, 23:15	5	6	8	10	11	13	15	17
01 Jan 2000, 23:20	5	6	8	10	11	13	15	17
01 Jan 2000, 23:25	5	6	8	10	11	13	15	17
01 Jan 2000, 23:30	5	6	8	10	11	13	15	17
01 Jan 2000, 23:35	5	6	8	10	11	13	15	17
01 Jan 2000, 23:40	5	6	8	10	11	13	15	17
01 Jan 2000, 23:45	5	6	8	10	11	12	15	17
01 Jan 2000, 23:50	4	6	7	9	10	10	14	16
01 Jan 2000, 23:55	4	5	6	8	9	9	12	13
01 Jan 2000, 24:00	3	5	6	7	8	7	11	12

E.2.3 TCT71

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	1	1	1	0	0
01 Jan 2000, 00:15	1	1	1	2	2	2	2	2
01 Jan 2000, 00:20	1	2	2	3	3	3	3	4
01 Jan 2000, 00:25	2	2	3	3	4	4	5	5
01 Jan 2000, 00:30	2	3	3	4	5	6	6	7
01 Jan 2000, 00:35	3	3	4	5	6	7	8	9
01 Jan 2000, 00:40	3	4	5	6	7	8	9	10
01 Jan 2000, 00:45	4	4	5	7	8	9	10	12
01 Jan 2000, 00:50	4	4	6	7	9	10	12	13
01 Jan 2000, 00:55	5	5	7	8	10	11	12	14
01 Jan 2000, 01:00	5	6	7	9	10	12	13	15
01 Jan 2000, 01:05	5	6	7	9	11	12	14	16
01 Jan 2000, 01:10	5	6	8	10	11	13	14	16
01 Jan 2000, 01:15	5	7	8	10	11	13	15	17
01 Jan 2000, 01:20	5	7	8	10	12	13	15	17
01 Jan 2000, 01:25	6	7	8	10	12	13	15	17
01 Jan 2000, 01:30	6	7	8	10	12	14	15	17
01 Jan 2000, 01:35	6	7	9	10	12	14	15	18
01 Jan 2000, 01:40	6	7	9	10	12	14	15	18
01 Jan 2000, 01:45	6	7	9	11	12	14	15	18
01 Jan 2000, 01:50	6	7	9	11	12	14	15	18
01 Jan 2000, 01:55	6	7	9	11	12	14	15	18
01 Jan 2000, 02:00	6	7	9	11	12	14	15	18
01 Jan 2000, 02:05	6	7	9	11	12	14	15	18
01 Jan 2000, 02:10	6	7	9	11	12	14	15	18
01 Jan 2000, 02:15	6	7	9	11	12	14	15	18
01 Jan 2000, 02:20	6	7	9	11	12	14	15	18
01 Jan 2000, 02:25	6	7	9	11	12	14	15	18
01 Jan 2000, 02:30	6	7	9	11	12	14	15	18
01 Jan 2000, 02:35	6	7	9	11	12	14	15	18
01 Jan 2000, 02:40	6	7	9	11	12	14	15	18
01 Jan 2000, 02:45	6	7	9	11	12	14	15	18
01 Jan 2000, 02:50	6	7	9	11	12	14	15	18
01 Jan 2000, 02:55	6	7	9	11	12	14	15	18
01 Jan 2000, 03:00	6	7	9	11	12	14	15	18

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:05	6	7	9	11	12	14	15	18
01 Jan 2000, 03:10	6	7	9	11	12	14	15	18
01 Jan 2000, 03:15	6	7	9	11	12	14	15	18
01 Jan 2000, 03:20	6	7	9	11	12	14	15	18
01 Jan 2000, 03:25	6	7	9	11	12	14	15	18
01 Jan 2000, 03:30	6	7	9	11	12	14	15	18
01 Jan 2000, 03:35	6	7	9	11	12	14	15	18
01 Jan 2000, 03:40	6	7	9	11	12	14	15	18
01 Jan 2000, 03:45	6	7	9	11	12	14	15	18
01 Jan 2000, 03:50	6	7	9	11	12	14	15	18
01 Jan 2000, 03:55	6	7	9	11	12	14	16	18
01 Jan 2000, 04:00	6	8	9	11	13	15	16	19
01 Jan 2000, 04:05	6	8	10	12	14	15	17	20
01 Jan 2000, 04:10	7	8	10	12	14	16	18	20
01 Jan 2000, 04:15	7	9	10	12	14	16	18	21
01 Jan 2000, 04:20	7	9	10	13	15	17	19	21
01 Jan 2000, 04:25	7	9	11	13	15	17	19	22
01 Jan 2000, 04:30	7	9	11	13	15	17	19	22
01 Jan 2000, 04:35	7	9	11	13	15	18	20	23
01 Jan 2000, 04:40	7	9	11	14	16	18	20	23
01 Jan 2000, 04:45	7	9	11	14	16	18	20	23
01 Jan 2000, 04:50	8	10	11	14	16	18	20	23
01 Jan 2000, 04:55	8	10	11	14	16	18	20	24
01 Jan 2000, 05:00	8	10	11	14	16	18	20	24
01 Jan 2000, 05:05	8	10	11	14	16	18	21	24
01 Jan 2000, 05:10	8	10	11	14	16	18	21	24
01 Jan 2000, 05:15	8	10	11	14	16	18	21	24
01 Jan 2000, 05:20	8	10	11	14	16	18	21	25
01 Jan 2000, 05:25	8	10	11	14	16	18	21	25
01 Jan 2000, 05:30	8	10	11	14	16	18	21	25
01 Jan 2000, 05:35	8	10	11	14	16	18	21	25
01 Jan 2000, 05:40	8	10	11	14	16	18	21	25
01 Jan 2000, 05:45	8	10	12	14	16	18	21	26
01 Jan 2000, 05:50	8	10	12	14	16	18	21	26
01 Jan 2000, 05:55	8	10	12	14	16	18	21	27
01 Jan 2000, 06:00	8	10	12	14	16	18	21	27
01 Jan 2000, 06:05	8	10	12	14	16	19	22	28
01 Jan 2000, 06:10	8	10	12	14	16	19	22	29
01 Jan 2000, 06:15	8	10	12	15	17	20	23	30
01 Jan 2000, 06:20	8	11	13	15	18	21	24	32

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:25	9	11	13	16	18	21	25	33
01 Jan 2000, 06:30	9	11	13	16	18	22	26	35
01 Jan 2000, 06:35	9	11	13	16	19	22	27	36
01 Jan 2000, 06:40	9	11	14	17	19	23	28	37
01 Jan 2000, 06:45	9	12	14	17	19	23	29	38
01 Jan 2000, 06:50	9	12	14	17	20	24	30	40
01 Jan 2000, 06:55	9	12	14	17	20	24	31	41
01 Jan 2000, 07:00	9	12	14	17	20	25	32	42
01 Jan 2000, 07:05	9	12	14	17	21	26	32	43
01 Jan 2000, 07:10	9	12	14	18	21	26	33	43
01 Jan 2000, 07:15	9	12	14	18	21	27	34	44
01 Jan 2000, 07:20	10	12	14	18	21	27	34	45
01 Jan 2000, 07:25	10	12	14	18	21	28	35	46
01 Jan 2000, 07:30	10	12	14	18	22	28	36	47
01 Jan 2000, 07:35	10	12	14	18	22	29	36	47
01 Jan 2000, 07:40	10	12	14	18	22	29	37	48
01 Jan 2000, 07:45	10	12	14	18	23	30	37	49
01 Jan 2000, 07:50	10	12	14	18	23	30	38	49
01 Jan 2000, 07:55	10	12	14	18	23	31	38	50
01 Jan 2000, 08:00	10	12	14	19	24	31	39	51
01 Jan 2000, 08:05	10	12	14	19	24	32	40	51
01 Jan 2000, 08:10	10	12	15	19	25	32	41	53
01 Jan 2000, 08:15	10	13	15	20	26	34	42	55
01 Jan 2000, 08:20	10	13	16	21	27	35	44	57
01 Jan 2000, 08:25	11	13	16	22	29	37	46	59
01 Jan 2000, 08:30	11	14	17	23	30	39	48	62
01 Jan 2000, 08:35	11	15	18	25	32	41	51	65
01 Jan 2000, 08:40	12	15	18	26	33	43	53	68
01 Jan 2000, 08:45	12	15	19	27	35	44	55	71
01 Jan 2000, 08:50	12	16	19	28	36	46	57	73
01 Jan 2000, 08:55	13	16	20	29	37	48	59	76
01 Jan 2000, 09:00	13	16	20	30	38	49	61	78
01 Jan 2000, 09:05	13	17	21	31	40	51	63	80
01 Jan 2000, 09:10	13	17	21	32	41	52	65	83
01 Jan 2000, 09:15	14	17	22	33	43	55	68	86
01 Jan 2000, 09:20	14	18	23	35	44	57	70	89
01 Jan 2000, 09:25	14	18	24	36	46	58	72	91
01 Jan 2000, 09:30	14	19	25	37	47	60	74	94
01 Jan 2000, 09:35	15	19	25	38	49	62	76	96
01 Jan 2000, 09:40	15	20	26	39	50	64	78	99

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:45	15	20	28	41	52	67	82	103
01 Jan 2000, 09:50	16	21	29	43	54	69	85	107
01 Jan 2000, 09:55	16	22	30	44	56	71	87	110
01 Jan 2000, 10:00	16	22	31	46	58	73	90	113
01 Jan 2000, 10:05	16	23	32	47	59	75	92	116
01 Jan 2000, 10:10	17	24	33	49	61	78	95	120
01 Jan 2000, 10:15	17	25	34	51	64	81	98	124
01 Jan 2000, 10:20	18	26	36	52	66	84	102	128
01 Jan 2000, 10:25	18	27	38	55	69	88	107	134
01 Jan 2000, 10:30	19	29	40	58	73	93	113	141
01 Jan 2000, 10:35	20	30	42	61	77	97	118	148
01 Jan 2000, 10:40	21	32	45	66	83	104	126	158
01 Jan 2000, 10:45	23	35	49	71	89	112	136	170
01 Jan 2000, 10:50	24	38	53	76	95	120	145	181
01 Jan 2000, 10:55	26	41	57	82	103	129	157	195
01 Jan 2000, 11:00	28	45	62	89	112	140	170	211
01 Jan 2000, 11:05	31	48	67	96	121	151	182	226
01 Jan 2000, 11:10	33	52	72	104	130	163	196	243
01 Jan 2000, 11:15	36	57	79	113	141	176	212	262
01 Jan 2000, 11:20	39	62	85	122	152	189	227	281
01 Jan 2000, 11:25	42	67	92	133	165	205	246	303
01 Jan 2000, 11:30	47	74	102	145	180	223	267	331
01 Jan 2000, 11:35	53	83	117	169	209	261	315	393
01 Jan 2000, 11:40	76	126	172	243	301	376	452	537
01 Jan 2000, 11:45	122	192	254	354	432	519	607	712
01 Jan 2000, 11:50	174	266	359	493	586	677	788	929
01 Jan 2000, 11:55	286	433	558	739	838	952	1089	1266
01 Jan 2000, 12:00	438	628	834	972	1109	1268	1472	1693
01 Jan 2000, 12:05	540	726	1019	1127	1377	1617	1845	2110
01 Jan 2000, 12:10	558	667	920	1253	1529	1814	2082	2464
01 Jan 2000, 12:15	494	642	896	1262	1519	1843	2181	2646
01 Jan 2000, 12:20	470	726	946	1264	1551	1894	2239	2716
01 Jan 2000, 12:25	485	698	903	1273	1555	1898	2238	2702
01 Jan 2000, 12:30	457	648	866	1181	1416	1708	2041	2495
01 Jan 2000, 12:35	423	615	767	1006	1199	1442	1693	2111
01 Jan 2000, 12:40	390	542	666	845	995	1180	1377	1635
01 Jan 2000, 12:45	362	474	577	722	831	971	1118	1325
01 Jan 2000, 12:50	299	416	503	624	713	818	929	1090
01 Jan 2000, 12:55	193	369	437	541	618	703	792	920
01 Jan 2000, 13:00	191	312	388	472	539	619	693	797

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:05	160	216	360	420	479	549	617	709
01 Jan 2000, 13:10	147	185	300	384	429	491	553	635
01 Jan 2000, 13:15	132	193	201	361	392	442	497	571
01 Jan 2000, 13:20	121	158	211	304	370	406	453	520
01 Jan 2000, 13:25	111	150	190	211	354	382	417	477
01 Jan 2000, 13:30	102	144	167	232	307	361	392	440
01 Jan 2000, 13:35	95	125	164	203	217	325	372	412
01 Jan 2000, 13:40	89	126	148	189	230	256	356	390
01 Jan 2000, 13:45	83	112	142	182	206	236	313	371
01 Jan 2000, 13:50	79	109	132	168	198	233	243	353
01 Jan 2000, 13:55	75	102	126	161	186	215	253	299
01 Jan 2000, 14:00	71	97	120	152	177	206	231	217
01 Jan 2000, 14:05	68	93	114	145	169	196	226	249
01 Jan 2000, 14:10	65	88	109	139	161	187	213	266
01 Jan 2000, 14:15	62	85	104	133	154	179	204	231
01 Jan 2000, 14:20	59	81	100	128	148	173	197	230
01 Jan 2000, 14:25	57	78	96	123	143	166	189	222
01 Jan 2000, 14:30	55	75	93	118	137	160	182	211
01 Jan 2000, 14:35	53	72	89	114	133	154	175	206
01 Jan 2000, 14:40	51	70	86	110	128	148	169	197
01 Jan 2000, 14:45	49	67	83	106	123	142	162	189
01 Jan 2000, 14:50	47	65	80	102	118	138	157	183
01 Jan 2000, 14:55	46	63	77	99	115	133	152	177
01 Jan 2000, 15:00	45	61	75	96	111	130	147	172
01 Jan 2000, 15:05	43	59	73	93	109	126	144	168
01 Jan 2000, 15:10	42	58	72	91	106	124	141	164
01 Jan 2000, 15:15	42	57	70	90	104	121	138	161
01 Jan 2000, 15:20	41	56	69	88	103	119	136	159
01 Jan 2000, 15:25	40	55	68	86	101	117	133	156
01 Jan 2000, 15:30	39	54	66	84	98	114	130	152
01 Jan 2000, 15:35	38	52	65	83	96	112	128	149
01 Jan 2000, 15:40	38	52	64	81	95	110	126	146
01 Jan 2000, 15:45	37	51	63	80	93	108	123	144
01 Jan 2000, 15:50	36	50	62	79	92	107	121	142
01 Jan 2000, 15:55	36	49	61	78	90	105	120	139
01 Jan 2000, 16:00	36	49	60	77	89	104	118	138
01 Jan 2000, 16:05	35	48	59	76	88	102	117	136
01 Jan 2000, 16:10	35	48	59	75	87	101	116	135
01 Jan 2000, 16:15	34	47	58	74	86	101	115	134
01 Jan 2000, 16:20	34	47	58	74	86	100	114	134

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:25	34	47	58	74	86	100	114	133
01 Jan 2000, 16:30	34	47	58	74	86	100	114	133
01 Jan 2000, 16:35	34	46	57	73	85	99	113	132
01 Jan 2000, 16:40	34	46	57	73	85	98	112	131
01 Jan 2000, 16:45	33	45	56	71	83	96	110	128
01 Jan 2000, 16:50	32	44	55	70	81	95	108	126
01 Jan 2000, 16:55	32	44	54	69	80	93	106	124
01 Jan 2000, 17:00	32	43	53	68	79	92	105	122
01 Jan 2000, 17:05	31	42	52	67	78	90	103	120
01 Jan 2000, 17:10	31	42	52	66	76	89	101	118
01 Jan 2000, 17:15	30	41	51	65	75	87	100	116
01 Jan 2000, 17:20	30	41	50	64	74	86	98	114
01 Jan 2000, 17:25	29	40	50	63	73	85	97	113
01 Jan 2000, 17:30	29	40	49	63	73	85	96	112
01 Jan 2000, 17:35	29	40	49	62	72	84	96	112
01 Jan 2000, 17:40	29	39	49	62	72	84	95	111
01 Jan 2000, 17:45	29	39	48	62	72	84	95	111
01 Jan 2000, 17:50	29	39	48	62	72	83	95	111
01 Jan 2000, 17:55	28	39	48	61	71	82	94	109
01 Jan 2000, 18:00	28	38	47	59	69	80	92	107
01 Jan 2000, 18:05	27	37	46	58	68	79	90	104
01 Jan 2000, 18:10	27	36	45	57	66	77	88	102
01 Jan 2000, 18:15	26	36	44	56	65	76	86	100
01 Jan 2000, 18:20	26	35	43	55	64	74	84	98
01 Jan 2000, 18:25	25	34	42	54	63	73	83	96
01 Jan 2000, 18:30	25	34	42	53	61	71	81	94
01 Jan 2000, 18:35	24	33	41	52	60	70	80	93
01 Jan 2000, 18:40	24	33	40	51	60	69	79	92
01 Jan 2000, 18:45	24	32	40	51	59	68	78	91
01 Jan 2000, 18:50	24	32	39	50	58	68	77	90
01 Jan 2000, 18:55	23	32	39	50	58	67	77	90
01 Jan 2000, 19:00	23	32	39	50	58	67	77	89
01 Jan 2000, 19:05	23	32	39	50	58	67	76	89
01 Jan 2000, 19:10	23	31	39	49	58	67	76	89
01 Jan 2000, 19:15	23	31	39	49	57	67	76	89
01 Jan 2000, 19:20	23	31	39	49	57	67	76	89
01 Jan 2000, 19:25	23	31	39	49	57	67	76	89
01 Jan 2000, 19:30	23	31	39	49	57	67	76	89
01 Jan 2000, 19:35	23	31	39	49	57	67	76	89
01 Jan 2000, 19:40	23	31	39	49	57	67	76	89

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:45	23	31	39	49	57	67	76	89
01 Jan 2000, 19:50	23	31	39	49	57	67	76	88
01 Jan 2000, 19:55	23	31	38	49	57	66	75	87
01 Jan 2000, 20:00	22	30	37	47	55	64	73	85
01 Jan 2000, 20:05	22	29	36	46	54	62	71	83
01 Jan 2000, 20:10	21	29	36	45	52	61	69	81
01 Jan 2000, 20:15	21	28	35	44	51	60	68	79
01 Jan 2000, 20:20	20	28	34	43	50	58	66	77
01 Jan 2000, 20:25	20	27	33	42	49	57	64	75
01 Jan 2000, 20:30	19	26	32	41	48	55	63	73
01 Jan 2000, 20:35	19	26	32	40	47	54	61	71
01 Jan 2000, 20:40	19	25	31	39	46	53	60	70
01 Jan 2000, 20:45	18	25	31	39	45	52	59	69
01 Jan 2000, 20:50	18	24	30	38	44	52	59	68
01 Jan 2000, 20:55	18	24	30	38	44	51	58	68
01 Jan 2000, 21:00	18	24	30	38	44	51	58	67
01 Jan 2000, 21:05	18	24	29	37	44	51	58	67
01 Jan 2000, 21:10	18	24	29	37	43	50	57	67
01 Jan 2000, 21:15	18	24	29	37	43	50	57	67
01 Jan 2000, 21:20	17	24	29	37	43	50	57	67
01 Jan 2000, 21:25	17	24	29	37	43	50	57	67
01 Jan 2000, 21:30	17	24	29	37	43	50	57	67
01 Jan 2000, 21:35	17	24	29	37	43	50	57	67
01 Jan 2000, 21:40	17	24	29	37	43	50	57	67
01 Jan 2000, 21:45	17	24	29	37	43	50	57	67
01 Jan 2000, 21:50	17	24	29	37	43	50	57	67
01 Jan 2000, 21:55	17	24	29	37	43	50	57	67
01 Jan 2000, 22:00	17	24	29	37	43	50	57	67
01 Jan 2000, 22:05	17	24	29	37	43	50	57	67
01 Jan 2000, 22:10	17	24	29	37	43	50	57	67
01 Jan 2000, 22:15	17	24	29	37	43	50	57	67
01 Jan 2000, 22:20	17	24	29	37	43	50	57	67
01 Jan 2000, 22:25	17	24	29	37	43	50	57	67
01 Jan 2000, 22:30	17	24	29	37	43	50	57	67
01 Jan 2000, 22:35	17	24	29	37	43	50	57	67
01 Jan 2000, 22:40	17	24	29	37	43	50	57	67
01 Jan 2000, 22:45	18	24	29	37	43	50	57	67
01 Jan 2000, 22:50	18	24	29	37	43	50	57	67
01 Jan 2000, 22:55	18	24	29	37	43	50	57	67
01 Jan 2000, 23:00	18	24	29	37	43	50	57	67

TCT71								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:05	18	24	29	37	43	50	57	67
01 Jan 2000, 23:10	18	24	29	37	43	50	57	67
01 Jan 2000, 23:15	18	24	29	37	43	50	57	67
01 Jan 2000, 23:20	18	24	29	37	43	50	57	67
01 Jan 2000, 23:25	18	24	29	37	43	50	57	67
01 Jan 2000, 23:30	18	24	29	37	43	50	57	67
01 Jan 2000, 23:35	18	24	29	37	43	50	57	67
01 Jan 2000, 23:40	18	24	29	37	43	50	57	67
01 Jan 2000, 23:45	18	24	29	37	43	50	57	67
01 Jan 2000, 23:50	18	24	29	37	43	50	57	67
01 Jan 2000, 23:55	17	23	29	37	43	49	56	65
01 Jan 2000, 24:00	17	23	28	35	41	48	54	63

E.2.4 TC-T59

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	1	0	0	0	0
01 Jan 2000, 00:15	1	1	1	2	1	1	1	0
01 Jan 2000, 00:20	2	3	3	4	4	4	4	2
01 Jan 2000, 00:25	4	5	6	7	7	7	8	4
01 Jan 2000, 00:30	6	8	9	12	12	12	13	9
01 Jan 2000, 00:35	8	11	14	17	17	19	20	15
01 Jan 2000, 00:40	11	15	18	22	24	26	28	22
01 Jan 2000, 00:45	14	19	23	28	30	33	37	32
01 Jan 2000, 00:50	16	22	27	33	36	40	45	41
01 Jan 2000, 00:55	19	25	30	37	41	47	52	51
01 Jan 2000, 01:00	21	27	33	41	46	52	58	59
01 Jan 2000, 01:05	22	29	35	44	49	56	63	66
01 Jan 2000, 01:10	24	31	37	46	51	59	66	72
01 Jan 2000, 01:15	25	32	38	47	53	61	68	76
01 Jan 2000, 01:20	25	33	39	48	54	62	70	79
01 Jan 2000, 01:25	26	33	40	49	55	63	71	81
01 Jan 2000, 01:30	26	34	40	49	56	64	72	82
01 Jan 2000, 01:35	27	34	40	49	56	64	72	83
01 Jan 2000, 01:40	27	34	40	50	56	64	73	83
01 Jan 2000, 01:45	27	34	41	50	57	65	73	84
01 Jan 2000, 01:50	27	34	41	50	57	65	73	84
01 Jan 2000, 01:55	27	34	41	50	57	65	73	84
01 Jan 2000, 02:00	27	34	41	50	57	65	73	84
01 Jan 2000, 02:05	27	34	41	50	57	65	73	84
01 Jan 2000, 02:10	27	34	41	50	57	65	73	84
01 Jan 2000, 02:15	27	34	41	50	57	65	73	84
01 Jan 2000, 02:20	27	34	41	50	57	65	73	84
01 Jan 2000, 02:25	27	34	41	50	57	65	73	84
01 Jan 2000, 02:30	27	34	41	50	57	65	73	84
01 Jan 2000, 02:35	27	34	41	50	57	65	73	84
01 Jan 2000, 02:40	27	34	41	50	57	65	73	84
01 Jan 2000, 02:45	27	34	41	50	57	65	73	84
01 Jan 2000, 02:50	27	34	41	50	57	65	73	84
01 Jan 2000, 02:55	27	34	41	50	57	65	73	84
01 Jan 2000, 03:00	27	34	41	50	57	65	73	84

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:05	27	34	41	50	57	65	73	84
01 Jan 2000, 03:10	27	34	41	50	57	65	73	84
01 Jan 2000, 03:15	27	34	41	50	57	65	73	84
01 Jan 2000, 03:20	27	34	41	50	57	65	73	84
01 Jan 2000, 03:25	27	34	41	50	57	65	73	84
01 Jan 2000, 03:30	27	34	41	50	57	65	73	84
01 Jan 2000, 03:35	27	34	41	50	57	65	73	84
01 Jan 2000, 03:40	27	34	41	50	57	65	73	84
01 Jan 2000, 03:45	27	34	41	50	57	65	73	84
01 Jan 2000, 03:50	27	34	41	50	57	65	73	84
01 Jan 2000, 03:55	27	35	41	50	57	65	73	84
01 Jan 2000, 04:00	27	35	41	51	57	66	74	84
01 Jan 2000, 04:05	28	36	42	52	59	67	75	85
01 Jan 2000, 04:10	29	37	44	54	60	69	77	87
01 Jan 2000, 04:15	30	38	45	55	63	71	80	89
01 Jan 2000, 04:20	31	39	47	58	65	74	83	93
01 Jan 2000, 04:25	32	41	49	60	67	77	86	97
01 Jan 2000, 04:30	33	42	50	61	69	79	89	100
01 Jan 2000, 04:35	33	43	51	63	71	81	91	103
01 Jan 2000, 04:40	34	44	52	64	72	83	93	106
01 Jan 2000, 04:45	35	44	53	65	73	84	95	108
01 Jan 2000, 04:50	35	45	53	65	74	85	96	109
01 Jan 2000, 04:55	35	45	54	66	75	85	96	111
01 Jan 2000, 05:00	36	45	54	66	75	86	97	111
01 Jan 2000, 05:05	36	46	54	66	75	86	97	112
01 Jan 2000, 05:10	36	46	54	66	75	86	97	112
01 Jan 2000, 05:15	36	46	54	66	76	86	97	113
01 Jan 2000, 05:20	36	46	54	66	76	86	97	113
01 Jan 2000, 05:25	36	46	54	66	76	86	97	114
01 Jan 2000, 05:30	36	46	54	66	76	86	97	114
01 Jan 2000, 05:35	36	46	54	66	76	86	98	115
01 Jan 2000, 05:40	36	46	54	66	76	86	98	116
01 Jan 2000, 05:45	36	46	54	66	76	86	98	117
01 Jan 2000, 05:50	36	46	54	66	76	86	98	118
01 Jan 2000, 05:55	36	46	54	66	76	87	99	120
01 Jan 2000, 06:00	36	46	54	66	76	87	99	121
01 Jan 2000, 06:05	36	46	54	66	76	87	99	123
01 Jan 2000, 06:10	36	46	54	67	76	87	100	125
01 Jan 2000, 06:15	37	46	55	67	76	88	102	127
01 Jan 2000, 06:20	37	47	56	69	78	89	104	130

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:25	38	48	57	70	79	92	108	134
01 Jan 2000, 06:30	39	50	59	72	82	95	112	140
01 Jan 2000, 06:35	40	51	61	74	84	98	117	146
01 Jan 2000, 06:40	41	52	62	76	87	102	122	153
01 Jan 2000, 06:45	42	53	64	78	89	105	127	159
01 Jan 2000, 06:50	43	54	65	79	91	108	131	165
01 Jan 2000, 06:55	43	55	66	80	92	111	135	170
01 Jan 2000, 07:00	44	56	66	81	94	113	138	174
01 Jan 2000, 07:05	44	56	67	82	95	115	141	177
01 Jan 2000, 07:10	44	57	67	83	96	117	143	180
01 Jan 2000, 07:15	45	57	67	83	97	119	145	183
01 Jan 2000, 07:20	45	57	68	83	97	121	147	186
01 Jan 2000, 07:25	45	57	68	83	98	122	149	188
01 Jan 2000, 07:30	45	57	68	84	99	124	151	190
01 Jan 2000, 07:35	45	57	68	84	100	126	153	192
01 Jan 2000, 07:40	45	57	68	84	101	127	155	194
01 Jan 2000, 07:45	45	57	68	84	103	129	156	196
01 Jan 2000, 07:50	45	57	68	85	104	130	158	198
01 Jan 2000, 07:55	45	57	68	85	105	131	160	200
01 Jan 2000, 08:00	45	57	68	86	106	133	161	202
01 Jan 2000, 08:05	45	57	68	87	107	134	163	204
01 Jan 2000, 08:10	45	57	68	88	109	136	165	206
01 Jan 2000, 08:15	46	58	69	89	110	138	167	209
01 Jan 2000, 08:20	46	59	70	91	113	141	171	213
01 Jan 2000, 08:25	47	60	71	95	117	146	176	220
01 Jan 2000, 08:30	48	62	74	99	122	152	184	229
01 Jan 2000, 08:35	50	64	77	103	128	159	192	240
01 Jan 2000, 08:40	52	66	80	109	135	168	203	253
01 Jan 2000, 08:45	54	69	83	114	142	177	213	267
01 Jan 2000, 08:50	55	71	87	120	148	185	224	280
01 Jan 2000, 08:55	57	73	90	125	154	193	233	292
01 Jan 2000, 09:00	59	75	92	129	160	200	241	302
01 Jan 2000, 09:05	60	77	95	133	165	205	248	309
01 Jan 2000, 09:10	61	78	97	136	169	210	253	316
01 Jan 2000, 09:15	62	79	99	140	173	215	259	323
01 Jan 2000, 09:20	63	81	102	144	178	221	266	331
01 Jan 2000, 09:25	64	83	105	148	183	227	273	341
01 Jan 2000, 09:30	65	85	109	153	189	234	282	352
01 Jan 2000, 09:35	67	87	112	158	195	242	290	362
01 Jan 2000, 09:40	68	89	116	163	201	249	299	372

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:45	69	91	119	168	207	256	307	383
01 Jan 2000, 09:50	71	93	123	173	213	264	317	394
01 Jan 2000, 09:55	72	96	127	179	220	272	327	407
01 Jan 2000, 10:00	74	99	132	185	228	281	338	420
01 Jan 2000, 10:05	75	102	136	190	235	290	348	432
01 Jan 2000, 10:10	77	105	140	196	241	298	357	443
01 Jan 2000, 10:15	78	108	144	202	248	306	367	455
01 Jan 2000, 10:20	80	111	149	208	255	315	377	467
01 Jan 2000, 10:25	82	115	154	215	264	325	390	483
01 Jan 2000, 10:30	84	120	160	224	275	338	405	501
01 Jan 2000, 10:35	87	126	168	234	287	353	424	525
01 Jan 2000, 10:40	91	133	177	247	303	373	448	554
01 Jan 2000, 10:45	96	141	188	263	322	397	478	590
01 Jan 2000, 10:50	101	151	202	282	345	426	512	633
01 Jan 2000, 10:55	108	163	217	304	372	460	552	682
01 Jan 2000, 11:00	116	176	235	329	402	498	597	737
01 Jan 2000, 11:05	125	190	255	356	435	539	646	796
01 Jan 2000, 11:10	135	206	276	385	471	583	698	859
01 Jan 2000, 11:15	146	223	299	416	508	630	752	924
01 Jan 2000, 11:20	157	241	324	449	548	678	809	992
01 Jan 2000, 11:25	170	261	350	485	590	729	869	1064
01 Jan 2000, 11:30	184	283	378	523	638	786	935	1143
01 Jan 2000, 11:35	202	311	415	573	699	867	1021	1251
01 Jan 2000, 11:40	234	364	484	668	825	1034	1198	1475
01 Jan 2000, 11:45	297	471	628	885	1085	1381	1598	1966
01 Jan 2000, 11:50	421	671	910	1302	1600	1987	2317	2757
01 Jan 2000, 11:55	661	1057	1421	1995	2397	2789	3165	3621
01 Jan 2000, 12:00	1091	1687	2200	2876	3234	3628	4021	4529
01 Jan 2000, 12:05	1673	2465	2993	3711	4048	4482	4994	5588
01 Jan 2000, 12:10	2205	3057	3643	4437	4859	5369	5995	6645
01 Jan 2000, 12:15	2495	3402	4083	4963	5506	6109	6799	7525
01 Jan 2000, 12:20	2509	3481	4254	5193	5879	6589	7313	8130
01 Jan 2000, 12:25	2312	3332	4173	5171	5947	6774	7513	8434
01 Jan 2000, 12:30	1992	2928	3851	4945	5782	6671	7449	8465
01 Jan 2000, 12:35	1642	2321	3217	4514	5414	6353	7177	8262
01 Jan 2000, 12:40	1334	1802	2372	3726	4811	5809	6710	7850
01 Jan 2000, 12:45	1089	1446	1792	2556	3675	4937	5996	7212
01 Jan 2000, 12:50	905	1199	1461	1858	2373	3419	4823	6246
01 Jan 2000, 12:55	769	1018	1238	1559	1809	2095	2866	4713
01 Jan 2000, 13:00	666	883	1073	1346	1567	1801	1976	2570

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:05	587	779	946	1189	1373	1590	1821	2015
01 Jan 2000, 13:10	522	695	847	1063	1231	1420	1598	1904
01 Jan 2000, 13:15	470	627	765	962	1112	1285	1456	1675
01 Jan 2000, 13:20	429	572	698	879	1016	1172	1328	1545
01 Jan 2000, 13:25	395	525	642	811	936	1078	1224	1420
01 Jan 2000, 13:30	366	487	595	752	869	999	1135	1318
01 Jan 2000, 13:35	341	454	555	703	812	935	1061	1233
01 Jan 2000, 13:40	320	427	521	660	764	880	998	1159
01 Jan 2000, 13:45	302	402	491	623	721	831	942	1095
01 Jan 2000, 13:50	286	381	464	589	683	786	892	1036
01 Jan 2000, 13:55	271	361	441	558	648	746	846	983
01 Jan 2000, 14:00	258	343	419	531	616	710	805	935
01 Jan 2000, 14:05	246	328	400	506	588	678	769	893
01 Jan 2000, 14:10	235	314	383	485	563	649	736	856
01 Jan 2000, 14:15	226	301	368	465	541	623	707	822
01 Jan 2000, 14:20	217	290	354	447	520	600	681	791
01 Jan 2000, 14:25	209	279	340	431	501	578	656	763
01 Jan 2000, 14:30	202	269	328	416	483	557	633	736
01 Jan 2000, 14:35	194	260	317	401	466	538	611	710
01 Jan 2000, 14:40	187	250	306	387	449	519	589	684
01 Jan 2000, 14:45	181	242	295	373	433	500	568	659
01 Jan 2000, 14:50	174	233	284	359	417	482	546	635
01 Jan 2000, 14:55	168	225	274	346	402	464	526	611
01 Jan 2000, 15:00	162	217	265	334	388	448	508	591
01 Jan 2000, 15:05	157	210	256	324	377	435	493	573
01 Jan 2000, 15:10	153	205	250	316	367	425	482	560
01 Jan 2000, 15:15	149	200	245	310	360	416	473	550
01 Jan 2000, 15:20	146	197	241	305	354	410	466	543
01 Jan 2000, 15:25	144	194	238	301	350	406	461	537
01 Jan 2000, 15:30	142	191	234	297	345	401	456	531
01 Jan 2000, 15:35	139	188	231	292	340	395	448	522
01 Jan 2000, 15:40	137	184	226	287	333	387	439	511
01 Jan 2000, 15:45	134	181	221	281	326	378	430	499
01 Jan 2000, 15:50	132	177	217	275	319	370	420	488
01 Jan 2000, 15:55	129	174	213	270	313	363	412	479
01 Jan 2000, 16:00	127	171	210	266	308	357	406	471
01 Jan 2000, 16:05	125	169	207	262	304	352	401	466
01 Jan 2000, 16:10	124	167	205	260	301	349	397	462
01 Jan 2000, 16:15	123	166	203	258	299	347	395	460
01 Jan 2000, 16:20	122	165	202	256	298	345	393	458

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:25	121	164	201	256	297	345	392	457
01 Jan 2000, 16:30	121	164	201	255	296	344	392	457
01 Jan 2000, 16:35	121	163	201	255	296	344	392	456
01 Jan 2000, 16:40	121	163	200	254	295	343	391	455
01 Jan 2000, 16:45	120	162	199	252	293	341	388	452
01 Jan 2000, 16:50	119	160	197	249	290	337	383	445
01 Jan 2000, 16:55	117	157	193	245	284	330	375	436
01 Jan 2000, 17:00	115	154	189	239	278	322	366	425
01 Jan 2000, 17:05	112	151	185	234	271	315	357	414
01 Jan 2000, 17:10	110	148	181	229	265	307	349	405
01 Jan 2000, 17:15	108	145	178	225	260	302	343	398
01 Jan 2000, 17:20	107	143	175	221	257	297	337	392
01 Jan 2000, 17:25	105	141	173	219	254	293	334	388
01 Jan 2000, 17:30	104	140	171	217	251	291	331	385
01 Jan 2000, 17:35	103	139	170	215	250	290	329	383
01 Jan 2000, 17:40	102	138	169	214	249	289	328	382
01 Jan 2000, 17:45	102	138	169	214	248	288	328	382
01 Jan 2000, 17:50	102	137	168	214	248	288	327	381
01 Jan 2000, 17:55	101	137	168	213	247	287	327	380
01 Jan 2000, 18:00	101	136	167	211	245	285	324	377
01 Jan 2000, 18:05	99	134	164	208	242	281	320	371
01 Jan 2000, 18:10	98	131	161	203	236	274	312	362
01 Jan 2000, 18:15	95	128	157	198	230	267	304	352
01 Jan 2000, 18:20	93	125	152	193	223	259	295	341
01 Jan 2000, 18:25	91	122	148	188	218	252	287	332
01 Jan 2000, 18:30	89	119	145	183	213	246	280	324
01 Jan 2000, 18:35	87	117	142	180	209	241	274	318
01 Jan 2000, 18:40	86	115	140	177	205	237	270	314
01 Jan 2000, 18:45	85	113	138	175	203	235	267	310
01 Jan 2000, 18:50	84	112	137	174	201	233	265	308
01 Jan 2000, 18:55	83	111	136	173	200	232	264	307
01 Jan 2000, 19:00	82	111	136	172	199	231	263	306
01 Jan 2000, 19:05	82	111	136	172	199	231	263	306
01 Jan 2000, 19:10	82	110	135	171	199	231	262	306
01 Jan 2000, 19:15	82	110	135	171	199	231	262	305
01 Jan 2000, 19:20	82	110	135	171	199	230	262	305
01 Jan 2000, 19:25	82	110	135	171	199	230	262	305
01 Jan 2000, 19:30	82	110	135	171	199	230	262	305
01 Jan 2000, 19:35	82	110	135	171	199	230	262	305
01 Jan 2000, 19:40	82	110	135	171	199	230	262	305

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:45	82	110	135	171	199	230	262	305
01 Jan 2000, 19:50	82	110	135	171	199	230	262	305
01 Jan 2000, 19:55	82	110	135	171	198	230	262	305
01 Jan 2000, 20:00	81	109	134	169	197	228	260	302
01 Jan 2000, 20:05	80	107	131	166	193	224	256	297
01 Jan 2000, 20:10	78	105	128	162	188	218	249	289
01 Jan 2000, 20:15	76	102	124	157	182	211	240	279
01 Jan 2000, 20:20	74	98	120	151	176	204	232	269
01 Jan 2000, 20:25	72	95	116	146	170	196	223	260
01 Jan 2000, 20:30	69	92	112	142	165	190	216	251
01 Jan 2000, 20:35	68	90	110	138	160	185	210	245
01 Jan 2000, 20:40	66	88	107	135	157	181	206	240
01 Jan 2000, 20:45	65	87	105	133	154	178	203	236
01 Jan 2000, 20:50	64	86	104	132	153	177	201	234
01 Jan 2000, 20:55	63	85	103	131	151	175	199	232
01 Jan 2000, 21:00	63	84	103	130	151	174	198	231
01 Jan 2000, 21:05	62	84	102	130	150	174	198	230
01 Jan 2000, 21:10	62	84	102	129	150	174	198	230
01 Jan 2000, 21:15	62	83	102	129	150	174	197	230
01 Jan 2000, 21:20	62	83	102	129	150	173	197	230
01 Jan 2000, 21:25	62	83	102	129	149	173	197	230
01 Jan 2000, 21:30	62	83	102	129	149	173	197	229
01 Jan 2000, 21:35	62	83	102	129	149	173	197	229
01 Jan 2000, 21:40	62	83	102	129	149	173	197	229
01 Jan 2000, 21:45	62	83	102	129	149	173	197	229
01 Jan 2000, 21:50	62	83	102	129	149	173	197	229
01 Jan 2000, 21:55	62	83	102	129	149	173	197	229
01 Jan 2000, 22:00	62	83	102	129	150	173	197	230
01 Jan 2000, 22:05	62	83	102	129	150	173	197	230
01 Jan 2000, 22:10	62	83	102	129	150	173	197	230
01 Jan 2000, 22:15	62	83	102	129	150	173	197	230
01 Jan 2000, 22:20	62	83	102	129	150	173	197	230
01 Jan 2000, 22:25	62	83	102	129	150	173	197	230
01 Jan 2000, 22:30	62	83	102	129	150	173	197	230
01 Jan 2000, 22:35	62	83	102	129	150	173	197	230
01 Jan 2000, 22:40	62	83	102	129	150	174	197	230
01 Jan 2000, 22:45	62	84	102	129	150	174	197	230
01 Jan 2000, 22:50	62	84	102	129	150	174	197	230
01 Jan 2000, 22:55	62	84	102	129	150	174	197	230
01 Jan 2000, 23:00	62	84	102	129	150	174	197	230

TCT-59								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:05	62	84	102	129	150	174	197	230
01 Jan 2000, 23:10	62	84	102	129	150	174	197	230
01 Jan 2000, 23:15	62	84	102	129	150	174	197	230
01 Jan 2000, 23:20	62	84	102	129	150	174	197	230
01 Jan 2000, 23:25	62	84	102	129	150	174	197	230
01 Jan 2000, 23:30	62	84	102	129	150	174	197	230
01 Jan 2000, 23:35	62	84	102	129	150	174	197	230
01 Jan 2000, 23:40	62	84	102	129	150	174	198	230
01 Jan 2000, 23:45	62	84	102	129	150	174	198	230
01 Jan 2000, 23:50	62	84	102	129	150	174	198	230
01 Jan 2000, 23:55	62	83	102	129	149	173	197	229
01 Jan 2000, 24:00	61	82	101	127	148	172	195	228

E.2.5 TC66

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	1	1
01 Jan 2000, 00:10	1	1	1	1	1	2	2	2
01 Jan 2000, 00:15	1	2	2	2	3	3	3	4
01 Jan 2000, 00:20	2	2	2	3	3	4	4	5
01 Jan 2000, 00:25	2	2	3	4	4	5	5	6
01 Jan 2000, 00:30	2	3	3	4	5	5	6	7
01 Jan 2000, 00:35	2	3	4	4	5	6	7	8
01 Jan 2000, 00:40	3	3	4	5	5	6	7	8
01 Jan 2000, 00:45	3	3	4	5	5	6	7	8
01 Jan 2000, 00:50	3	3	4	5	5	6	7	8
01 Jan 2000, 00:55	3	3	4	5	5	6	7	8
01 Jan 2000, 01:00	3	3	4	5	5	6	7	8
01 Jan 2000, 01:05	3	3	4	5	5	6	7	8
01 Jan 2000, 01:10	3	3	4	5	5	6	7	8
01 Jan 2000, 01:15	3	3	4	5	5	6	7	8
01 Jan 2000, 01:20	3	3	4	5	5	6	7	8
01 Jan 2000, 01:25	3	3	4	5	5	6	7	8
01 Jan 2000, 01:30	3	3	4	5	5	6	7	8
01 Jan 2000, 01:35	3	3	4	5	5	6	7	8
01 Jan 2000, 01:40	3	3	4	5	5	6	7	8
01 Jan 2000, 01:45	3	3	4	5	5	6	7	8
01 Jan 2000, 01:50	3	3	4	5	5	6	7	8
01 Jan 2000, 01:55	3	3	4	5	5	6	7	8
01 Jan 2000, 02:00	3	3	4	5	5	6	7	8
01 Jan 2000, 02:05	3	3	4	5	5	6	7	8
01 Jan 2000, 02:10	3	3	4	5	5	6	7	8
01 Jan 2000, 02:15	3	3	4	5	5	6	7	8
01 Jan 2000, 02:20	3	3	4	5	5	6	7	8
01 Jan 2000, 02:25	3	3	4	5	5	6	7	8
01 Jan 2000, 02:30	3	3	4	5	5	6	7	8
01 Jan 2000, 02:35	3	3	4	5	5	6	7	8
01 Jan 2000, 02:40	3	3	4	5	5	6	7	8
01 Jan 2000, 02:45	3	3	4	5	5	6	7	8
01 Jan 2000, 02:50	3	3	4	5	5	6	7	8
01 Jan 2000, 02:55	3	3	4	5	5	6	7	8
01 Jan 2000, 03:00	3	3	4	5	5	6	7	8

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:05	3	3	4	5	5	6	7	8
01 Jan 2000, 03:10	3	3	4	5	5	6	7	8
01 Jan 2000, 03:15	3	3	4	5	5	6	7	8
01 Jan 2000, 03:20	3	3	4	5	5	6	7	8
01 Jan 2000, 03:25	3	3	4	5	5	6	7	8
01 Jan 2000, 03:30	3	3	4	5	5	6	7	8
01 Jan 2000, 03:35	3	3	4	5	5	6	7	8
01 Jan 2000, 03:40	3	3	4	5	5	6	7	8
01 Jan 2000, 03:45	3	3	4	5	5	6	7	8
01 Jan 2000, 03:50	3	3	4	5	6	6	7	8
01 Jan 2000, 03:55	3	4	4	5	6	7	8	9
01 Jan 2000, 04:00	3	4	5	6	7	8	9	10
01 Jan 2000, 04:05	3	4	5	6	7	8	9	10
01 Jan 2000, 04:10	3	4	5	6	7	8	9	11
01 Jan 2000, 04:15	3	4	5	6	7	8	9	11
01 Jan 2000, 04:20	3	4	5	6	7	8	9	11
01 Jan 2000, 04:25	3	4	5	6	7	8	9	11
01 Jan 2000, 04:30	3	4	5	6	7	8	9	11
01 Jan 2000, 04:35	3	4	5	6	7	8	9	11
01 Jan 2000, 04:40	3	4	5	6	7	8	9	12
01 Jan 2000, 04:45	3	4	5	6	7	8	9	12
01 Jan 2000, 04:50	3	4	5	6	7	8	9	12
01 Jan 2000, 04:55	3	4	5	6	7	8	10	12
01 Jan 2000, 05:00	3	4	5	6	7	8	10	12
01 Jan 2000, 05:05	3	4	5	6	7	8	10	13
01 Jan 2000, 05:10	3	4	5	6	7	8	10	13
01 Jan 2000, 05:15	3	4	5	6	7	8	10	13
01 Jan 2000, 05:20	3	4	5	6	7	8	10	13
01 Jan 2000, 05:25	3	4	5	6	7	9	10	13
01 Jan 2000, 05:30	3	4	5	6	7	9	11	13
01 Jan 2000, 05:35	3	4	5	6	7	9	11	14
01 Jan 2000, 05:40	3	4	5	6	7	9	11	14
01 Jan 2000, 05:45	3	4	5	6	7	9	11	14
01 Jan 2000, 05:50	3	4	5	6	7	9	11	14
01 Jan 2000, 05:55	3	4	5	6	7	9	11	14
01 Jan 2000, 06:00	3	4	5	6	7	9	11	14
01 Jan 2000, 06:05	4	4	5	7	8	10	12	15
01 Jan 2000, 06:10	4	5	6	7	8	11	13	16
01 Jan 2000, 06:15	4	5	6	7	9	11	14	18
01 Jan 2000, 06:20	4	5	6	8	10	12	15	18

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:25	4	5	6	8	10	12	15	19
01 Jan 2000, 06:30	4	5	6	8	10	13	15	19
01 Jan 2000, 06:35	4	5	6	8	10	13	16	19
01 Jan 2000, 06:40	4	5	6	8	10	13	16	20
01 Jan 2000, 06:45	4	5	6	8	11	13	16	20
01 Jan 2000, 06:50	4	5	6	9	11	13	16	20
01 Jan 2000, 06:55	4	5	6	9	11	13	16	20
01 Jan 2000, 07:00	4	5	6	9	11	14	16	21
01 Jan 2000, 07:05	4	5	6	9	11	14	17	21
01 Jan 2000, 07:10	4	5	6	9	11	14	17	21
01 Jan 2000, 07:15	4	5	6	9	11	14	17	21
01 Jan 2000, 07:20	4	5	7	9	11	14	17	21
01 Jan 2000, 07:25	4	5	7	9	12	14	17	22
01 Jan 2000, 07:30	4	5	7	9	12	14	17	22
01 Jan 2000, 07:35	4	5	7	9	12	15	18	22
01 Jan 2000, 07:40	4	5	7	10	12	15	18	22
01 Jan 2000, 07:45	4	5	7	10	12	15	18	22
01 Jan 2000, 07:50	4	5	7	10	12	15	18	22
01 Jan 2000, 07:55	4	5	7	10	12	15	18	23
01 Jan 2000, 08:00	4	5	7	10	12	15	18	23
01 Jan 2000, 08:05	4	6	7	10	13	16	19	23
01 Jan 2000, 08:10	5	6	8	11	14	17	20	25
01 Jan 2000, 08:15	5	6	8	12	14	18	21	27
01 Jan 2000, 08:20	5	7	9	12	15	19	23	28
01 Jan 2000, 08:25	5	7	10	13	16	20	24	30
01 Jan 2000, 08:30	6	8	10	14	17	21	26	32
01 Jan 2000, 08:35	6	8	10	15	18	22	27	33
01 Jan 2000, 08:40	6	8	11	15	18	23	27	34
01 Jan 2000, 08:45	6	8	11	15	19	23	28	34
01 Jan 2000, 08:50	6	8	11	15	19	23	28	34
01 Jan 2000, 08:55	6	8	11	16	19	24	28	35
01 Jan 2000, 09:00	6	9	11	16	19	24	28	35
01 Jan 2000, 09:05	6	9	12	16	20	24	29	36
01 Jan 2000, 09:10	6	9	12	17	21	26	31	38
01 Jan 2000, 09:15	7	10	13	18	22	27	32	39
01 Jan 2000, 09:20	7	10	13	18	23	28	33	40
01 Jan 2000, 09:25	7	10	14	19	23	28	34	41
01 Jan 2000, 09:30	7	10	14	19	23	29	34	42
01 Jan 2000, 09:35	7	11	14	20	24	29	35	43
01 Jan 2000, 09:40	8	11	15	21	25	31	36	45

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:45	8	12	16	21	26	32	38	46
01 Jan 2000, 09:50	8	12	16	22	27	33	39	48
01 Jan 2000, 09:55	8	12	16	22	27	33	40	48
01 Jan 2000, 10:00	8	13	17	23	28	34	40	49
01 Jan 2000, 10:05	9	13	17	23	28	35	41	50
01 Jan 2000, 10:10	9	14	18	24	30	36	43	52
01 Jan 2000, 10:15	9	14	18	25	31	37	44	54
01 Jan 2000, 10:20	10	15	19	27	32	39	46	56
01 Jan 2000, 10:25	11	16	21	29	35	42	50	60
01 Jan 2000, 10:30	12	17	22	31	37	45	53	64
01 Jan 2000, 10:35	12	18	24	33	40	48	56	68
01 Jan 2000, 10:40	14	20	26	36	44	53	62	75
01 Jan 2000, 10:45	15	22	29	39	47	57	67	81
01 Jan 2000, 10:50	16	24	31	42	50	61	72	86
01 Jan 2000, 10:55	18	26	34	45	55	66	78	94
01 Jan 2000, 11:00	19	28	36	49	59	71	83	100
01 Jan 2000, 11:05	20	30	38	52	62	75	88	106
01 Jan 2000, 11:10	22	32	41	56	67	81	95	114
01 Jan 2000, 11:15	23	34	44	59	71	86	101	121
01 Jan 2000, 11:20	25	36	47	63	76	91	107	129
01 Jan 2000, 11:25	28	40	52	70	84	100	117	141
01 Jan 2000, 11:30	30	44	56	75	90	108	126	151
01 Jan 2000, 11:35	43	62	80	107	128	153	178	213
01 Jan 2000, 11:40	77	111	142	188	225	268	311	371
01 Jan 2000, 11:45	111	158	202	267	318	378	438	521
01 Jan 2000, 11:50	163	231	293	384	455	539	623	739
01 Jan 2000, 11:55	266	373	470	612	721	850	979	1156
01 Jan 2000, 12:00	367	510	638	826	969	1138	1307	1537
01 Jan 2000, 12:05	386	533	663	854	999	1169	1340	1572
01 Jan 2000, 12:10	294	404	502	644	753	880	1007	1180
01 Jan 2000, 12:15	194	266	329	422	492	574	657	769
01 Jan 2000, 12:20	136	186	230	294	343	400	457	534
01 Jan 2000, 12:25	100	136	167	213	249	289	330	386
01 Jan 2000, 12:30	77	105	129	164	191	222	254	296
01 Jan 2000, 12:35	65	88	108	137	160	186	211	247
01 Jan 2000, 12:40	56	76	93	119	138	160	182	212
01 Jan 2000, 12:45	50	68	84	106	123	143	163	190
01 Jan 2000, 12:50	46	63	77	97	113	131	149	174
01 Jan 2000, 12:55	42	57	69	88	102	118	135	157
01 Jan 2000, 13:00	39	52	64	81	94	109	124	145

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:05	36	49	60	76	88	103	117	136
01 Jan 2000, 13:10	34	46	56	71	82	95	108	126
01 Jan 2000, 13:15	32	43	53	66	77	89	101	118
01 Jan 2000, 13:20	30	41	50	63	73	85	97	112
01 Jan 2000, 13:25	29	39	47	60	69	80	91	106
01 Jan 2000, 13:30	27	37	45	57	66	76	87	101
01 Jan 2000, 13:35	26	35	43	54	63	73	83	97
01 Jan 2000, 13:40	25	33	40	51	59	69	78	91
01 Jan 2000, 13:45	23	31	38	48	56	65	74	86
01 Jan 2000, 13:50	22	30	37	47	54	62	71	83
01 Jan 2000, 13:55	21	29	35	45	52	60	68	79
01 Jan 2000, 14:00	21	28	34	43	50	58	66	76
01 Jan 2000, 14:05	20	27	33	42	48	56	64	74
01 Jan 2000, 14:10	19	26	32	40	47	54	61	71
01 Jan 2000, 14:15	19	25	31	39	45	52	59	69
01 Jan 2000, 14:20	18	24	30	38	44	50	57	67
01 Jan 2000, 14:25	17	23	29	36	42	48	55	64
01 Jan 2000, 14:30	17	23	28	35	40	47	53	61
01 Jan 2000, 14:35	16	22	27	34	39	45	51	59
01 Jan 2000, 14:40	15	21	25	32	37	43	49	57
01 Jan 2000, 14:45	15	20	24	31	35	41	47	54
01 Jan 2000, 14:50	14	19	24	30	35	40	45	53
01 Jan 2000, 14:55	14	19	23	30	34	40	45	52
01 Jan 2000, 15:00	14	19	23	29	34	39	45	52
01 Jan 2000, 15:05	14	19	23	29	34	39	45	52
01 Jan 2000, 15:10	14	19	23	29	34	39	44	52
01 Jan 2000, 15:15	14	19	23	29	34	39	44	52
01 Jan 2000, 15:20	14	19	23	29	33	39	44	51
01 Jan 2000, 15:25	13	18	22	28	32	37	42	49
01 Jan 2000, 15:30	13	17	21	26	30	35	40	47
01 Jan 2000, 15:35	13	17	20	26	30	34	39	45
01 Jan 2000, 15:40	12	17	20	25	29	34	39	45
01 Jan 2000, 15:45	12	16	20	25	29	34	38	45
01 Jan 2000, 15:50	12	16	20	25	29	34	38	44
01 Jan 2000, 15:55	12	16	20	25	29	34	38	44
01 Jan 2000, 16:00	12	16	20	25	29	34	38	44
01 Jan 2000, 16:05	12	16	20	25	29	34	38	44
01 Jan 2000, 16:10	12	16	20	25	29	34	38	44
01 Jan 2000, 16:15	12	16	20	25	29	34	38	44
01 Jan 2000, 16:20	12	16	20	25	29	34	38	44

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:25	12	16	20	25	29	34	38	44
01 Jan 2000, 16:30	12	16	20	25	29	34	38	44
01 Jan 2000, 16:35	12	16	20	25	29	33	37	44
01 Jan 2000, 16:40	11	15	19	23	27	31	36	41
01 Jan 2000, 16:45	11	15	18	22	26	30	34	39
01 Jan 2000, 16:50	11	14	17	22	25	29	33	38
01 Jan 2000, 16:55	10	14	17	21	25	28	32	38
01 Jan 2000, 17:00	10	14	17	21	24	28	32	37
01 Jan 2000, 17:05	10	14	17	21	24	28	32	37
01 Jan 2000, 17:10	10	14	17	21	24	28	32	37
01 Jan 2000, 17:15	10	14	17	21	24	28	32	37
01 Jan 2000, 17:20	10	14	17	21	24	28	32	37
01 Jan 2000, 17:25	10	14	17	21	24	28	32	37
01 Jan 2000, 17:30	10	14	17	21	24	28	32	37
01 Jan 2000, 17:35	10	14	17	21	24	28	32	37
01 Jan 2000, 17:40	10	14	17	21	24	28	32	37
01 Jan 2000, 17:45	10	14	17	21	24	28	32	37
01 Jan 2000, 17:50	10	13	16	21	24	27	31	36
01 Jan 2000, 17:55	9	13	15	19	22	26	29	34
01 Jan 2000, 18:00	9	12	14	18	21	24	27	32
01 Jan 2000, 18:05	9	11	14	17	20	23	26	31
01 Jan 2000, 18:10	8	11	14	17	20	23	26	30
01 Jan 2000, 18:15	8	11	14	17	20	23	26	30
01 Jan 2000, 18:20	8	11	13	17	20	23	26	30
01 Jan 2000, 18:25	8	11	13	17	19	23	26	30
01 Jan 2000, 18:30	8	11	13	17	19	23	26	30
01 Jan 2000, 18:35	8	11	13	17	19	23	26	30
01 Jan 2000, 18:40	8	11	13	17	19	23	26	30
01 Jan 2000, 18:45	8	11	13	17	19	23	26	30
01 Jan 2000, 18:50	8	11	13	17	19	23	26	30
01 Jan 2000, 18:55	8	11	13	17	19	23	26	30
01 Jan 2000, 19:00	8	11	13	17	19	23	26	30
01 Jan 2000, 19:05	8	11	13	17	19	23	26	30
01 Jan 2000, 19:10	8	11	13	17	19	23	26	30
01 Jan 2000, 19:15	8	11	13	17	19	23	26	30
01 Jan 2000, 19:20	8	11	13	17	19	23	26	30
01 Jan 2000, 19:25	8	11	13	17	19	23	26	30
01 Jan 2000, 19:30	8	11	13	17	19	23	26	30
01 Jan 2000, 19:35	8	11	13	17	19	23	26	30
01 Jan 2000, 19:40	8	11	13	17	19	23	26	30

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:45	8	11	13	17	19	23	26	30
01 Jan 2000, 19:50	8	11	13	16	19	22	25	29
01 Jan 2000, 19:55	7	10	12	15	17	19	23	27
01 Jan 2000, 20:00	7	9	11	14	16	18	21	25
01 Jan 2000, 20:05	7	9	11	13	15	17	20	23
01 Jan 2000, 20:10	6	9	10	13	15	17	20	23
01 Jan 2000, 20:15	6	8	10	13	15	17	19	23
01 Jan 2000, 20:20	6	8	10	13	15	17	19	22
01 Jan 2000, 20:25	6	8	10	13	15	17	19	22
01 Jan 2000, 20:30	6	8	10	13	15	17	19	22
01 Jan 2000, 20:35	6	8	10	13	15	17	19	22
01 Jan 2000, 20:40	6	8	10	13	15	17	19	22
01 Jan 2000, 20:45	6	8	10	13	15	17	19	22
01 Jan 2000, 20:50	6	8	10	13	15	17	19	22
01 Jan 2000, 20:55	6	8	10	13	15	17	19	22
01 Jan 2000, 21:00	6	8	10	13	15	17	19	22
01 Jan 2000, 21:05	6	8	10	13	15	17	19	22
01 Jan 2000, 21:10	6	8	10	13	15	17	19	22
01 Jan 2000, 21:15	6	8	10	13	15	17	19	22
01 Jan 2000, 21:20	6	8	10	13	15	17	19	22
01 Jan 2000, 21:25	6	8	10	13	15	17	19	22
01 Jan 2000, 21:30	6	8	10	13	15	17	19	22
01 Jan 2000, 21:35	6	8	10	13	15	17	19	22
01 Jan 2000, 21:40	6	8	10	13	15	17	19	22
01 Jan 2000, 21:45	6	8	10	13	15	17	19	22
01 Jan 2000, 21:50	6	8	10	13	15	17	19	22
01 Jan 2000, 21:55	6	8	10	13	15	17	19	22
01 Jan 2000, 22:00	6	8	10	13	15	17	19	22
01 Jan 2000, 22:05	6	8	10	13	15	17	19	22
01 Jan 2000, 22:10	6	8	10	13	15	17	19	22
01 Jan 2000, 22:15	6	8	10	13	15	17	19	22
01 Jan 2000, 22:20	6	8	10	13	15	17	19	22
01 Jan 2000, 22:25	6	8	10	13	15	17	19	22
01 Jan 2000, 22:30	6	8	10	13	15	17	19	22
01 Jan 2000, 22:35	6	8	10	13	15	17	19	22
01 Jan 2000, 22:40	6	8	10	13	15	17	19	22
01 Jan 2000, 22:45	6	8	10	13	15	17	19	22
01 Jan 2000, 22:50	6	8	10	13	15	17	19	22
01 Jan 2000, 22:55	6	8	10	13	15	17	19	22
01 Jan 2000, 23:00	6	8	10	13	15	17	19	22

TC66								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:05	6	8	10	13	15	17	19	22
01 Jan 2000, 23:10	6	8	10	13	15	17	19	22
01 Jan 2000, 23:15	6	8	10	13	15	17	19	22
01 Jan 2000, 23:20	6	8	10	13	15	17	19	22
01 Jan 2000, 23:25	6	8	10	13	15	17	19	22
01 Jan 2000, 23:30	6	8	10	13	15	17	19	22
01 Jan 2000, 23:35	6	8	10	13	15	17	19	22
01 Jan 2000, 23:40	6	8	10	13	15	17	19	22
01 Jan 2000, 23:45	6	8	10	13	15	17	19	22
01 Jan 2000, 23:50	6	8	10	12	14	16	19	21
01 Jan 2000, 23:55	5	7	9	11	13	15	17	19
01 Jan 2000, 24:00	5	6	8	10	11	13	15	17

E.2.6 TC79

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	1	0	1	1
01 Jan 2000, 00:10	1	1	1	1	2	2	2	2
01 Jan 2000, 00:15	1	1	2	2	2	3	3	4
01 Jan 2000, 00:20	1	2	2	3	3	4	4	5
01 Jan 2000, 00:25	2	2	3	3	4	5	5	6
01 Jan 2000, 00:30	2	3	3	4	4	5	5	6
01 Jan 2000, 00:35	2	3	3	4	4	5	6	7
01 Jan 2000, 00:40	2	3	3	4	5	5	6	7
01 Jan 2000, 00:45	2	3	3	4	5	5	6	7
01 Jan 2000, 00:50	2	3	3	4	5	5	6	7
01 Jan 2000, 00:55	2	3	3	4	5	5	6	7
01 Jan 2000, 01:00	2	3	3	4	5	5	6	7
01 Jan 2000, 01:05	2	3	3	4	5	5	6	7
01 Jan 2000, 01:10	2	3	3	4	5	5	6	7
01 Jan 2000, 01:15	2	3	3	4	5	5	6	7
01 Jan 2000, 01:20	2	3	3	4	5	5	6	7
01 Jan 2000, 01:25	2	3	3	4	5	5	6	7
01 Jan 2000, 01:30	2	3	3	4	5	5	6	7
01 Jan 2000, 01:35	2	3	3	4	5	5	6	7
01 Jan 2000, 01:40	2	3	3	4	5	5	6	7
01 Jan 2000, 01:45	2	3	3	4	5	5	6	7
01 Jan 2000, 01:50	2	3	3	4	5	5	6	7
01 Jan 2000, 01:55	2	3	3	4	5	5	6	7
01 Jan 2000, 02:00	2	3	3	4	5	5	6	7
01 Jan 2000, 02:05	2	3	3	4	5	5	6	7
01 Jan 2000, 02:10	2	3	3	4	5	5	6	7
01 Jan 2000, 02:15	2	3	3	4	5	5	6	7
01 Jan 2000, 02:20	2	3	3	4	5	5	6	7
01 Jan 2000, 02:25	2	3	3	4	5	5	6	7
01 Jan 2000, 02:30	2	3	3	4	5	5	6	7
01 Jan 2000, 02:35	2	3	3	4	5	5	6	7
01 Jan 2000, 02:40	2	3	3	4	5	5	6	7
01 Jan 2000, 02:45	2	3	3	4	5	5	6	7
01 Jan 2000, 02:50	2	3	3	4	5	5	6	7
01 Jan 2000, 02:55	2	3	3	4	5	5	6	7
01 Jan 2000, 03:00	2	3	3	4	5	5	6	7

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:05	2	3	3	4	5	5	6	7
01 Jan 2000, 03:10	2	3	3	4	5	5	6	7
01 Jan 2000, 03:15	2	3	3	4	5	5	6	7
01 Jan 2000, 03:20	2	3	3	4	5	5	6	7
01 Jan 2000, 03:25	2	3	3	4	5	5	6	7
01 Jan 2000, 03:30	2	3	3	4	5	5	6	7
01 Jan 2000, 03:35	2	3	3	4	5	5	6	7
01 Jan 2000, 03:40	2	3	3	4	5	5	6	7
01 Jan 2000, 03:45	2	3	3	4	5	5	6	7
01 Jan 2000, 03:50	2	3	3	4	5	6	6	7
01 Jan 2000, 03:55	3	3	4	5	5	6	7	8
01 Jan 2000, 04:00	3	4	4	5	6	7	8	9
01 Jan 2000, 04:05	3	4	4	5	6	7	8	9
01 Jan 2000, 04:10	3	4	4	5	6	7	8	9
01 Jan 2000, 04:15	3	4	4	5	6	7	8	9
01 Jan 2000, 04:20	3	4	4	5	6	7	8	9
01 Jan 2000, 04:25	3	4	4	5	6	7	8	9
01 Jan 2000, 04:30	3	4	4	5	6	7	8	9
01 Jan 2000, 04:35	3	4	4	5	6	7	8	9
01 Jan 2000, 04:40	3	4	4	5	6	7	8	9
01 Jan 2000, 04:45	3	4	4	5	6	7	8	9
01 Jan 2000, 04:50	3	4	4	5	6	7	8	9
01 Jan 2000, 04:55	3	4	4	5	6	7	8	10
01 Jan 2000, 05:00	3	4	4	5	6	7	8	10
01 Jan 2000, 05:05	3	4	4	5	6	7	8	10
01 Jan 2000, 05:10	3	4	4	5	6	7	8	10
01 Jan 2000, 05:15	3	4	4	5	6	7	8	10
01 Jan 2000, 05:20	3	4	4	5	6	7	8	10
01 Jan 2000, 05:25	3	4	4	5	6	7	8	11
01 Jan 2000, 05:30	3	4	4	5	6	7	8	11
01 Jan 2000, 05:35	3	4	4	5	6	7	8	11
01 Jan 2000, 05:40	3	4	4	5	6	7	9	11
01 Jan 2000, 05:45	3	4	4	5	6	7	9	11
01 Jan 2000, 05:50	3	4	4	5	6	7	9	11
01 Jan 2000, 05:55	3	4	4	5	6	7	9	11
01 Jan 2000, 06:00	3	4	4	5	6	7	9	11
01 Jan 2000, 06:05	3	4	5	6	6	8	10	12
01 Jan 2000, 06:10	3	4	5	6	7	9	11	13
01 Jan 2000, 06:15	4	4	5	6	7	9	11	14
01 Jan 2000, 06:20	4	5	5	7	8	10	12	15

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:25	4	5	5	7	8	10	12	15
01 Jan 2000, 06:30	4	5	6	7	8	10	12	15
01 Jan 2000, 06:35	4	5	6	7	8	10	12	16
01 Jan 2000, 06:40	4	5	6	7	8	10	13	16
01 Jan 2000, 06:45	4	5	6	7	8	10	13	16
01 Jan 2000, 06:50	4	5	6	7	8	11	13	16
01 Jan 2000, 06:55	4	5	6	7	9	11	13	16
01 Jan 2000, 07:00	4	5	6	7	9	11	13	17
01 Jan 2000, 07:05	4	5	6	7	9	11	13	17
01 Jan 2000, 07:10	4	5	6	7	9	11	13	17
01 Jan 2000, 07:15	4	5	6	7	9	11	14	17
01 Jan 2000, 07:20	4	5	6	7	9	11	14	17
01 Jan 2000, 07:25	4	5	6	7	9	11	14	17
01 Jan 2000, 07:30	4	5	6	7	9	12	14	18
01 Jan 2000, 07:35	4	5	6	7	9	12	14	18
01 Jan 2000, 07:40	4	5	6	8	9	12	14	18
01 Jan 2000, 07:45	4	5	6	8	10	12	14	18
01 Jan 2000, 07:50	4	5	6	8	10	12	15	18
01 Jan 2000, 07:55	4	5	6	8	10	12	15	18
01 Jan 2000, 08:00	4	5	6	8	10	12	15	19
01 Jan 2000, 08:05	4	5	6	8	10	13	15	19
01 Jan 2000, 08:10	4	5	6	9	11	14	17	21
01 Jan 2000, 08:15	4	5	7	9	12	15	18	22
01 Jan 2000, 08:20	4	6	7	10	12	15	19	23
01 Jan 2000, 08:25	5	6	8	11	13	17	20	25
01 Jan 2000, 08:30	5	6	8	11	14	18	21	26
01 Jan 2000, 08:35	5	6	8	12	15	18	22	27
01 Jan 2000, 08:40	5	6	9	12	15	18	22	28
01 Jan 2000, 08:45	5	7	9	12	15	19	23	28
01 Jan 2000, 08:50	5	7	9	12	15	19	23	28
01 Jan 2000, 08:55	5	7	9	13	15	19	23	29
01 Jan 2000, 09:00	5	7	9	13	16	19	23	29
01 Jan 2000, 09:05	5	7	9	13	16	20	24	30
01 Jan 2000, 09:10	6	7	10	14	17	21	25	31
01 Jan 2000, 09:15	6	8	10	15	18	22	27	33
01 Jan 2000, 09:20	6	8	11	15	18	23	27	34
01 Jan 2000, 09:25	6	8	11	15	19	23	28	34
01 Jan 2000, 09:30	6	8	11	15	19	23	28	34
01 Jan 2000, 09:35	6	9	11	16	20	24	29	35
01 Jan 2000, 09:40	6	9	12	17	21	25	30	37

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:45	6	9	13	18	22	26	32	39
01 Jan 2000, 09:50	7	10	13	18	22	27	32	40
01 Jan 2000, 09:55	7	10	13	18	22	27	33	40
01 Jan 2000, 10:00	7	10	13	18	23	28	33	41
01 Jan 2000, 10:05	7	10	14	19	23	29	34	42
01 Jan 2000, 10:10	7	11	14	20	24	30	36	44
01 Jan 2000, 10:15	7	11	15	21	25	31	37	45
01 Jan 2000, 10:20	8	12	16	22	27	33	39	48
01 Jan 2000, 10:25	9	13	17	24	29	36	42	52
01 Jan 2000, 10:30	9	14	18	25	31	38	45	55
01 Jan 2000, 10:35	10	15	20	27	33	41	48	59
01 Jan 2000, 10:40	11	17	22	30	37	45	53	65
01 Jan 2000, 10:45	12	18	24	33	40	48	57	69
01 Jan 2000, 10:50	13	19	25	35	42	52	61	74
01 Jan 2000, 10:55	14	21	28	38	47	57	67	81
01 Jan 2000, 11:00	15	23	30	41	50	60	71	86
01 Jan 2000, 11:05	16	24	32	43	53	64	75	91
01 Jan 2000, 11:10	18	26	35	47	57	69	81	99
01 Jan 2000, 11:15	19	28	37	50	60	73	86	104
01 Jan 2000, 11:20	20	30	39	54	65	78	92	111
01 Jan 2000, 11:25	23	34	44	59	72	86	102	123
01 Jan 2000, 11:30	25	36	47	64	77	93	109	131
01 Jan 2000, 11:35	40	58	75	101	122	146	171	206
01 Jan 2000, 11:40	73	106	136	183	219	263	307	367
01 Jan 2000, 11:45	99	143	184	246	294	351	409	489
01 Jan 2000, 11:50	152	218	278	368	437	520	603	717
01 Jan 2000, 11:55	252	356	451	591	700	828	956	1132
01 Jan 2000, 12:00	334	468	588	765	902	1062	1222	1441
01 Jan 2000, 12:05	322	448	560	725	851	999	1148	1350
01 Jan 2000, 12:10	216	299	373	482	565	662	760	892
01 Jan 2000, 12:15	134	185	230	296	347	406	465	545
01 Jan 2000, 12:20	95	131	163	209	244	285	326	382
01 Jan 2000, 12:25	70	96	119	153	178	208	238	279
01 Jan 2000, 12:30	56	77	96	122	143	166	190	222
01 Jan 2000, 12:35	49	67	83	106	123	144	164	192
01 Jan 2000, 12:40	43	59	73	93	108	126	144	168
01 Jan 2000, 12:45	40	54	67	86	100	116	132	155
01 Jan 2000, 12:50	37	51	63	80	93	108	124	144
01 Jan 2000, 12:55	34	46	57	72	84	98	112	131
01 Jan 2000, 13:00	32	43	53	67	78	91	104	121

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:05	30	41	50	64	74	86	98	114
01 Jan 2000, 13:10	28	37	46	59	68	79	91	106
01 Jan 2000, 13:15	26	35	44	56	65	75	86	100
01 Jan 2000, 13:20	25	34	42	53	62	72	82	95
01 Jan 2000, 13:25	23	32	39	50	58	67	77	90
01 Jan 2000, 13:30	23	31	38	48	56	65	74	86
01 Jan 2000, 13:35	22	29	36	46	53	62	70	82
01 Jan 2000, 13:40	20	27	34	43	50	58	66	77
01 Jan 2000, 13:45	19	26	32	41	47	55	63	73
01 Jan 2000, 13:50	19	25	31	39	46	53	60	70
01 Jan 2000, 13:55	18	24	30	38	44	51	58	67
01 Jan 2000, 14:00	17	23	29	37	42	49	56	65
01 Jan 2000, 14:05	17	23	28	35	41	48	54	64
01 Jan 2000, 14:10	16	22	27	34	39	46	52	61
01 Jan 2000, 14:15	16	21	26	33	38	44	51	59
01 Jan 2000, 14:20	15	20	25	32	37	43	49	57
01 Jan 2000, 14:25	14	20	24	30	35	41	47	54
01 Jan 2000, 14:30	14	19	23	29	34	40	45	53
01 Jan 2000, 14:35	13	18	22	28	33	38	44	51
01 Jan 2000, 14:40	13	17	21	27	31	36	41	48
01 Jan 2000, 14:45	12	17	20	26	30	35	40	46
01 Jan 2000, 14:50	12	16	20	25	30	34	39	45
01 Jan 2000, 14:55	12	16	20	25	29	34	39	45
01 Jan 2000, 15:00	12	16	20	25	29	34	39	45
01 Jan 2000, 15:05	12	16	20	25	29	34	39	45
01 Jan 2000, 15:10	12	16	20	25	29	34	39	45
01 Jan 2000, 15:15	12	16	20	25	29	34	39	45
01 Jan 2000, 15:20	12	16	19	25	28	33	38	44
01 Jan 2000, 15:25	11	15	18	23	27	31	36	41
01 Jan 2000, 15:30	11	14	18	22	26	30	34	40
01 Jan 2000, 15:35	10	14	17	22	25	29	34	39
01 Jan 2000, 15:40	10	14	17	22	25	29	33	39
01 Jan 2000, 15:45	10	14	17	22	25	29	33	39
01 Jan 2000, 15:50	10	14	17	22	25	29	33	39
01 Jan 2000, 15:55	10	14	17	22	25	29	33	39
01 Jan 2000, 16:00	10	14	17	22	25	29	33	39
01 Jan 2000, 16:05	10	14	17	22	25	29	33	39
01 Jan 2000, 16:10	10	14	17	22	25	29	33	39
01 Jan 2000, 16:15	10	14	17	22	25	29	33	39
01 Jan 2000, 16:20	10	14	17	22	25	29	33	39

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:25	10	14	17	22	25	29	33	39
01 Jan 2000, 16:30	10	14	17	22	25	29	33	39
01 Jan 2000, 16:35	10	14	17	21	24	28	32	38
01 Jan 2000, 16:40	9	13	16	20	23	27	30	35
01 Jan 2000, 16:45	9	12	15	19	22	25	29	33
01 Jan 2000, 16:50	9	12	15	18	21	25	28	33
01 Jan 2000, 16:55	9	12	14	18	21	24	28	32
01 Jan 2000, 17:00	9	12	14	18	21	24	28	32
01 Jan 2000, 17:05	9	12	14	18	21	24	28	32
01 Jan 2000, 17:10	9	12	14	18	21	24	28	32
01 Jan 2000, 17:15	9	12	14	18	21	24	28	32
01 Jan 2000, 17:20	9	12	14	18	21	24	28	32
01 Jan 2000, 17:25	9	12	14	18	21	24	28	32
01 Jan 2000, 17:30	9	12	14	18	21	24	28	32
01 Jan 2000, 17:35	9	12	14	18	21	24	28	32
01 Jan 2000, 17:40	9	12	14	18	21	24	28	32
01 Jan 2000, 17:45	9	12	14	18	21	24	28	32
01 Jan 2000, 17:50	8	11	14	18	20	24	27	31
01 Jan 2000, 17:55	8	10	13	16	19	22	25	29
01 Jan 2000, 18:00	7	10	12	15	18	20	23	27
01 Jan 2000, 18:05	7	10	12	15	17	20	23	26
01 Jan 2000, 18:10	7	9	12	15	17	20	22	26
01 Jan 2000, 18:15	7	9	12	15	17	20	22	26
01 Jan 2000, 18:20	7	9	12	15	17	20	22	26
01 Jan 2000, 18:25	7	9	12	15	17	20	22	26
01 Jan 2000, 18:30	7	9	12	15	17	19	22	26
01 Jan 2000, 18:35	7	9	12	15	17	19	22	26
01 Jan 2000, 18:40	7	9	12	15	17	20	22	26
01 Jan 2000, 18:45	7	9	12	15	17	20	22	26
01 Jan 2000, 18:50	7	9	12	15	17	20	22	26
01 Jan 2000, 18:55	7	9	12	15	17	20	22	26
01 Jan 2000, 19:00	7	9	12	15	17	20	22	26
01 Jan 2000, 19:05	7	9	12	15	17	20	22	26
01 Jan 2000, 19:10	7	9	12	15	17	20	22	26
01 Jan 2000, 19:15	7	9	12	15	17	20	22	26
01 Jan 2000, 19:20	7	9	12	15	17	20	22	26
01 Jan 2000, 19:25	7	9	12	15	17	20	22	26
01 Jan 2000, 19:30	7	9	12	15	17	20	22	26
01 Jan 2000, 19:35	7	9	12	15	17	20	22	26
01 Jan 2000, 19:40	7	9	12	15	17	20	22	26

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:45	7	9	12	15	17	20	22	26
01 Jan 2000, 19:50	7	9	11	14	16	19	21	25
01 Jan 2000, 19:55	6	8	10	13	15	17	19	22
01 Jan 2000, 20:00	6	8	9	12	14	16	18	21
01 Jan 2000, 20:05	5	7	9	11	13	15	17	20
01 Jan 2000, 20:10	5	7	9	11	13	15	17	20
01 Jan 2000, 20:15	5	7	9	11	13	15	17	19
01 Jan 2000, 20:20	5	7	9	11	13	15	17	19
01 Jan 2000, 20:25	5	7	9	11	13	15	17	19
01 Jan 2000, 20:30	5	7	9	11	13	15	17	19
01 Jan 2000, 20:35	5	7	9	11	13	15	17	19
01 Jan 2000, 20:40	5	7	9	11	13	15	17	19
01 Jan 2000, 20:45	5	7	9	11	13	15	17	19
01 Jan 2000, 20:50	5	7	9	11	13	15	17	19
01 Jan 2000, 20:55	5	7	9	11	13	15	17	19
01 Jan 2000, 21:00	5	7	9	11	13	15	17	19
01 Jan 2000, 21:05	5	7	9	11	13	15	17	19
01 Jan 2000, 21:10	5	7	9	11	13	15	17	19
01 Jan 2000, 21:15	5	7	9	11	13	15	17	19
01 Jan 2000, 21:20	5	7	9	11	13	15	17	19
01 Jan 2000, 21:25	5	7	9	11	13	15	17	19
01 Jan 2000, 21:30	5	7	9	11	13	15	17	19
01 Jan 2000, 21:35	5	7	9	11	13	15	17	19
01 Jan 2000, 21:40	5	7	9	11	13	15	17	19
01 Jan 2000, 21:45	5	7	9	11	13	15	17	19
01 Jan 2000, 21:50	5	7	9	11	13	15	17	19
01 Jan 2000, 21:55	5	7	9	11	13	15	17	19
01 Jan 2000, 22:00	5	7	9	11	13	15	17	19
01 Jan 2000, 22:05	5	7	9	11	13	15	17	19
01 Jan 2000, 22:10	5	7	9	11	13	15	17	19
01 Jan 2000, 22:15	5	7	9	11	13	15	17	19
01 Jan 2000, 22:20	5	7	9	11	13	15	17	19
01 Jan 2000, 22:25	5	7	9	11	13	15	17	19
01 Jan 2000, 22:30	5	7	9	11	13	15	17	19
01 Jan 2000, 22:35	5	7	9	11	13	15	17	19
01 Jan 2000, 22:40	5	7	9	11	13	15	17	19
01 Jan 2000, 22:45	5	7	9	11	13	15	17	19
01 Jan 2000, 22:50	5	7	9	11	13	15	17	19
01 Jan 2000, 22:55	5	7	9	11	13	15	17	19
01 Jan 2000, 23:00	5	7	9	11	13	15	17	19

TC79								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:05	5	7	9	11	13	15	17	19
01 Jan 2000, 23:10	5	7	9	11	13	15	17	19
01 Jan 2000, 23:15	5	7	9	11	13	15	17	19
01 Jan 2000, 23:20	5	7	9	11	13	15	17	19
01 Jan 2000, 23:25	5	7	9	11	13	15	17	19
01 Jan 2000, 23:30	5	7	9	11	13	15	17	19
01 Jan 2000, 23:35	5	7	9	11	13	15	17	19
01 Jan 2000, 23:40	5	7	9	11	13	15	17	19
01 Jan 2000, 23:45	5	7	9	11	13	15	17	19
01 Jan 2000, 23:50	5	7	8	10	12	14	16	18
01 Jan 2000, 23:55	4	6	7	9	10	12	14	16
01 Jan 2000, 24:00	4	5	6	8	9	11	12	14

E.2.7 TCT50-TC64

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	0	0	1	1	1
01 Jan 2000, 00:15	0	1	1	1	1	3	2	2
01 Jan 2000, 00:20	2	2	2	2	4	6	4	4
01 Jan 2000, 00:25	2	3	4	5	5	8	7	8
01 Jan 2000, 00:30	3	5	5	7	8	10	10	11
01 Jan 2000, 00:35	5	6	7	8	9	13	12	14
01 Jan 2000, 00:40	6	7	8	10	12	14	15	17
01 Jan 2000, 00:45	6	8	9	12	13	16	16	19
01 Jan 2000, 00:50	6	9	10	12	13	16	18	20
01 Jan 2000, 00:55	7	9	10	13	14	17	19	22
01 Jan 2000, 01:00	7	9	10	13	15	17	19	22
01 Jan 2000, 01:05	7	9	10	13	15	17	19	22
01 Jan 2000, 01:10	7	9	11	13	15	17	19	22
01 Jan 2000, 01:15	7	9	11	13	16	17	20	23
01 Jan 2000, 01:20	7	9	11	13	16	17	20	23
01 Jan 2000, 01:25	7	9	11	13	16	17	20	23
01 Jan 2000, 01:30	7	9	11	13	16	17	20	23
01 Jan 2000, 01:35	7	9	11	13	16	17	20	23
01 Jan 2000, 01:40	7	9	11	13	16	17	20	23
01 Jan 2000, 01:45	7	9	11	13	16	17	20	23
01 Jan 2000, 01:50	7	9	11	13	16	17	20	23
01 Jan 2000, 01:55	7	9	11	13	16	17	20	23
01 Jan 2000, 02:00	7	9	11	13	16	17	20	23
01 Jan 2000, 02:05	7	9	11	13	16	17	20	23
01 Jan 2000, 02:10	7	9	11	13	16	17	20	23
01 Jan 2000, 02:15	7	9	11	13	16	17	20	23
01 Jan 2000, 02:20	7	9	11	13	16	17	20	23
01 Jan 2000, 02:25	7	9	11	13	16	17	20	23
01 Jan 2000, 02:30	7	9	11	13	16	17	20	23
01 Jan 2000, 02:35	7	9	11	13	16	17	20	23
01 Jan 2000, 02:40	7	9	11	13	16	17	20	23
01 Jan 2000, 02:45	7	9	11	13	16	17	20	23
01 Jan 2000, 02:50	7	9	11	13	16	17	20	23
01 Jan 2000, 02:55	7	9	11	13	16	17	20	23

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	7	9	11	13	16	17	20	23
01 Jan 2000, 03:05	7	9	11	13	16	17	20	23
01 Jan 2000, 03:10	7	9	11	13	16	17	20	23
01 Jan 2000, 03:15	7	9	11	13	16	17	20	23
01 Jan 2000, 03:20	7	9	11	13	16	17	20	23
01 Jan 2000, 03:25	7	9	11	13	16	17	20	23
01 Jan 2000, 03:30	7	9	11	13	16	17	20	23
01 Jan 2000, 03:35	7	9	11	13	16	17	20	23
01 Jan 2000, 03:40	7	9	11	13	16	17	20	23
01 Jan 2000, 03:45	7	9	11	13	16	17	20	23
01 Jan 2000, 03:50	7	9	11	13	16	17	20	23
01 Jan 2000, 03:55	7	9	12	13	16	18	20	23
01 Jan 2000, 04:00	7	10	12	15	16	18	21	24
01 Jan 2000, 04:05	8	10	12	15	16	19	22	25
01 Jan 2000, 04:10	9	10	13	16	18	21	23	27
01 Jan 2000, 04:15	9	12	13	16	19	22	25	28
01 Jan 2000, 04:20	9	12	14	17	19	22	25	29
01 Jan 2000, 04:25	10	12	14	17	20	23	26	29
01 Jan 2000, 04:30	10	12	14	17	20	23	26	30
01 Jan 2000, 04:35	10	12	14	17	20	23	26	30
01 Jan 2000, 04:40	10	13	14	17	20	23	26	30
01 Jan 2000, 04:45	10	13	14	17	20	23	26	30
01 Jan 2000, 04:50	10	13	14	17	20	23	26	30
01 Jan 2000, 04:55	10	13	14	17	20	23	26	30
01 Jan 2000, 05:00	10	13	14	17	20	23	26	30
01 Jan 2000, 05:05	10	13	14	17	20	23	26	30
01 Jan 2000, 05:10	10	13	14	17	20	23	26	30
01 Jan 2000, 05:15	10	13	14	17	20	23	26	30
01 Jan 2000, 05:20	10	13	14	17	20	23	26	30
01 Jan 2000, 05:25	10	13	14	17	20	23	26	31
01 Jan 2000, 05:30	10	13	14	17	20	23	26	32
01 Jan 2000, 05:35	10	13	14	17	20	23	26	32
01 Jan 2000, 05:40	10	13	14	17	20	23	26	32
01 Jan 2000, 05:45	10	13	14	17	20	23	26	33
01 Jan 2000, 05:50	10	13	14	17	20	23	26	33
01 Jan 2000, 05:55	10	13	14	17	20	23	26	33
01 Jan 2000, 06:00	10	13	14	17	20	23	27	34
01 Jan 2000, 06:05	10	13	15	18	20	23	27	34
01 Jan 2000, 06:10	10	13	15	18	20	23	28	35
01 Jan 2000, 06:15	10	13	15	19	21	25	29	36

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	10	13	16	19	22	25	31	38
01 Jan 2000, 06:25	10	14	16	20	23	26	32	41
01 Jan 2000, 06:30	12	15	17	21	23	28	34	43
01 Jan 2000, 06:35	12	15	17	22	25	29	35	45
01 Jan 2000, 06:40	12	15	18	22	25	29	36	46
01 Jan 2000, 06:45	12	16	18	22	25	30	37	47
01 Jan 2000, 06:50	12	16	19	22	26	30	37	47
01 Jan 2000, 06:55	12	16	19	22	26	32	39	48
01 Jan 2000, 07:00	12	16	19	23	26	32	39	49
01 Jan 2000, 07:05	12	16	19	23	26	32	39	50
01 Jan 2000, 07:10	12	16	19	23	26	33	40	50
01 Jan 2000, 07:15	12	16	19	23	26	33	40	51
01 Jan 2000, 07:20	12	16	19	23	27	33	41	51
01 Jan 2000, 07:25	12	16	19	23	27	34	41	52
01 Jan 2000, 07:30	12	16	19	23	27	34	41	52
01 Jan 2000, 07:35	12	16	19	23	27	34	43	53
01 Jan 2000, 07:40	12	16	19	23	28	35	43	54
01 Jan 2000, 07:45	12	16	19	23	29	36	43	54
01 Jan 2000, 07:50	12	16	19	23	29	36	44	55
01 Jan 2000, 07:55	12	16	19	23	29	36	44	55
01 Jan 2000, 08:00	12	16	19	23	29	37	44	55
01 Jan 2000, 08:05	12	16	19	23	30	37	44	56
01 Jan 2000, 08:10	12	16	19	24	30	37	46	57
01 Jan 2000, 08:15	13	16	19	25	31	39	47	59
01 Jan 2000, 08:20	13	16	19	26	32	41	49	62
01 Jan 2000, 08:25	13	17	21	28	34	42	52	65
01 Jan 2000, 08:30	15	18	22	29	36	45	55	69
01 Jan 2000, 08:35	15	19	22	31	38	48	58	72
01 Jan 2000, 08:40	16	20	23	32	41	50	61	76
01 Jan 2000, 08:45	16	20	25	35	43	53	64	80
01 Jan 2000, 08:50	16	20	25	36	44	54	65	82
01 Jan 2000, 08:55	17	22	26	36	44	56	68	83
01 Jan 2000, 09:00	17	22	26	37	46	57	68	85
01 Jan 2000, 09:05	17	22	26	37	47	58	69	86
01 Jan 2000, 09:10	17	22	27	39	47	59	71	88
01 Jan 2000, 09:15	17	22	28	39	49	60	73	91
01 Jan 2000, 09:20	18	22	29	41	50	63	75	94
01 Jan 2000, 09:25	19	23	30	43	53	65	78	97
01 Jan 2000, 09:30	19	23	32	44	54	67	81	100
01 Jan 2000, 09:35	19	25	32	46	56	69	83	103

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	19	25	33	46	57	71	85	105
01 Jan 2000, 09:45	20	26	34	48	59	73	87	108
01 Jan 2000, 09:50	20	26	35	49	61	75	90	112
01 Jan 2000, 09:55	20	28	36	51	63	78	93	116
01 Jan 2000, 10:00	22	29	38	53	65	81	96	118
01 Jan 2000, 10:05	22	29	39	54	67	82	99	121
01 Jan 2000, 10:10	22	30	40	56	68	84	100	124
01 Jan 2000, 10:15	22	31	41	57	70	87	103	127
01 Jan 2000, 10:20	23	32	42	59	73	90	106	131
01 Jan 2000, 10:25	23	34	45	62	76	93	111	136
01 Jan 2000, 10:30	24	35	47	65	79	97	117	144
01 Jan 2000, 10:35	25	38	50	69	85	103	123	151
01 Jan 2000, 10:40	27	40	53	74	91	111	132	161
01 Jan 2000, 10:45	28	43	57	79	97	118	141	173
01 Jan 2000, 10:50	31	47	62	86	105	128	152	187
01 Jan 2000, 10:55	34	51	67	93	114	138	165	201
01 Jan 2000, 11:00	37	55	73	100	123	149	178	217
01 Jan 2000, 11:05	40	60	79	108	132	162	192	234
01 Jan 2000, 11:10	43	64	85	117	142	173	206	252
01 Jan 2000, 11:15	47	70	91	125	153	186	221	269
01 Jan 2000, 11:20	50	74	98	135	165	200	236	288
01 Jan 2000, 11:25	54	80	106	145	176	214	253	308
01 Jan 2000, 11:30	59	87	114	156	190	230	271	331
01 Jan 2000, 11:35	66	98	128	175	211	257	304	371
01 Jan 2000, 11:40	82	120	157	212	257	312	368	463
01 Jan 2000, 11:45	112	166	217	295	359	437	518	658
01 Jan 2000, 11:50	170	252	330	454	552	674	800	997
01 Jan 2000, 11:55	265	390	510	695	845	1031	1223	1527
01 Jan 2000, 12:00	407	598	783	1074	1307	1592	1886	2306
01 Jan 2000, 12:05	597	876	1142	1556	1878	2257	2632	3130
01 Jan 2000, 12:10	765	1102	1411	1868	2215	2618	3030	3582
01 Jan 2000, 12:15	796	1115	1398	1809	2125	2500	2873	3360
01 Jan 2000, 12:20	697	959	1189	1519	1771	2070	2367	2767
01 Jan 2000, 12:25	556	760	938	1195	1386	1608	1835	2154
01 Jan 2000, 12:30	431	589	728	925	1076	1246	1412	1662
01 Jan 2000, 12:35	337	460	569	724	842	978	1113	1308
01 Jan 2000, 12:40	269	369	456	583	677	787	896	1057
01 Jan 2000, 12:45	221	304	376	480	558	649	740	872
01 Jan 2000, 12:50	187	256	317	406	473	551	627	739
01 Jan 2000, 12:55	162	222	276	352	410	478	544	640

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	143	197	243	312	363	423	481	564
01 Jan 2000, 13:05	129	176	218	278	323	377	430	503
01 Jan 2000, 13:10	117	159	196	251	293	340	388	454
01 Jan 2000, 13:15	106	145	179	229	267	310	354	414
01 Jan 2000, 13:20	98	133	165	210	245	285	326	380
01 Jan 2000, 13:25	91	124	152	195	227	265	301	352
01 Jan 2000, 13:30	85	116	143	182	212	247	281	329
01 Jan 2000, 13:35	80	109	135	172	200	233	265	309
01 Jan 2000, 13:40	75	103	127	162	189	219	251	293
01 Jan 2000, 13:45	72	98	121	154	179	208	238	277
01 Jan 2000, 13:50	68	93	114	146	170	197	226	263
01 Jan 2000, 13:55	64	89	109	139	161	188	214	249
01 Jan 2000, 14:00	62	85	104	132	154	179	204	239
01 Jan 2000, 14:05	59	81	100	127	148	171	195	228
01 Jan 2000, 14:10	56	78	96	122	142	165	188	220
01 Jan 2000, 14:15	55	75	92	118	137	159	182	212
01 Jan 2000, 14:20	53	72	89	113	131	153	175	204
01 Jan 2000, 14:25	51	70	86	109	127	148	168	197
01 Jan 2000, 14:30	49	67	83	105	123	142	163	190
01 Jan 2000, 14:35	48	64	79	101	118	138	157	183
01 Jan 2000, 14:40	45	63	77	98	113	132	150	176
01 Jan 2000, 14:45	44	60	74	94	109	127	145	169
01 Jan 2000, 14:50	42	58	71	90	105	122	139	162
01 Jan 2000, 14:55	41	55	68	87	101	118	134	156
01 Jan 2000, 15:00	40	54	66	84	98	114	130	152
01 Jan 2000, 15:05	39	53	65	83	96	112	128	149
01 Jan 2000, 15:10	39	52	64	81	95	110	126	147
01 Jan 2000, 15:15	38	51	64	81	94	109	124	146
01 Jan 2000, 15:20	38	51	63	80	94	109	124	145
01 Jan 2000, 15:25	37	51	63	80	92	107	122	143
01 Jan 2000, 15:30	37	51	62	78	92	107	121	141
01 Jan 2000, 15:35	36	49	60	77	89	104	118	137
01 Jan 2000, 15:40	35	48	59	74	86	100	114	133
01 Jan 2000, 15:45	34	47	57	73	84	98	111	130
01 Jan 2000, 15:50	33	45	55	70	83	96	109	128
01 Jan 2000, 15:55	33	44	55	70	81	95	108	125
01 Jan 2000, 16:00	33	44	55	69	81	94	107	125
01 Jan 2000, 16:05	33	44	54	69	80	94	106	124
01 Jan 2000, 16:10	32	44	54	69	80	94	106	124
01 Jan 2000, 16:15	32	44	54	69	80	93	106	124

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	32	44	54	69	80	93	106	124
01 Jan 2000, 16:25	32	44	54	69	80	93	106	123
01 Jan 2000, 16:30	32	44	54	69	80	93	106	123
01 Jan 2000, 16:35	32	44	54	69	80	93	106	123
01 Jan 2000, 16:40	32	44	54	69	80	92	106	122
01 Jan 2000, 16:45	32	43	53	67	78	90	104	120
01 Jan 2000, 16:50	31	42	52	65	76	89	100	117
01 Jan 2000, 16:55	30	41	49	63	73	85	97	113
01 Jan 2000, 17:00	29	40	48	62	71	82	93	110
01 Jan 2000, 17:05	29	39	47	59	69	81	92	107
01 Jan 2000, 17:10	28	37	47	59	69	80	91	106
01 Jan 2000, 17:15	28	37	46	58	68	79	90	105
01 Jan 2000, 17:20	28	37	46	58	68	79	90	104
01 Jan 2000, 17:25	28	37	46	58	68	79	88	104
01 Jan 2000, 17:30	28	37	46	58	68	77	88	103
01 Jan 2000, 17:35	28	37	46	58	67	77	88	103
01 Jan 2000, 17:40	28	37	46	58	66	77	88	103
01 Jan 2000, 17:45	28	37	46	58	66	77	88	103
01 Jan 2000, 17:50	28	37	46	58	66	77	88	103
01 Jan 2000, 17:55	27	37	45	58	66	77	88	102
01 Jan 2000, 18:00	26	36	44	56	65	75	86	100
01 Jan 2000, 18:05	26	35	42	54	63	73	83	97
01 Jan 2000, 18:10	25	33	41	52	60	70	79	93
01 Jan 2000, 18:15	23	32	40	49	58	67	77	89
01 Jan 2000, 18:20	23	32	38	48	56	66	74	87
01 Jan 2000, 18:25	22	30	37	48	55	65	73	85
01 Jan 2000, 18:30	22	30	37	47	55	63	72	84
01 Jan 2000, 18:35	22	30	37	47	54	63	72	84
01 Jan 2000, 18:40	22	30	37	47	54	62	72	83
01 Jan 2000, 18:45	22	30	36	47	54	62	72	83
01 Jan 2000, 18:50	22	30	36	47	54	62	71	83
01 Jan 2000, 18:55	22	29	36	47	54	62	71	83
01 Jan 2000, 19:00	22	29	36	46	54	62	71	83
01 Jan 2000, 19:05	22	29	36	46	54	62	71	83
01 Jan 2000, 19:10	22	29	36	46	54	62	71	83
01 Jan 2000, 19:15	22	29	36	46	54	62	71	83
01 Jan 2000, 19:20	22	29	36	46	54	62	71	83
01 Jan 2000, 19:25	22	29	36	46	54	62	71	83
01 Jan 2000, 19:30	22	30	36	46	54	62	71	83
01 Jan 2000, 19:35	22	30	36	47	54	62	71	83

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	22	30	36	47	54	62	71	83
01 Jan 2000, 19:45	22	30	36	47	54	62	71	83
01 Jan 2000, 19:50	22	30	36	47	54	62	70	83
01 Jan 2000, 19:55	22	29	36	45	53	62	70	82
01 Jan 2000, 20:00	22	29	35	45	52	60	68	79
01 Jan 2000, 20:05	20	27	34	43	50	57	65	76
01 Jan 2000, 20:10	19	26	32	41	47	54	62	72
01 Jan 2000, 20:15	18	25	30	38	45	52	59	68
01 Jan 2000, 20:20	18	24	29	37	43	50	56	66
01 Jan 2000, 20:25	18	23	29	36	42	48	55	65
01 Jan 2000, 20:30	17	23	28	36	41	48	55	63
01 Jan 2000, 20:35	17	22	28	36	41	48	54	63
01 Jan 2000, 20:40	17	22	28	35	41	47	54	62
01 Jan 2000, 20:45	17	22	28	35	40	47	54	62
01 Jan 2000, 20:50	17	22	28	35	40	47	54	62
01 Jan 2000, 20:55	17	22	28	35	40	47	54	62
01 Jan 2000, 21:00	17	22	28	35	40	47	54	62
01 Jan 2000, 21:05	17	22	28	35	40	47	54	62
01 Jan 2000, 21:10	17	22	28	35	40	47	54	62
01 Jan 2000, 21:15	17	22	28	35	40	47	54	62
01 Jan 2000, 21:20	17	22	28	35	40	47	54	62
01 Jan 2000, 21:25	17	22	28	35	40	47	54	62
01 Jan 2000, 21:30	17	22	28	35	40	47	54	62
01 Jan 2000, 21:35	17	22	28	35	40	47	54	62
01 Jan 2000, 21:40	17	22	28	35	40	47	54	62
01 Jan 2000, 21:45	17	22	28	35	40	47	54	62
01 Jan 2000, 21:50	17	22	28	35	40	47	54	62
01 Jan 2000, 21:55	17	22	28	35	40	47	54	62
01 Jan 2000, 22:00	17	22	28	35	40	47	54	62
01 Jan 2000, 22:05	17	22	28	35	40	47	54	62
01 Jan 2000, 22:10	17	22	28	35	40	47	54	62
01 Jan 2000, 22:15	17	22	28	35	40	47	54	62
01 Jan 2000, 22:20	17	22	28	35	40	47	54	62
01 Jan 2000, 22:25	17	22	28	35	40	47	54	62
01 Jan 2000, 22:30	17	22	28	35	40	47	54	62
01 Jan 2000, 22:35	17	22	28	35	40	47	54	62
01 Jan 2000, 22:40	17	22	28	35	40	47	54	62
01 Jan 2000, 22:45	17	22	28	35	40	47	54	62
01 Jan 2000, 22:50	17	22	28	35	40	47	54	62
01 Jan 2000, 22:55	17	22	28	35	40	47	54	62

TCT50-TC64								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	17	22	28	35	40	47	54	62
01 Jan 2000, 23:05	17	22	28	35	40	47	54	62
01 Jan 2000, 23:10	17	22	28	35	40	47	54	62
01 Jan 2000, 23:15	17	22	28	35	40	47	54	62
01 Jan 2000, 23:20	17	22	28	35	40	47	54	62
01 Jan 2000, 23:25	17	22	28	35	40	47	54	62
01 Jan 2000, 23:30	17	22	28	35	40	47	54	62
01 Jan 2000, 23:35	17	22	28	35	40	47	54	62
01 Jan 2000, 23:40	17	22	28	35	40	47	54	62
01 Jan 2000, 23:45	17	22	28	35	40	47	54	62
01 Jan 2000, 23:50	17	22	28	35	40	47	54	62
01 Jan 2000, 23:55	16	22	27	34	40	46	53	61
01 Jan 2000, 24:00	16	22	27	34	39	45	51	59

E.2.8 TC-T40

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	0	0	0	0	0
01 Jan 2000, 00:15	0	0	0	0	0	0	0	0
01 Jan 2000, 00:20	0	0	0	0	0	1	1	1
01 Jan 2000, 00:25	0	1	1	1	1	1	2	2
01 Jan 2000, 00:30	1	1	1	1	2	2	2	3
01 Jan 2000, 00:35	1	1	2	2	2	3	3	4
01 Jan 2000, 00:40	1	2	2	2	3	3	4	4
01 Jan 2000, 00:45	2	2	2	3	3	4	4	5
01 Jan 2000, 00:50	2	2	3	3	4	4	5	5
01 Jan 2000, 00:55	2	2	3	3	4	4	5	6
01 Jan 2000, 01:00	2	2	3	3	4	4	5	6
01 Jan 2000, 01:05	2	2	3	3	4	4	5	6
01 Jan 2000, 01:10	2	2	3	3	4	4	5	6
01 Jan 2000, 01:15	2	2	3	3	4	4	5	6
01 Jan 2000, 01:20	2	2	3	3	4	4	5	6
01 Jan 2000, 01:25	2	2	3	3	4	4	5	6
01 Jan 2000, 01:30	2	2	3	3	4	4	5	6
01 Jan 2000, 01:35	2	2	3	3	4	4	5	6
01 Jan 2000, 01:40	2	2	3	3	4	4	5	6
01 Jan 2000, 01:45	2	2	3	3	4	4	5	6
01 Jan 2000, 01:50	2	2	3	3	4	4	5	6
01 Jan 2000, 01:55	2	2	3	3	4	4	5	6
01 Jan 2000, 02:00	2	2	3	3	4	4	5	6
01 Jan 2000, 02:05	2	2	3	3	4	4	5	6
01 Jan 2000, 02:10	2	2	3	3	4	4	5	6
01 Jan 2000, 02:15	2	2	3	3	4	4	5	6
01 Jan 2000, 02:20	2	2	3	3	4	4	5	6
01 Jan 2000, 02:25	2	2	3	3	4	4	5	6
01 Jan 2000, 02:30	2	2	3	3	4	4	5	6
01 Jan 2000, 02:35	2	2	3	3	4	4	5	6
01 Jan 2000, 02:40	2	2	3	3	4	4	5	6
01 Jan 2000, 02:45	2	2	3	3	4	4	5	6
01 Jan 2000, 02:50	2	2	3	3	4	4	5	6
01 Jan 2000, 02:55	2	2	3	3	4	4	5	6

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	2	2	3	3	4	4	5	6
01 Jan 2000, 03:05	2	2	3	3	4	4	5	6
01 Jan 2000, 03:10	2	2	3	3	4	4	5	6
01 Jan 2000, 03:15	2	2	3	3	4	4	5	6
01 Jan 2000, 03:20	2	2	3	3	4	4	5	6
01 Jan 2000, 03:25	2	2	3	3	4	4	5	6
01 Jan 2000, 03:30	2	2	3	3	4	4	5	6
01 Jan 2000, 03:35	2	2	3	3	4	4	5	6
01 Jan 2000, 03:40	2	2	3	3	4	4	5	6
01 Jan 2000, 03:45	2	2	3	3	4	4	5	6
01 Jan 2000, 03:50	2	2	3	3	4	4	5	6
01 Jan 2000, 03:55	2	2	3	3	4	4	5	6
01 Jan 2000, 04:00	2	2	3	3	4	5	5	6
01 Jan 2000, 04:05	2	2	3	4	4	5	5	6
01 Jan 2000, 04:10	2	3	3	4	4	5	6	7
01 Jan 2000, 04:15	2	3	3	4	5	5	6	7
01 Jan 2000, 04:20	2	3	4	4	5	6	6	7
01 Jan 2000, 04:25	2	3	4	4	5	6	7	7
01 Jan 2000, 04:30	2	3	4	4	5	6	7	8
01 Jan 2000, 04:35	2	3	4	5	5	6	7	8
01 Jan 2000, 04:40	2	3	4	5	5	6	7	8
01 Jan 2000, 04:45	2	3	4	5	5	6	7	8
01 Jan 2000, 04:50	2	3	4	5	5	6	7	8
01 Jan 2000, 04:55	2	3	4	5	5	6	7	8
01 Jan 2000, 05:00	2	3	4	5	5	6	7	8
01 Jan 2000, 05:05	2	3	4	5	5	6	7	8
01 Jan 2000, 05:10	2	3	4	5	5	6	7	8
01 Jan 2000, 05:15	2	3	4	5	5	6	7	8
01 Jan 2000, 05:20	2	3	4	5	5	6	7	8
01 Jan 2000, 05:25	2	3	4	5	5	6	7	8
01 Jan 2000, 05:30	2	3	4	5	5	6	7	8
01 Jan 2000, 05:35	2	3	4	5	5	6	7	8
01 Jan 2000, 05:40	2	3	4	5	5	6	7	8
01 Jan 2000, 05:45	2	3	4	5	5	6	7	9
01 Jan 2000, 05:50	2	3	4	5	5	6	7	9
01 Jan 2000, 05:55	2	3	4	5	5	6	7	9
01 Jan 2000, 06:00	2	3	4	5	5	6	7	9
01 Jan 2000, 06:05	2	3	4	5	5	6	7	9
01 Jan 2000, 06:10	2	3	4	5	5	6	7	9
01 Jan 2000, 06:15	2	3	4	5	5	6	7	10

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	3	3	4	5	5	6	8	10
01 Jan 2000, 06:25	3	3	4	5	6	7	8	11
01 Jan 2000, 06:30	3	4	4	5	6	7	9	12
01 Jan 2000, 06:35	3	4	4	5	6	7	9	13
01 Jan 2000, 06:40	3	4	5	6	6	8	10	13
01 Jan 2000, 06:45	3	4	5	6	6	8	10	13
01 Jan 2000, 06:50	3	4	5	6	6	8	10	14
01 Jan 2000, 06:55	3	4	5	6	6	8	11	14
01 Jan 2000, 07:00	3	4	5	6	7	8	11	14
01 Jan 2000, 07:05	3	4	5	6	7	9	11	14
01 Jan 2000, 07:10	3	4	5	6	7	9	11	15
01 Jan 2000, 07:15	3	4	5	6	7	9	11	15
01 Jan 2000, 07:20	3	4	5	6	7	9	12	15
01 Jan 2000, 07:25	3	4	5	6	7	9	12	15
01 Jan 2000, 07:30	3	4	5	6	7	9	12	16
01 Jan 2000, 07:35	3	4	5	6	7	10	12	16
01 Jan 2000, 07:40	3	4	5	6	7	10	12	16
01 Jan 2000, 07:45	3	4	5	6	8	10	12	16
01 Jan 2000, 07:50	3	4	5	6	8	10	13	16
01 Jan 2000, 07:55	3	4	5	6	8	10	13	17
01 Jan 2000, 08:00	3	4	5	6	8	10	13	17
01 Jan 2000, 08:05	3	4	5	6	8	11	13	17
01 Jan 2000, 08:10	3	4	5	6	8	11	13	17
01 Jan 2000, 08:15	3	4	5	6	8	11	14	18
01 Jan 2000, 08:20	3	4	5	7	9	12	14	19
01 Jan 2000, 08:25	3	4	5	7	9	12	15	20
01 Jan 2000, 08:30	4	4	5	8	10	13	16	21
01 Jan 2000, 08:35	4	5	6	8	11	14	17	22
01 Jan 2000, 08:40	4	5	6	9	11	15	19	24
01 Jan 2000, 08:45	4	5	6	9	12	16	19	25
01 Jan 2000, 08:50	4	5	6	10	13	16	20	26
01 Jan 2000, 08:55	4	5	7	10	13	17	21	26
01 Jan 2000, 09:00	4	5	7	10	13	17	21	27
01 Jan 2000, 09:05	4	5	7	10	14	17	21	27
01 Jan 2000, 09:10	4	5	7	11	14	18	22	28
01 Jan 2000, 09:15	4	6	7	11	14	18	22	28
01 Jan 2000, 09:20	4	6	7	11	15	19	23	29
01 Jan 2000, 09:25	5	6	8	12	15	20	24	31
01 Jan 2000, 09:30	5	6	8	13	16	21	25	32
01 Jan 2000, 09:35	5	6	9	13	17	21	26	33

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	5	6	9	13	17	22	27	34
01 Jan 2000, 09:45	5	6	9	14	17	22	27	34
01 Jan 2000, 09:50	5	7	10	14	18	23	28	36
01 Jan 2000, 09:55	5	7	10	15	19	24	30	37
01 Jan 2000, 10:00	5	7	10	16	20	25	31	39
01 Jan 2000, 10:05	5	8	11	16	20	26	31	39
01 Jan 2000, 10:10	5	8	11	16	21	26	32	40
01 Jan 2000, 10:15	6	8	11	17	21	27	33	41
01 Jan 2000, 10:20	6	8	12	17	22	28	34	42
01 Jan 2000, 10:25	6	9	12	18	23	29	35	44
01 Jan 2000, 10:30	6	9	13	19	24	30	37	46
01 Jan 2000, 10:35	6	10	14	20	26	33	39	49
01 Jan 2000, 10:40	7	11	15	22	27	35	42	52
01 Jan 2000, 10:45	7	12	16	23	30	37	45	56
01 Jan 2000, 10:50	8	13	17	25	32	41	49	61
01 Jan 2000, 10:55	8	14	19	28	35	44	53	66
01 Jan 2000, 11:00	9	15	21	30	38	48	57	71
01 Jan 2000, 11:05	10	17	23	33	41	52	62	77
01 Jan 2000, 11:10	11	18	25	36	45	56	67	83
01 Jan 2000, 11:15	12	20	27	39	48	60	72	89
01 Jan 2000, 11:20	13	21	29	42	52	65	78	96
01 Jan 2000, 11:25	14	23	32	45	56	69	83	103
01 Jan 2000, 11:30	16	25	34	49	60	74	89	110
01 Jan 2000, 11:35	17	27	37	53	66	81	97	120
01 Jan 2000, 11:40	19	31	42	60	74	89	108	134
01 Jan 2000, 11:45	25	40	54	78	98	123	150	191
01 Jan 2000, 11:50	39	64	89	130	164	214	263	335
01 Jan 2000, 11:55	67	108	150	219	277	346	425	539
01 Jan 2000, 12:00	109	175	244	357	453	578	707	890
01 Jan 2000, 12:05	181	293	403	578	719	893	1063	1286
01 Jan 2000, 12:10	267	407	535	723	863	1022	1178	1393
01 Jan 2000, 12:15	294	415	520	667	775	901	1029	1205
01 Jan 2000, 12:20	250	340	417	525	605	693	779	897
01 Jan 2000, 12:25	188	254	312	392	451	521	588	674
01 Jan 2000, 12:30	139	191	234	296	341	390	440	508
01 Jan 2000, 12:35	106	145	179	227	263	308	348	403
01 Jan 2000, 12:40	83	116	144	184	214	245	278	322
01 Jan 2000, 12:45	69	94	117	149	173	203	231	268
01 Jan 2000, 12:50	57	80	100	128	149	174	198	230
01 Jan 2000, 12:55	51	70	87	112	131	153	174	203

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	45	63	79	101	118	137	157	183
01 Jan 2000, 13:05	41	57	71	91	107	124	142	165
01 Jan 2000, 13:10	37	52	65	83	97	113	128	150
01 Jan 2000, 13:15	34	48	59	76	89	104	118	138
01 Jan 2000, 13:20	32	44	55	71	82	96	110	128
01 Jan 2000, 13:25	30	41	51	66	77	90	103	120
01 Jan 2000, 13:30	28	39	49	62	73	85	97	113
01 Jan 2000, 13:35	27	37	46	59	69	80	92	107
01 Jan 2000, 13:40	25	35	44	56	65	76	87	102
01 Jan 2000, 13:45	24	33	42	53	62	73	83	97
01 Jan 2000, 13:50	23	32	40	51	59	69	79	92
01 Jan 2000, 13:55	22	30	37	48	56	65	75	87
01 Jan 2000, 14:00	21	29	36	46	53	62	71	83
01 Jan 2000, 14:05	20	28	34	44	51	60	68	80
01 Jan 2000, 14:10	19	27	33	42	49	57	66	77
01 Jan 2000, 14:15	19	26	32	41	48	55	63	74
01 Jan 2000, 14:20	18	25	31	39	46	54	61	72
01 Jan 2000, 14:25	17	24	30	38	44	52	59	69
01 Jan 2000, 14:30	17	23	29	37	43	50	57	67
01 Jan 2000, 14:35	16	22	28	35	41	48	55	64
01 Jan 2000, 14:40	16	22	27	34	40	46	53	62
01 Jan 2000, 14:45	15	21	26	33	38	45	51	60
01 Jan 2000, 14:50	15	20	25	32	37	43	49	57
01 Jan 2000, 14:55	14	19	24	30	35	41	47	55
01 Jan 2000, 15:00	13	18	23	29	34	40	45	53
01 Jan 2000, 15:05	13	18	22	29	33	39	44	52
01 Jan 2000, 15:10	13	18	22	28	33	38	44	51
01 Jan 2000, 15:15	13	18	22	28	33	38	44	51
01 Jan 2000, 15:20	13	18	22	28	33	38	43	51
01 Jan 2000, 15:25	13	18	22	28	32	38	43	51
01 Jan 2000, 15:30	13	17	22	28	32	38	43	50
01 Jan 2000, 15:35	12	17	21	27	32	36	42	49
01 Jan 2000, 15:40	12	16	20	26	30	35	40	47
01 Jan 2000, 15:45	12	16	20	25	29	34	39	45
01 Jan 2000, 15:50	11	16	19	25	29	33	38	45
01 Jan 2000, 15:55	11	15	19	24	28	33	38	44
01 Jan 2000, 16:00	11	15	19	24	28	33	37	44
01 Jan 2000, 16:05	11	15	19	24	28	33	37	44
01 Jan 2000, 16:10	11	15	19	24	28	33	37	44
01 Jan 2000, 16:15	11	15	19	24	28	33	37	43

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	11	15	19	24	28	33	37	43
01 Jan 2000, 16:25	11	15	19	24	28	33	37	43
01 Jan 2000, 16:30	11	15	19	24	28	33	37	43
01 Jan 2000, 16:35	11	15	19	24	28	33	37	43
01 Jan 2000, 16:40	11	15	19	24	28	33	37	43
01 Jan 2000, 16:45	11	15	19	24	28	32	37	43
01 Jan 2000, 16:50	11	15	18	23	27	31	36	42
01 Jan 2000, 16:55	10	14	17	22	26	30	34	40
01 Jan 2000, 17:00	10	14	17	21	25	29	33	38
01 Jan 2000, 17:05	10	13	16	21	24	28	32	37
01 Jan 2000, 17:10	9	13	16	20	24	27	32	37
01 Jan 2000, 17:15	9	13	16	20	24	27	31	37
01 Jan 2000, 17:20	9	13	16	20	24	27	31	36
01 Jan 2000, 17:25	9	13	16	20	23	27	31	36
01 Jan 2000, 17:30	9	13	16	20	23	27	31	36
01 Jan 2000, 17:35	9	13	16	20	23	27	31	36
01 Jan 2000, 17:40	9	13	16	20	23	27	31	36
01 Jan 2000, 17:45	9	13	16	20	23	27	31	36
01 Jan 2000, 17:50	9	13	16	20	23	27	31	36
01 Jan 2000, 17:55	9	13	16	20	23	27	31	36
01 Jan 2000, 18:00	9	13	16	20	23	27	31	36
01 Jan 2000, 18:05	9	12	15	19	23	26	30	34
01 Jan 2000, 18:10	9	12	14	18	21	25	28	33
01 Jan 2000, 18:15	8	11	14	17	20	23	27	31
01 Jan 2000, 18:20	8	11	13	17	20	23	26	30
01 Jan 2000, 18:25	8	10	13	17	19	22	25	30
01 Jan 2000, 18:30	8	10	13	16	19	22	25	29
01 Jan 2000, 18:35	8	10	13	16	19	22	25	29
01 Jan 2000, 18:40	8	10	13	16	19	22	25	29
01 Jan 2000, 18:45	8	10	13	16	19	22	25	29
01 Jan 2000, 18:50	7	10	13	16	19	22	25	29
01 Jan 2000, 18:55	7	10	13	16	19	22	25	29
01 Jan 2000, 19:00	7	10	13	16	19	22	25	29
01 Jan 2000, 19:05	7	10	13	16	19	22	25	29
01 Jan 2000, 19:10	8	10	13	16	19	22	25	29
01 Jan 2000, 19:15	8	10	13	16	19	22	25	29
01 Jan 2000, 19:20	8	10	13	16	19	22	25	29
01 Jan 2000, 19:25	8	10	13	16	19	22	25	29
01 Jan 2000, 19:30	8	10	13	16	19	22	25	29
01 Jan 2000, 19:35	8	10	13	16	19	22	25	29

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	8	10	13	16	19	22	25	29
01 Jan 2000, 19:45	8	10	13	16	19	22	25	29
01 Jan 2000, 19:50	8	10	13	16	19	22	25	29
01 Jan 2000, 19:55	8	10	13	16	19	22	25	29
01 Jan 2000, 20:00	7	10	13	16	19	22	25	29
01 Jan 2000, 20:05	7	10	12	15	18	21	23	27
01 Jan 2000, 20:10	7	9	11	14	17	19	22	25
01 Jan 2000, 20:15	6	9	11	14	16	18	21	24
01 Jan 2000, 20:20	6	8	10	13	15	17	20	23
01 Jan 2000, 20:25	6	8	10	13	15	17	19	23
01 Jan 2000, 20:30	6	8	10	12	14	17	19	22
01 Jan 2000, 20:35	6	8	10	12	14	17	19	22
01 Jan 2000, 20:40	6	8	10	12	14	17	19	22
01 Jan 2000, 20:45	6	8	10	12	14	17	19	22
01 Jan 2000, 20:50	6	8	10	12	14	16	19	22
01 Jan 2000, 20:55	6	8	10	12	14	16	19	22
01 Jan 2000, 21:00	6	8	10	12	14	16	19	22
01 Jan 2000, 21:05	6	8	10	12	14	16	19	22
01 Jan 2000, 21:10	6	8	10	12	14	16	19	22
01 Jan 2000, 21:15	6	8	10	12	14	16	19	22
01 Jan 2000, 21:20	6	8	10	12	14	16	19	22
01 Jan 2000, 21:25	6	8	10	12	14	16	19	22
01 Jan 2000, 21:30	6	8	10	12	14	16	19	22
01 Jan 2000, 21:35	6	8	10	12	14	16	19	22
01 Jan 2000, 21:40	6	8	10	12	14	16	19	22
01 Jan 2000, 21:45	6	8	10	12	14	16	19	22
01 Jan 2000, 21:50	6	8	10	12	14	16	19	22
01 Jan 2000, 21:55	6	8	10	12	14	16	19	22
01 Jan 2000, 22:00	6	8	10	12	14	16	19	22
01 Jan 2000, 22:05	6	8	10	12	14	16	19	22
01 Jan 2000, 22:10	6	8	10	12	14	16	19	22
01 Jan 2000, 22:15	6	8	10	12	14	16	19	22
01 Jan 2000, 22:20	6	8	10	12	14	16	19	22
01 Jan 2000, 22:25	6	8	10	12	14	16	19	22
01 Jan 2000, 22:30	6	8	10	12	14	16	19	22
01 Jan 2000, 22:35	6	8	10	12	14	16	19	22
01 Jan 2000, 22:40	6	8	10	12	14	17	19	22
01 Jan 2000, 22:45	6	8	10	12	14	17	19	22
01 Jan 2000, 22:50	6	8	10	12	14	17	19	22
01 Jan 2000, 22:55	6	8	10	12	14	17	19	22

TC-T40								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	6	8	10	12	14	17	19	22
01 Jan 2000, 23:05	6	8	10	12	14	17	19	22
01 Jan 2000, 23:10	6	8	10	12	14	17	19	22
01 Jan 2000, 23:15	6	8	10	12	14	17	19	22
01 Jan 2000, 23:20	6	8	10	12	14	17	19	22
01 Jan 2000, 23:25	6	8	10	12	14	17	19	22
01 Jan 2000, 23:30	6	8	10	12	14	17	19	22
01 Jan 2000, 23:35	6	8	10	12	14	17	19	22
01 Jan 2000, 23:40	6	8	10	12	14	17	19	22
01 Jan 2000, 23:45	6	8	10	12	14	17	19	22
01 Jan 2000, 23:50	6	8	10	12	14	17	19	22
01 Jan 2000, 23:55	6	8	10	12	14	17	19	22
01 Jan 2000, 24:00	6	8	10	12	14	16	18	22

E.2.9 TC57

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	1	1	1	1	1
01 Jan 2000, 00:15	1	1	1	1	1	2	2	2
01 Jan 2000, 00:20	1	1	1	2	2	2	3	3
01 Jan 2000, 00:25	1	2	2	2	3	3	3	4
01 Jan 2000, 00:30	2	2	2	3	3	4	4	5
01 Jan 2000, 00:35	2	2	3	3	4	4	5	6
01 Jan 2000, 00:40	2	2	3	4	4	5	5	6
01 Jan 2000, 00:45	2	3	3	4	4	5	5	6
01 Jan 2000, 00:50	2	3	3	4	4	5	6	6
01 Jan 2000, 00:55	2	3	3	4	4	5	6	7
01 Jan 2000, 01:00	2	3	3	4	4	5	6	7
01 Jan 2000, 01:05	2	3	3	4	4	5	6	7
01 Jan 2000, 01:10	2	3	3	4	4	5	6	7
01 Jan 2000, 01:15	2	3	3	4	4	5	6	7
01 Jan 2000, 01:20	2	3	3	4	5	5	6	7
01 Jan 2000, 01:25	2	3	3	4	5	5	6	7
01 Jan 2000, 01:30	2	3	3	4	5	5	6	7
01 Jan 2000, 01:35	2	3	3	4	5	5	6	7
01 Jan 2000, 01:40	2	3	3	4	5	5	6	7
01 Jan 2000, 01:45	2	3	3	4	5	5	6	7
01 Jan 2000, 01:50	2	3	3	4	5	5	6	7
01 Jan 2000, 01:55	2	3	3	4	5	5	6	7
01 Jan 2000, 02:00	2	3	3	4	5	5	6	7
01 Jan 2000, 02:05	2	3	3	4	5	5	6	7
01 Jan 2000, 02:10	2	3	3	4	5	5	6	7
01 Jan 2000, 02:15	2	3	3	4	5	5	6	7
01 Jan 2000, 02:20	2	3	3	4	5	5	6	7
01 Jan 2000, 02:25	2	3	3	4	5	5	6	7
01 Jan 2000, 02:30	2	3	3	4	5	5	6	7
01 Jan 2000, 02:35	2	3	3	4	5	5	6	7
01 Jan 2000, 02:40	2	3	3	4	5	5	6	7
01 Jan 2000, 02:45	2	3	3	4	5	5	6	7
01 Jan 2000, 02:50	2	3	3	4	5	5	6	7
01 Jan 2000, 02:55	2	3	3	4	5	5	6	7

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	2	3	3	4	5	5	6	7
01 Jan 2000, 03:05	2	3	3	4	5	5	6	7
01 Jan 2000, 03:10	2	3	3	4	5	5	6	7
01 Jan 2000, 03:15	2	3	3	4	5	5	6	7
01 Jan 2000, 03:20	2	3	3	4	5	5	6	7
01 Jan 2000, 03:25	2	3	3	4	5	5	6	7
01 Jan 2000, 03:30	2	3	3	4	5	5	6	7
01 Jan 2000, 03:35	2	3	3	4	5	5	6	7
01 Jan 2000, 03:40	2	3	3	4	5	5	6	7
01 Jan 2000, 03:45	2	3	3	4	5	5	6	7
01 Jan 2000, 03:50	2	3	3	4	5	5	6	7
01 Jan 2000, 03:55	2	3	3	4	5	6	6	7
01 Jan 2000, 04:00	2	3	4	5	5	6	7	8
01 Jan 2000, 04:05	3	3	4	5	5	6	7	8
01 Jan 2000, 04:10	3	3	4	5	6	7	7	8
01 Jan 2000, 04:15	3	4	4	5	6	7	7	9
01 Jan 2000, 04:20	3	4	4	5	6	7	8	9
01 Jan 2000, 04:25	3	4	4	5	6	7	8	9
01 Jan 2000, 04:30	3	4	4	5	6	7	8	9
01 Jan 2000, 04:35	3	4	4	5	6	7	8	9
01 Jan 2000, 04:40	3	4	4	5	6	7	8	9
01 Jan 2000, 04:45	3	4	4	5	6	7	8	10
01 Jan 2000, 04:50	3	4	4	5	6	7	8	10
01 Jan 2000, 04:55	3	4	4	5	6	7	8	10
01 Jan 2000, 05:00	3	4	4	5	6	7	8	10
01 Jan 2000, 05:05	3	4	4	5	6	7	8	10
01 Jan 2000, 05:10	3	4	4	5	6	7	8	10
01 Jan 2000, 05:15	3	4	4	5	6	7	8	11
01 Jan 2000, 05:20	3	4	4	5	6	7	8	11
01 Jan 2000, 05:25	3	4	4	5	6	7	9	11
01 Jan 2000, 05:30	3	4	4	5	6	7	9	11
01 Jan 2000, 05:35	3	4	4	5	6	7	9	11
01 Jan 2000, 05:40	3	4	4	5	6	7	9	11
01 Jan 2000, 05:45	3	4	4	5	6	7	9	12
01 Jan 2000, 05:50	3	4	4	5	6	7	9	12
01 Jan 2000, 05:55	3	4	4	5	6	8	9	12
01 Jan 2000, 06:00	3	4	4	5	6	8	9	12
01 Jan 2000, 06:05	3	4	4	5	6	8	10	12
01 Jan 2000, 06:10	3	4	5	6	7	8	10	13
01 Jan 2000, 06:15	3	4	5	6	7	9	11	14

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	3	4	5	6	7	9	12	15
01 Jan 2000, 06:25	3	4	5	6	8	10	12	15
01 Jan 2000, 06:30	3	4	5	6	8	10	12	16
01 Jan 2000, 06:35	4	4	5	7	8	10	13	16
01 Jan 2000, 06:40	4	5	5	7	8	11	13	16
01 Jan 2000, 06:45	4	5	5	7	9	11	13	17
01 Jan 2000, 06:50	4	5	5	7	9	11	13	17
01 Jan 2000, 06:55	4	5	5	7	9	11	14	17
01 Jan 2000, 07:00	4	5	5	7	9	11	14	17
01 Jan 2000, 07:05	4	5	5	7	9	11	14	17
01 Jan 2000, 07:10	4	5	5	7	9	12	14	18
01 Jan 2000, 07:15	4	5	5	7	9	12	14	18
01 Jan 2000, 07:20	4	5	5	8	9	12	14	18
01 Jan 2000, 07:25	4	5	5	8	10	12	15	18
01 Jan 2000, 07:30	4	5	5	8	10	12	15	18
01 Jan 2000, 07:35	4	5	5	8	10	12	15	19
01 Jan 2000, 07:40	4	5	6	8	10	12	15	19
01 Jan 2000, 07:45	4	5	6	8	10	12	15	19
01 Jan 2000, 07:50	4	5	6	8	10	13	15	19
01 Jan 2000, 07:55	4	5	6	8	10	13	15	19
01 Jan 2000, 08:00	4	5	6	8	10	13	16	19
01 Jan 2000, 08:05	4	5	6	8	11	13	16	20
01 Jan 2000, 08:10	4	5	6	9	11	14	16	20
01 Jan 2000, 08:15	4	5	7	9	12	14	17	22
01 Jan 2000, 08:20	4	5	7	10	12	15	18	23
01 Jan 2000, 08:25	4	6	7	10	13	16	19	24
01 Jan 2000, 08:30	5	6	8	11	14	17	21	26
01 Jan 2000, 08:35	5	6	8	12	15	18	22	27
01 Jan 2000, 08:40	5	6	9	12	15	19	22	28
01 Jan 2000, 08:45	5	7	9	13	15	19	23	29
01 Jan 2000, 08:50	5	7	9	13	16	20	23	29
01 Jan 2000, 08:55	5	7	9	13	16	20	24	29
01 Jan 2000, 09:00	5	7	9	13	16	20	24	30
01 Jan 2000, 09:05	5	7	10	13	17	20	24	30
01 Jan 2000, 09:10	5	7	10	14	17	21	25	31
01 Jan 2000, 09:15	5	8	10	15	18	22	26	32
01 Jan 2000, 09:20	5	8	11	15	19	23	27	34
01 Jan 2000, 09:25	6	8	11	16	19	24	28	35
01 Jan 2000, 09:30	6	9	11	16	20	24	29	35
01 Jan 2000, 09:35	6	9	12	16	20	25	29	36

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	6	9	12	17	21	25	30	37
01 Jan 2000, 09:45	6	9	13	17	21	26	31	38
01 Jan 2000, 09:50	6	10	13	18	22	27	32	40
01 Jan 2000, 09:55	7	10	13	19	23	28	33	41
01 Jan 2000, 10:00	7	10	14	19	23	28	34	41
01 Jan 2000, 10:05	7	11	14	19	24	29	35	42
01 Jan 2000, 10:10	7	11	14	20	24	30	36	43
01 Jan 2000, 10:15	8	11	15	21	25	31	37	45
01 Jan 2000, 10:20	8	12	16	22	26	32	38	47
01 Jan 2000, 10:25	8	13	17	23	28	34	40	49
01 Jan 2000, 10:30	9	13	18	24	30	36	43	52
01 Jan 2000, 10:35	10	14	19	26	32	39	46	56
01 Jan 2000, 10:40	10	16	21	28	34	42	50	60
01 Jan 2000, 10:45	11	17	22	31	37	45	54	65
01 Jan 2000, 10:50	12	18	24	33	40	49	58	70
01 Jan 2000, 10:55	13	20	26	36	44	53	62	75
01 Jan 2000, 11:00	15	22	28	39	47	57	67	81
01 Jan 2000, 11:05	16	23	31	42	50	61	72	87
01 Jan 2000, 11:10	17	25	33	45	54	65	77	93
01 Jan 2000, 11:15	18	27	35	48	58	70	82	99
01 Jan 2000, 11:20	20	29	38	51	62	74	87	105
01 Jan 2000, 11:25	21	31	41	55	67	80	94	113
01 Jan 2000, 11:30	23	34	44	60	72	87	101	122
01 Jan 2000, 11:35	29	43	55	74	89	107	125	150
01 Jan 2000, 11:40	45	66	85	114	136	163	190	227
01 Jan 2000, 11:45	68	99	127	169	202	241	280	335
01 Jan 2000, 11:50	102	146	187	248	295	350	407	484
01 Jan 2000, 11:55	160	227	288	378	448	530	613	726
01 Jan 2000, 12:00	234	329	414	540	636	750	864	1020
01 Jan 2000, 12:05	287	400	501	650	763	897	1013	1213
01 Jan 2000, 12:10	280	389	485	627	735	862	989	1162
01 Jan 2000, 12:15	228	315	393	507	593	694	796	934
01 Jan 2000, 12:20	170	235	292	376	440	514	589	690
01 Jan 2000, 12:25	126	173	215	276	322	376	430	504
01 Jan 2000, 12:30	97	133	165	211	246	288	329	385
01 Jan 2000, 12:35	77	105	131	167	194	227	259	303
01 Jan 2000, 12:40	64	87	107	137	159	186	212	248
01 Jan 2000, 12:45	55	74	92	117	136	158	181	211
01 Jan 2000, 12:50	48	65	81	103	119	139	158	185
01 Jan 2000, 12:55	43	58	72	91	106	123	141	164

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	39	53	65	82	96	111	126	147
01 Jan 2000, 13:05	35	48	59	75	87	101	115	134
01 Jan 2000, 13:10	32	44	54	68	79	92	105	122
01 Jan 2000, 13:15	30	41	50	63	73	85	97	113
01 Jan 2000, 13:20	28	38	47	59	69	80	91	106
01 Jan 2000, 13:25	27	36	44	56	65	75	86	100
01 Jan 2000, 13:30	25	34	42	53	61	71	81	94
01 Jan 2000, 13:35	24	32	40	50	58	68	77	90
01 Jan 2000, 13:40	23	31	38	48	55	64	73	85
01 Jan 2000, 13:45	22	29	36	45	52	61	69	81
01 Jan 2000, 13:50	21	28	34	43	50	58	66	77
01 Jan 2000, 13:55	20	27	33	41	48	55	63	73
01 Jan 2000, 14:00	19	26	31	40	46	53	60	70
01 Jan 2000, 14:05	18	25	30	38	44	51	58	68
01 Jan 2000, 14:10	18	24	29	37	43	49	56	65
01 Jan 2000, 14:15	17	23	28	35	41	48	54	63
01 Jan 2000, 14:20	16	22	27	34	40	46	52	61
01 Jan 2000, 14:25	16	21	26	33	38	44	50	59
01 Jan 2000, 14:30	15	21	25	32	37	43	49	56
01 Jan 2000, 14:35	15	20	24	31	35	41	47	54
01 Jan 2000, 14:40	14	19	23	29	34	39	45	52
01 Jan 2000, 14:45	14	18	22	28	33	38	43	50
01 Jan 2000, 14:50	13	18	22	27	31	36	41	48
01 Jan 2000, 14:55	13	17	21	27	31	36	40	47
01 Jan 2000, 15:00	13	17	21	26	30	35	40	46
01 Jan 2000, 15:05	13	17	21	26	30	35	40	46
01 Jan 2000, 15:10	12	17	20	26	30	35	39	46
01 Jan 2000, 15:15	12	17	20	26	30	34	39	46
01 Jan 2000, 15:20	12	17	20	26	30	34	39	45
01 Jan 2000, 15:25	12	16	20	25	29	33	38	44
01 Jan 2000, 15:30	12	16	19	24	28	32	37	43
01 Jan 2000, 15:35	11	15	18	23	27	31	35	41
01 Jan 2000, 15:40	11	15	18	23	26	30	35	40
01 Jan 2000, 15:45	11	15	18	22	26	30	34	40
01 Jan 2000, 15:50	11	14	18	22	26	30	34	39
01 Jan 2000, 15:55	11	14	18	22	26	30	34	39
01 Jan 2000, 16:00	11	14	18	22	26	30	34	39
01 Jan 2000, 16:05	11	14	18	22	26	30	34	39
01 Jan 2000, 16:10	11	14	17	22	26	30	34	39
01 Jan 2000, 16:15	11	14	17	22	26	30	34	39

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	11	14	17	22	26	30	34	39
01 Jan 2000, 16:25	11	14	17	22	26	30	34	39
01 Jan 2000, 16:30	11	14	17	22	26	30	34	39
01 Jan 2000, 16:35	11	14	17	22	25	29	33	39
01 Jan 2000, 16:40	10	14	17	21	25	29	32	38
01 Jan 2000, 16:45	10	13	16	20	24	27	31	36
01 Jan 2000, 16:50	10	13	16	20	23	26	30	35
01 Jan 2000, 16:55	9	12	15	19	22	26	29	34
01 Jan 2000, 17:00	9	12	15	19	22	25	29	33
01 Jan 2000, 17:05	9	12	15	19	22	25	28	33
01 Jan 2000, 17:10	9	12	15	19	21	25	28	33
01 Jan 2000, 17:15	9	12	15	19	21	25	28	33
01 Jan 2000, 17:20	9	12	15	18	21	25	28	33
01 Jan 2000, 17:25	9	12	15	18	21	25	28	33
01 Jan 2000, 17:30	9	12	15	18	21	25	28	33
01 Jan 2000, 17:35	9	12	15	18	21	25	28	33
01 Jan 2000, 17:40	9	12	15	18	21	25	28	33
01 Jan 2000, 17:45	9	12	15	18	21	25	28	33
01 Jan 2000, 17:50	9	12	15	18	21	24	28	32
01 Jan 2000, 17:55	9	12	14	18	20	24	27	31
01 Jan 2000, 18:00	8	11	13	17	19	23	26	30
01 Jan 2000, 18:05	8	10	13	16	19	21	24	28
01 Jan 2000, 18:10	8	10	12	16	18	21	24	27
01 Jan 2000, 18:15	7	10	12	15	18	20	23	27
01 Jan 2000, 18:20	7	10	12	15	17	20	23	27
01 Jan 2000, 18:25	7	10	12	15	17	20	23	26
01 Jan 2000, 18:30	7	10	12	15	17	20	23	26
01 Jan 2000, 18:35	7	10	12	15	17	20	23	26
01 Jan 2000, 18:40	7	10	12	15	17	20	22	26
01 Jan 2000, 18:45	7	10	12	15	17	20	22	26
01 Jan 2000, 18:50	7	10	12	15	17	20	22	26
01 Jan 2000, 18:55	7	10	12	15	17	20	22	26
01 Jan 2000, 19:00	7	10	12	15	17	20	22	26
01 Jan 2000, 19:05	7	10	12	15	17	20	22	26
01 Jan 2000, 19:10	7	10	12	15	17	20	22	26
01 Jan 2000, 19:15	7	10	12	15	17	20	22	26
01 Jan 2000, 19:20	7	10	12	15	17	20	22	26
01 Jan 2000, 19:25	7	10	12	15	17	20	22	26
01 Jan 2000, 19:30	7	10	12	15	17	20	22	26
01 Jan 2000, 19:35	7	10	12	15	17	20	22	26

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	7	10	12	15	17	20	22	26
01 Jan 2000, 19:45	7	10	12	15	17	20	22	26
01 Jan 2000, 19:50	7	10	12	15	17	20	22	26
01 Jan 2000, 19:55	7	9	11	14	16	19	21	25
01 Jan 2000, 20:00	6	9	10	13	15	18	20	23
01 Jan 2000, 20:05	6	8	10	12	14	17	19	22
01 Jan 2000, 20:10	6	8	9	12	14	16	18	21
01 Jan 2000, 20:15	6	8	9	12	13	15	18	20
01 Jan 2000, 20:20	6	7	9	11	13	15	17	20
01 Jan 2000, 20:25	6	7	9	11	13	15	17	20
01 Jan 2000, 20:30	5	7	9	11	13	15	17	20
01 Jan 2000, 20:35	5	7	9	11	13	15	17	20
01 Jan 2000, 20:40	5	7	9	11	13	15	17	20
01 Jan 2000, 20:45	5	7	9	11	13	15	17	20
01 Jan 2000, 20:50	5	7	9	11	13	15	17	20
01 Jan 2000, 20:55	5	7	9	11	13	15	17	20
01 Jan 2000, 21:00	5	7	9	11	13	15	17	20
01 Jan 2000, 21:05	5	7	9	11	13	15	17	20
01 Jan 2000, 21:10	5	7	9	11	13	15	17	20
01 Jan 2000, 21:15	5	7	9	11	13	15	17	20
01 Jan 2000, 21:20	5	7	9	11	13	15	17	20
01 Jan 2000, 21:25	5	7	9	11	13	15	17	20
01 Jan 2000, 21:30	5	7	9	11	13	15	17	20
01 Jan 2000, 21:35	5	7	9	11	13	15	17	20
01 Jan 2000, 21:40	5	7	9	11	13	15	17	20
01 Jan 2000, 21:45	5	7	9	11	13	15	17	20
01 Jan 2000, 21:50	5	7	9	11	13	15	17	20
01 Jan 2000, 21:55	5	7	9	11	13	15	17	20
01 Jan 2000, 22:00	5	7	9	11	13	15	17	20
01 Jan 2000, 22:05	5	7	9	11	13	15	17	20
01 Jan 2000, 22:10	5	7	9	11	13	15	17	20
01 Jan 2000, 22:15	5	7	9	11	13	15	17	20
01 Jan 2000, 22:20	5	7	9	11	13	15	17	20
01 Jan 2000, 22:25	5	7	9	11	13	15	17	20
01 Jan 2000, 22:30	5	7	9	11	13	15	17	20
01 Jan 2000, 22:35	5	7	9	11	13	15	17	20
01 Jan 2000, 22:40	5	7	9	11	13	15	17	20
01 Jan 2000, 22:45	5	7	9	11	13	15	17	20
01 Jan 2000, 22:50	5	7	9	11	13	15	17	20
01 Jan 2000, 22:55	5	7	9	11	13	15	17	20

TC57								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	5	7	9	11	13	15	17	20
01 Jan 2000, 23:05	5	7	9	11	13	15	17	20
01 Jan 2000, 23:10	5	7	9	11	13	15	17	20
01 Jan 2000, 23:15	5	7	9	11	13	15	17	20
01 Jan 2000, 23:20	5	7	9	11	13	15	17	20
01 Jan 2000, 23:25	5	7	9	11	13	15	17	20
01 Jan 2000, 23:30	5	7	9	11	13	15	17	20
01 Jan 2000, 23:35	5	7	9	11	13	15	17	20
01 Jan 2000, 23:40	5	7	9	11	13	15	17	20
01 Jan 2000, 23:45	5	7	9	11	13	15	17	20
01 Jan 2000, 23:50	5	7	9	11	13	15	17	19
01 Jan 2000, 23:55	5	7	8	10	12	14	16	18
01 Jan 2000, 24:00	5	6	8	10	11	13	14	17

E.2.10 TC41

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	0	0	0	1	1
01 Jan 2000, 00:15	0	1	1	1	1	1	1	1
01 Jan 2000, 00:20	1	1	1	1	1	2	2	2
01 Jan 2000, 00:25	1	1	1	2	2	2	2	3
01 Jan 2000, 00:30	1	1	2	2	2	3	3	3
01 Jan 2000, 00:35	1	2	2	2	3	3	3	4
01 Jan 2000, 00:40	1	2	2	2	3	3	4	4
01 Jan 2000, 00:45	1	2	2	3	3	3	4	4
01 Jan 2000, 00:50	1	2	2	3	3	3	4	4
01 Jan 2000, 00:55	1	2	2	3	3	4	4	5
01 Jan 2000, 01:00	1	2	2	3	3	4	4	5
01 Jan 2000, 01:05	1	2	2	3	3	4	4	5
01 Jan 2000, 01:10	1	2	2	3	3	4	4	5
01 Jan 2000, 01:15	1	2	2	3	3	4	4	5
01 Jan 2000, 01:20	1	2	2	3	3	4	4	5
01 Jan 2000, 01:25	1	2	2	3	3	4	4	5
01 Jan 2000, 01:30	1	2	2	3	3	4	4	5
01 Jan 2000, 01:35	1	2	2	3	3	4	4	5
01 Jan 2000, 01:40	1	2	2	3	3	4	4	5
01 Jan 2000, 01:45	1	2	2	3	3	4	4	5
01 Jan 2000, 01:50	1	2	2	3	3	4	4	5
01 Jan 2000, 01:55	1	2	2	3	3	4	4	5
01 Jan 2000, 02:00	1	2	2	3	3	4	4	5
01 Jan 2000, 02:05	1	2	2	3	3	4	4	5
01 Jan 2000, 02:10	1	2	2	3	3	4	4	5
01 Jan 2000, 02:15	1	2	2	3	3	4	4	5
01 Jan 2000, 02:20	1	2	2	3	3	4	4	5
01 Jan 2000, 02:25	1	2	2	3	3	4	4	5
01 Jan 2000, 02:30	1	2	2	3	3	4	4	5
01 Jan 2000, 02:35	1	2	2	3	3	4	4	5
01 Jan 2000, 02:40	1	2	2	3	3	4	4	5
01 Jan 2000, 02:45	1	2	2	3	3	4	4	5
01 Jan 2000, 02:50	1	2	2	3	3	4	4	5
01 Jan 2000, 02:55	1	2	2	3	3	4	4	5

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	1	2	2	3	3	4	4	5
01 Jan 2000, 03:05	1	2	2	3	3	4	4	5
01 Jan 2000, 03:10	1	2	2	3	3	4	4	5
01 Jan 2000, 03:15	1	2	2	3	3	4	4	5
01 Jan 2000, 03:20	1	2	2	3	3	4	4	5
01 Jan 2000, 03:25	1	2	2	3	3	4	4	5
01 Jan 2000, 03:30	1	2	2	3	3	4	4	5
01 Jan 2000, 03:35	1	2	2	3	3	4	4	5
01 Jan 2000, 03:40	1	2	2	3	3	4	4	5
01 Jan 2000, 03:45	1	2	2	3	3	4	4	5
01 Jan 2000, 03:50	2	2	2	3	3	4	4	5
01 Jan 2000, 03:55	2	2	2	3	3	4	4	5
01 Jan 2000, 04:00	2	2	3	3	4	4	5	5
01 Jan 2000, 04:05	2	2	3	3	4	4	5	6
01 Jan 2000, 04:10	2	2	3	3	4	5	5	6
01 Jan 2000, 04:15	2	2	3	4	4	5	5	6
01 Jan 2000, 04:20	2	2	3	4	4	5	5	6
01 Jan 2000, 04:25	2	3	3	4	4	5	5	6
01 Jan 2000, 04:30	2	3	3	4	4	5	5	6
01 Jan 2000, 04:35	2	3	3	4	4	5	5	6
01 Jan 2000, 04:40	2	3	3	4	4	5	5	7
01 Jan 2000, 04:45	2	3	3	4	4	5	5	7
01 Jan 2000, 04:50	2	3	3	4	4	5	5	7
01 Jan 2000, 04:55	2	3	3	4	4	5	5	7
01 Jan 2000, 05:00	2	3	3	4	4	5	6	7
01 Jan 2000, 05:05	2	3	3	4	4	5	6	7
01 Jan 2000, 05:10	2	3	3	4	4	5	6	7
01 Jan 2000, 05:15	2	3	3	4	4	5	6	8
01 Jan 2000, 05:20	2	3	3	4	4	5	6	8
01 Jan 2000, 05:25	2	3	3	4	4	5	6	8
01 Jan 2000, 05:30	2	3	3	4	4	5	6	8
01 Jan 2000, 05:35	2	3	3	4	4	5	6	8
01 Jan 2000, 05:40	2	3	3	4	4	5	6	8
01 Jan 2000, 05:45	2	3	3	4	4	5	6	8
01 Jan 2000, 05:50	2	3	3	4	4	5	7	8
01 Jan 2000, 05:55	2	3	3	4	4	5	7	8
01 Jan 2000, 06:00	2	3	3	4	4	5	7	9
01 Jan 2000, 06:05	2	3	3	4	4	6	7	9
01 Jan 2000, 06:10	2	3	3	4	5	6	7	9
01 Jan 2000, 06:15	2	3	3	4	5	6	8	10

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	2	3	3	4	5	7	8	11
01 Jan 2000, 06:25	2	3	4	4	5	7	9	11
01 Jan 2000, 06:30	2	3	4	5	6	7	9	11
01 Jan 2000, 06:35	2	3	4	5	6	7	9	12
01 Jan 2000, 06:40	2	3	4	5	6	8	9	12
01 Jan 2000, 06:45	2	3	4	5	6	8	9	12
01 Jan 2000, 06:50	2	3	4	5	6	8	10	12
01 Jan 2000, 06:55	2	3	4	5	6	8	10	12
01 Jan 2000, 07:00	2	3	4	5	6	8	10	12
01 Jan 2000, 07:05	2	3	4	5	6	8	10	13
01 Jan 2000, 07:10	2	3	4	5	7	8	10	13
01 Jan 2000, 07:15	2	3	4	5	7	8	10	13
01 Jan 2000, 07:20	2	3	4	5	7	8	10	13
01 Jan 2000, 07:25	2	3	4	5	7	9	10	13
01 Jan 2000, 07:30	2	3	4	5	7	9	11	13
01 Jan 2000, 07:35	2	3	4	6	7	9	11	13
01 Jan 2000, 07:40	2	3	4	6	7	9	11	14
01 Jan 2000, 07:45	2	3	4	6	7	9	11	14
01 Jan 2000, 07:50	2	3	4	6	7	9	11	14
01 Jan 2000, 07:55	2	3	4	6	7	9	11	14
01 Jan 2000, 08:00	2	3	4	6	7	9	11	14
01 Jan 2000, 08:05	3	3	4	6	8	9	11	14
01 Jan 2000, 08:10	3	3	4	6	8	10	12	15
01 Jan 2000, 08:15	3	3	5	7	8	10	13	16
01 Jan 2000, 08:20	3	4	5	7	9	11	13	17
01 Jan 2000, 08:25	3	4	5	7	9	12	14	18
01 Jan 2000, 08:30	3	4	6	8	10	12	15	19
01 Jan 2000, 08:35	3	4	6	8	10	13	16	20
01 Jan 2000, 08:40	3	4	6	9	11	14	16	20
01 Jan 2000, 08:45	3	5	6	9	11	14	17	21
01 Jan 2000, 08:50	3	5	6	9	11	14	17	21
01 Jan 2000, 08:55	3	5	7	9	12	14	17	21
01 Jan 2000, 09:00	3	5	7	9	12	15	17	22
01 Jan 2000, 09:05	4	5	7	10	12	15	18	22
01 Jan 2000, 09:10	4	5	7	10	12	15	18	23
01 Jan 2000, 09:15	4	5	7	10	13	16	19	24
01 Jan 2000, 09:20	4	6	8	11	13	17	20	25
01 Jan 2000, 09:25	4	6	8	11	14	17	20	25
01 Jan 2000, 09:30	4	6	8	11	14	17	21	26
01 Jan 2000, 09:35	4	6	8	12	14	18	21	26

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	4	6	9	12	15	18	22	27
01 Jan 2000, 09:45	4	7	9	13	16	19	23	28
01 Jan 2000, 09:50	5	7	9	13	16	20	24	29
01 Jan 2000, 09:55	5	7	10	13	17	20	24	30
01 Jan 2000, 10:00	5	7	10	14	17	21	25	30
01 Jan 2000, 10:05	5	8	10	14	17	21	25	31
01 Jan 2000, 10:10	5	8	10	14	18	22	26	32
01 Jan 2000, 10:15	5	8	11	15	18	23	27	33
01 Jan 2000, 10:20	6	8	11	16	19	24	28	34
01 Jan 2000, 10:25	6	9	12	17	20	25	30	36
01 Jan 2000, 10:30	6	10	13	18	22	27	31	38
01 Jan 2000, 10:35	7	10	14	19	23	28	34	41
01 Jan 2000, 10:40	7	11	15	21	25	31	36	44
01 Jan 2000, 10:45	8	12	16	22	27	33	39	48
01 Jan 2000, 10:50	9	13	17	24	29	36	42	51
01 Jan 2000, 10:55	10	14	19	26	32	39	46	55
01 Jan 2000, 11:00	10	16	21	28	34	42	49	60
01 Jan 2000, 11:05	11	17	22	30	37	45	53	64
01 Jan 2000, 11:10	12	18	24	32	39	48	56	68
01 Jan 2000, 11:15	13	19	26	35	42	51	60	73
01 Jan 2000, 11:20	14	21	27	37	45	54	64	77
01 Jan 2000, 11:25	15	23	30	40	49	59	69	83
01 Jan 2000, 11:30	17	25	32	44	53	64	75	90
01 Jan 2000, 11:35	21	31	40	54	65	79	92	111
01 Jan 2000, 11:40	32	48	62	83	99	119	139	167
01 Jan 2000, 11:45	49	72	92	123	148	177	206	246
01 Jan 2000, 11:50	74	106	136	181	216	257	299	356
01 Jan 2000, 11:55	116	165	210	277	328	389	450	534
01 Jan 2000, 12:00	170	240	303	396	467	551	636	751
01 Jan 2000, 12:05	209	293	368	478	562	661	760	895
01 Jan 2000, 12:10	205	286	357	463	543	637	732	860
01 Jan 2000, 12:15	168	233	290	375	439	515	590	693
01 Jan 2000, 12:20	126	174	216	279	326	382	438	514
01 Jan 2000, 12:25	93	128	159	204	239	280	320	375
01 Jan 2000, 12:30	72	98	122	157	183	214	245	287
01 Jan 2000, 12:35	57	78	97	124	145	169	193	226
01 Jan 2000, 12:40	47	64	80	102	118	138	158	184
01 Jan 2000, 12:45	40	55	68	87	101	118	134	157
01 Jan 2000, 12:50	36	48	60	76	89	103	118	137
01 Jan 2000, 12:55	32	43	53	68	79	92	105	122

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	29	39	48	61	71	82	94	110
01 Jan 2000, 13:05	26	35	44	55	64	75	85	99
01 Jan 2000, 13:10	24	32	40	51	59	68	78	91
01 Jan 2000, 13:15	22	30	37	47	54	63	72	84
01 Jan 2000, 13:20	21	28	35	44	51	59	67	79
01 Jan 2000, 13:25	20	27	33	41	48	56	64	74
01 Jan 2000, 13:30	19	25	31	39	46	53	60	70
01 Jan 2000, 13:35	18	24	29	37	43	50	57	67
01 Jan 2000, 13:40	17	23	28	35	41	48	54	63
01 Jan 2000, 13:45	16	22	26	34	39	45	51	60
01 Jan 2000, 13:50	15	21	25	32	37	43	49	57
01 Jan 2000, 13:55	15	20	24	30	35	41	47	54
01 Jan 2000, 14:00	14	19	23	29	34	39	45	52
01 Jan 2000, 14:05	13	18	22	28	33	38	43	50
01 Jan 2000, 14:10	13	18	22	27	32	37	42	48
01 Jan 2000, 14:15	13	17	21	26	30	35	40	47
01 Jan 2000, 14:20	12	16	20	25	29	34	39	45
01 Jan 2000, 14:25	12	16	19	24	28	33	37	44
01 Jan 2000, 14:30	11	15	19	24	27	32	36	42
01 Jan 2000, 14:35	11	15	18	23	26	30	35	40
01 Jan 2000, 14:40	10	14	17	22	25	29	33	39
01 Jan 2000, 14:45	10	13	17	21	24	28	32	37
01 Jan 2000, 14:50	10	13	16	20	23	27	31	36
01 Jan 2000, 14:55	9	13	16	20	23	26	30	35
01 Jan 2000, 15:00	9	13	15	19	22	26	30	34
01 Jan 2000, 15:05	9	12	15	19	22	26	29	34
01 Jan 2000, 15:10	9	12	15	19	22	26	29	34
01 Jan 2000, 15:15	9	12	15	19	22	26	29	34
01 Jan 2000, 15:20	9	12	15	19	22	25	29	34
01 Jan 2000, 15:25	9	12	15	18	21	25	28	33
01 Jan 2000, 15:30	9	12	14	18	21	24	27	32
01 Jan 2000, 15:35	8	11	14	17	20	23	26	31
01 Jan 2000, 15:40	8	11	13	17	20	23	26	30
01 Jan 2000, 15:45	8	11	13	17	19	22	25	29
01 Jan 2000, 15:50	8	11	13	16	19	22	25	29
01 Jan 2000, 15:55	8	11	13	16	19	22	25	29
01 Jan 2000, 16:00	8	11	13	16	19	22	25	29
01 Jan 2000, 16:05	8	11	13	16	19	22	25	29
01 Jan 2000, 16:10	8	11	13	16	19	22	25	29
01 Jan 2000, 16:15	8	11	13	16	19	22	25	29

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	8	11	13	16	19	22	25	29
01 Jan 2000, 16:25	8	11	13	16	19	22	25	29
01 Jan 2000, 16:30	8	11	13	16	19	22	25	29
01 Jan 2000, 16:35	8	11	13	16	19	22	25	29
01 Jan 2000, 16:40	8	10	13	16	18	21	24	28
01 Jan 2000, 16:45	7	10	12	15	18	20	23	27
01 Jan 2000, 16:50	7	9	12	15	17	20	22	26
01 Jan 2000, 16:55	7	9	11	14	16	19	22	25
01 Jan 2000, 17:00	7	9	11	14	16	19	21	25
01 Jan 2000, 17:05	7	9	11	14	16	19	21	24
01 Jan 2000, 17:10	7	9	11	14	16	18	21	24
01 Jan 2000, 17:15	7	9	11	14	16	18	21	24
01 Jan 2000, 17:20	7	9	11	14	16	18	21	24
01 Jan 2000, 17:25	7	9	11	14	16	18	21	24
01 Jan 2000, 17:30	7	9	11	14	16	18	21	24
01 Jan 2000, 17:35	7	9	11	14	16	18	21	24
01 Jan 2000, 17:40	7	9	11	14	16	18	21	24
01 Jan 2000, 17:45	7	9	11	14	16	18	21	24
01 Jan 2000, 17:50	7	9	11	14	16	18	21	24
01 Jan 2000, 17:55	6	9	10	13	15	18	20	23
01 Jan 2000, 18:00	6	8	10	12	14	17	19	22
01 Jan 2000, 18:05	6	8	9	12	14	16	18	21
01 Jan 2000, 18:10	6	7	9	12	13	15	17	20
01 Jan 2000, 18:15	5	7	9	11	13	15	17	20
01 Jan 2000, 18:20	5	7	9	11	13	15	17	20
01 Jan 2000, 18:25	5	7	9	11	13	15	17	20
01 Jan 2000, 18:30	5	7	9	11	13	15	17	19
01 Jan 2000, 18:35	5	7	9	11	13	15	17	19
01 Jan 2000, 18:40	5	7	9	11	13	15	17	19
01 Jan 2000, 18:45	5	7	9	11	13	15	17	19
01 Jan 2000, 18:50	5	7	9	11	13	15	17	19
01 Jan 2000, 18:55	5	7	9	11	13	15	17	19
01 Jan 2000, 19:00	5	7	9	11	13	15	17	19
01 Jan 2000, 19:05	5	7	9	11	13	15	17	19
01 Jan 2000, 19:10	5	7	9	11	13	15	17	19
01 Jan 2000, 19:15	5	7	9	11	13	15	17	19
01 Jan 2000, 19:20	5	7	9	11	13	15	17	19
01 Jan 2000, 19:25	5	7	9	11	13	15	17	19
01 Jan 2000, 19:30	5	7	9	11	13	15	17	19
01 Jan 2000, 19:35	5	7	9	11	13	15	17	19

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	5	7	9	11	13	15	17	19
01 Jan 2000, 19:45	5	7	9	11	13	15	17	19
01 Jan 2000, 19:50	5	7	9	11	13	15	16	19
01 Jan 2000, 19:55	5	7	8	10	12	14	16	18
01 Jan 2000, 20:00	5	6	8	10	11	13	15	17
01 Jan 2000, 20:05	4	6	7	9	11	12	14	16
01 Jan 2000, 20:10	4	6	7	9	10	12	13	16
01 Jan 2000, 20:15	4	6	7	9	10	11	13	15
01 Jan 2000, 20:20	4	6	7	8	10	11	13	15
01 Jan 2000, 20:25	4	5	7	8	10	11	13	15
01 Jan 2000, 20:30	4	5	7	8	10	11	13	15
01 Jan 2000, 20:35	4	5	7	8	10	11	13	15
01 Jan 2000, 20:40	4	5	7	8	10	11	13	15
01 Jan 2000, 20:45	4	5	7	8	10	11	13	15
01 Jan 2000, 20:50	4	5	7	8	10	11	13	15
01 Jan 2000, 20:55	4	5	7	8	10	11	13	15
01 Jan 2000, 21:00	4	5	7	8	10	11	13	15
01 Jan 2000, 21:05	4	5	7	8	10	11	13	15
01 Jan 2000, 21:10	4	5	7	8	10	11	13	15
01 Jan 2000, 21:15	4	5	7	8	10	11	13	15
01 Jan 2000, 21:20	4	5	7	8	10	11	13	15
01 Jan 2000, 21:25	4	5	7	8	10	11	13	15
01 Jan 2000, 21:30	4	5	7	8	10	11	13	15
01 Jan 2000, 21:35	4	5	7	8	10	11	13	15
01 Jan 2000, 21:40	4	5	7	8	10	11	13	15
01 Jan 2000, 21:45	4	5	7	8	10	11	13	15
01 Jan 2000, 21:50	4	5	7	8	10	11	13	15
01 Jan 2000, 21:55	4	5	7	8	10	11	13	15
01 Jan 2000, 22:00	4	5	7	8	10	11	13	15
01 Jan 2000, 22:05	4	5	7	8	10	11	13	15
01 Jan 2000, 22:10	4	5	7	8	10	11	13	15
01 Jan 2000, 22:15	4	5	7	8	10	11	13	15
01 Jan 2000, 22:20	4	5	7	8	10	11	13	15
01 Jan 2000, 22:25	4	5	7	8	10	11	13	15
01 Jan 2000, 22:30	4	5	7	8	10	11	13	15
01 Jan 2000, 22:35	4	5	7	8	10	11	13	15
01 Jan 2000, 22:40	4	5	7	8	10	11	13	15
01 Jan 2000, 22:45	4	5	7	8	10	11	13	15
01 Jan 2000, 22:50	4	5	7	8	10	11	13	15
01 Jan 2000, 22:55	4	5	7	8	10	11	13	15

TC41								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	4	5	7	8	10	11	13	15
01 Jan 2000, 23:05	4	5	7	8	10	11	13	15
01 Jan 2000, 23:10	4	5	7	8	10	11	13	15
01 Jan 2000, 23:15	4	5	7	8	10	11	13	15
01 Jan 2000, 23:20	4	5	7	8	10	11	13	15
01 Jan 2000, 23:25	4	5	7	8	10	11	13	15
01 Jan 2000, 23:30	4	5	7	8	10	11	13	15
01 Jan 2000, 23:35	4	5	7	8	10	11	13	15
01 Jan 2000, 23:40	4	5	7	8	10	11	13	15
01 Jan 2000, 23:45	4	5	7	8	10	11	13	15
01 Jan 2000, 23:50	4	5	6	8	9	11	12	14
01 Jan 2000, 23:55	4	5	6	8	9	10	12	14
01 Jan 2000, 24:00	3	5	6	7	8	9	11	12

E.2.11 TC48

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	0	0	0	0	0
01 Jan 2000, 00:15	0	0	1	1	1	1	1	1
01 Jan 2000, 00:20	1	1	1	1	1	2	2	2
01 Jan 2000, 00:25	1	1	1	2	2	2	3	3
01 Jan 2000, 00:30	1	2	2	2	3	3	4	4
01 Jan 2000, 00:35	2	2	2	3	3	4	4	5
01 Jan 2000, 00:40	2	2	3	3	4	5	5	6
01 Jan 2000, 00:45	2	3	3	4	4	5	6	6
01 Jan 2000, 00:50	2	3	3	4	5	5	6	7
01 Jan 2000, 00:55	2	3	4	4	5	6	6	7
01 Jan 2000, 01:00	2	3	4	4	5	6	7	8
01 Jan 2000, 01:05	2	3	4	5	5	6	7	8
01 Jan 2000, 01:10	3	3	4	5	5	6	7	8
01 Jan 2000, 01:15	3	3	4	5	5	6	7	8
01 Jan 2000, 01:20	3	3	4	5	5	6	7	8
01 Jan 2000, 01:25	3	3	4	5	5	6	7	8
01 Jan 2000, 01:30	3	3	4	5	5	6	7	8
01 Jan 2000, 01:35	3	3	4	5	5	6	7	8
01 Jan 2000, 01:40	3	3	4	5	5	6	7	8
01 Jan 2000, 01:45	3	3	4	5	5	6	7	8
01 Jan 2000, 01:50	3	3	4	5	5	6	7	8
01 Jan 2000, 01:55	3	3	4	5	5	6	7	8
01 Jan 2000, 02:00	3	3	4	5	5	6	7	8
01 Jan 2000, 02:05	3	3	4	5	5	6	7	8
01 Jan 2000, 02:10	3	3	4	5	5	6	7	8
01 Jan 2000, 02:15	3	3	4	5	5	6	7	8
01 Jan 2000, 02:20	3	3	4	5	5	6	7	8
01 Jan 2000, 02:25	3	3	4	5	5	6	7	8
01 Jan 2000, 02:30	3	3	4	5	5	6	7	8
01 Jan 2000, 02:35	3	3	4	5	5	6	7	8
01 Jan 2000, 02:40	3	3	4	5	5	6	7	8
01 Jan 2000, 02:45	3	3	4	5	5	6	7	8
01 Jan 2000, 02:50	3	3	4	5	5	6	7	8
01 Jan 2000, 02:55	3	3	4	5	5	6	7	8

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	3	3	4	5	5	6	7	8
01 Jan 2000, 03:05	3	3	4	5	5	6	7	8
01 Jan 2000, 03:10	3	3	4	5	5	6	7	8
01 Jan 2000, 03:15	3	3	4	5	5	6	7	8
01 Jan 2000, 03:20	3	3	4	5	5	6	7	8
01 Jan 2000, 03:25	3	3	4	5	5	6	7	8
01 Jan 2000, 03:30	3	3	4	5	5	6	7	8
01 Jan 2000, 03:35	3	3	4	5	5	6	7	8
01 Jan 2000, 03:40	3	3	4	5	5	6	7	8
01 Jan 2000, 03:45	3	3	4	5	5	6	7	8
01 Jan 2000, 03:50	3	3	4	5	5	6	7	8
01 Jan 2000, 03:55	3	3	4	5	6	6	7	8
01 Jan 2000, 04:00	3	4	4	5	6	7	7	9
01 Jan 2000, 04:05	3	4	4	5	6	7	8	9
01 Jan 2000, 04:10	3	4	5	6	6	7	8	9
01 Jan 2000, 04:15	3	4	5	6	7	8	9	10
01 Jan 2000, 04:20	3	4	5	6	7	8	9	10
01 Jan 2000, 04:25	3	4	5	6	7	8	9	10
01 Jan 2000, 04:30	3	4	5	6	7	8	9	10
01 Jan 2000, 04:35	3	4	5	6	7	8	9	11
01 Jan 2000, 04:40	3	4	5	6	7	8	9	11
01 Jan 2000, 04:45	3	4	5	6	7	8	9	11
01 Jan 2000, 04:50	3	4	5	6	7	8	9	11
01 Jan 2000, 04:55	3	4	5	6	7	8	9	11
01 Jan 2000, 05:00	3	4	5	6	7	8	9	11
01 Jan 2000, 05:05	3	4	5	6	7	8	9	11
01 Jan 2000, 05:10	3	4	5	6	7	8	9	11
01 Jan 2000, 05:15	3	4	5	6	7	8	9	11
01 Jan 2000, 05:20	3	4	5	6	7	8	9	11
01 Jan 2000, 05:25	3	4	5	6	7	8	9	11
01 Jan 2000, 05:30	3	4	5	6	7	8	9	12
01 Jan 2000, 05:35	3	4	5	6	7	8	10	12
01 Jan 2000, 05:40	3	4	5	6	7	8	10	12
01 Jan 2000, 05:45	3	4	5	6	7	8	10	12
01 Jan 2000, 05:50	3	4	5	6	7	8	10	12
01 Jan 2000, 05:55	3	4	5	6	7	8	10	12
01 Jan 2000, 06:00	3	4	5	6	7	8	10	12
01 Jan 2000, 06:05	3	4	5	6	7	8	10	12
01 Jan 2000, 06:10	4	4	5	6	7	9	10	13
01 Jan 2000, 06:15	4	5	5	7	8	9	11	13

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	4	5	6	7	8	9	11	14
01 Jan 2000, 06:25	4	5	6	7	8	10	12	15
01 Jan 2000, 06:30	4	5	6	7	8	10	12	15
01 Jan 2000, 06:35	4	5	6	8	9	11	13	16
01 Jan 2000, 06:40	4	5	6	8	9	11	13	16
01 Jan 2000, 06:45	4	5	6	8	9	11	13	16
01 Jan 2000, 06:50	4	5	6	8	9	11	13	17
01 Jan 2000, 06:55	4	5	6	8	9	11	14	17
01 Jan 2000, 07:00	4	5	6	8	9	12	14	17
01 Jan 2000, 07:05	4	5	6	8	10	12	14	17
01 Jan 2000, 07:10	4	5	6	8	10	12	14	17
01 Jan 2000, 07:15	4	5	6	8	10	12	14	17
01 Jan 2000, 07:20	4	5	6	8	10	12	14	18
01 Jan 2000, 07:25	4	5	6	8	10	12	14	18
01 Jan 2000, 07:30	4	5	6	8	10	12	15	18
01 Jan 2000, 07:35	4	5	6	8	10	12	15	18
01 Jan 2000, 07:40	4	5	6	8	10	12	15	18
01 Jan 2000, 07:45	4	5	6	8	10	13	15	18
01 Jan 2000, 07:50	4	5	6	9	10	13	15	18
01 Jan 2000, 07:55	4	5	6	9	10	13	15	19
01 Jan 2000, 08:00	4	5	6	9	11	13	15	19
01 Jan 2000, 08:05	4	6	7	9	11	13	15	19
01 Jan 2000, 08:10	4	6	7	9	11	13	16	19
01 Jan 2000, 08:15	4	6	7	9	11	14	16	20
01 Jan 2000, 08:20	5	6	7	10	12	14	17	21
01 Jan 2000, 08:25	5	6	7	10	12	15	18	22
01 Jan 2000, 08:30	5	6	8	11	13	16	19	23
01 Jan 2000, 08:35	5	7	8	11	14	17	20	24
01 Jan 2000, 08:40	5	7	9	12	14	17	21	25
01 Jan 2000, 08:45	6	7	9	12	15	18	21	26
01 Jan 2000, 08:50	6	7	9	13	15	18	22	27
01 Jan 2000, 08:55	6	7	9	13	16	19	22	27
01 Jan 2000, 09:00	6	8	10	13	16	19	23	28
01 Jan 2000, 09:05	6	8	10	13	16	20	23	28
01 Jan 2000, 09:10	6	8	10	14	16	20	24	29
01 Jan 2000, 09:15	6	8	10	14	17	21	24	30
01 Jan 2000, 09:20	6	8	11	14	17	21	25	31
01 Jan 2000, 09:25	6	9	11	15	18	22	26	31
01 Jan 2000, 09:30	7	9	11	15	19	22	27	32
01 Jan 2000, 09:35	7	9	12	16	19	23	27	33

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	7	9	12	16	19	24	28	34
01 Jan 2000, 09:45	7	10	12	17	20	24	29	35
01 Jan 2000, 09:50	7	10	13	17	21	25	30	36
01 Jan 2000, 09:55	7	10	13	18	21	26	30	37
01 Jan 2000, 10:00	7	10	13	18	22	26	31	38
01 Jan 2000, 10:05	8	11	14	19	22	27	32	39
01 Jan 2000, 10:10	8	11	14	19	23	28	33	40
01 Jan 2000, 10:15	8	11	14	20	24	28	33	41
01 Jan 2000, 10:20	8	12	15	20	24	29	35	42
01 Jan 2000, 10:25	8	12	16	21	25	31	36	44
01 Jan 2000, 10:30	9	13	16	22	27	32	38	46
01 Jan 2000, 10:35	9	14	17	23	28	34	40	48
01 Jan 2000, 10:40	10	14	19	25	30	36	42	51
01 Jan 2000, 10:45	11	15	20	27	32	39	45	55
01 Jan 2000, 10:50	12	17	21	29	35	42	49	59
01 Jan 2000, 10:55	13	18	23	31	37	45	52	63
01 Jan 2000, 11:00	14	19	25	33	40	48	56	68
01 Jan 2000, 11:05	15	21	27	36	43	52	60	73
01 Jan 2000, 11:10	16	22	29	38	46	55	65	78
01 Jan 2000, 11:15	17	24	31	41	49	59	69	83
01 Jan 2000, 11:20	18	26	33	44	53	63	74	89
01 Jan 2000, 11:25	19	28	35	47	56	68	79	95
01 Jan 2000, 11:30	21	30	38	51	61	73	85	101
01 Jan 2000, 11:35	24	34	44	58	70	83	97	116
01 Jan 2000, 11:40	31	44	56	74	89	106	123	147
01 Jan 2000, 11:45	44	62	78	103	123	146	170	202
01 Jan 2000, 11:50	64	90	114	150	178	211	245	291
01 Jan 2000, 11:55	95	133	168	219	260	307	355	421
01 Jan 2000, 12:00	140	195	245	319	376	443	511	604
01 Jan 2000, 12:05	187	259	324	420	494	580	668	788
01 Jan 2000, 12:10	220	303	378	488	572	671	771	908
01 Jan 2000, 12:15	224	309	384	495	579	679	779	916
01 Jan 2000, 12:20	206	283	351	451	528	618	709	832
01 Jan 2000, 12:25	174	239	296	380	444	520	595	698
01 Jan 2000, 12:30	141	193	239	306	358	418	478	561
01 Jan 2000, 12:35	113	154	190	244	284	332	380	445
01 Jan 2000, 12:40	93	126	156	199	233	271	310	363
01 Jan 2000, 12:45	77	105	130	166	193	226	258	302
01 Jan 2000, 12:50	66	89	110	140	163	190	217	254
01 Jan 2000, 12:55	57	77	95	121	140	163	186	218

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	50	67	83	105	122	142	162	190
01 Jan 2000, 13:05	44	59	73	93	108	126	143	167
01 Jan 2000, 13:10	39	53	65	83	97	112	128	149
01 Jan 2000, 13:15	36	48	59	75	87	101	115	134
01 Jan 2000, 13:20	33	44	54	68	79	92	105	122
01 Jan 2000, 13:25	30	40	50	63	73	85	96	112
01 Jan 2000, 13:30	28	37	46	58	67	78	89	104
01 Jan 2000, 13:35	26	35	43	54	63	73	83	96
01 Jan 2000, 13:40	24	33	40	50	58	68	77	90
01 Jan 2000, 13:45	23	30	37	47	55	63	72	84
01 Jan 2000, 13:50	21	29	35	44	51	60	68	79
01 Jan 2000, 13:55	20	27	33	42	49	57	64	75
01 Jan 2000, 14:00	19	26	32	40	46	54	61	71
01 Jan 2000, 14:05	19	25	30	38	44	51	59	68
01 Jan 2000, 14:10	18	24	29	37	43	49	56	65
01 Jan 2000, 14:15	17	23	28	35	41	48	54	63
01 Jan 2000, 14:20	16	22	27	34	39	46	52	61
01 Jan 2000, 14:25	16	21	26	33	38	44	50	58
01 Jan 2000, 14:30	15	21	25	32	37	42	48	56
01 Jan 2000, 14:35	15	20	24	30	35	41	46	54
01 Jan 2000, 14:40	14	19	23	29	34	39	45	52
01 Jan 2000, 14:45	14	18	22	28	33	38	43	50
01 Jan 2000, 14:50	13	18	21	27	31	36	41	48
01 Jan 2000, 14:55	13	17	21	26	30	35	40	46
01 Jan 2000, 15:00	12	17	20	25	29	34	39	45
01 Jan 2000, 15:05	12	16	20	25	29	33	38	44
01 Jan 2000, 15:10	12	16	19	25	28	33	37	43
01 Jan 2000, 15:15	12	16	19	24	28	33	37	43
01 Jan 2000, 15:20	12	16	19	24	28	32	37	43
01 Jan 2000, 15:25	12	15	19	24	27	32	36	42
01 Jan 2000, 15:30	11	15	18	23	27	31	35	41
01 Jan 2000, 15:35	11	15	18	23	26	30	34	40
01 Jan 2000, 15:40	11	14	18	22	26	30	34	39
01 Jan 2000, 15:45	11	14	17	22	25	29	33	38
01 Jan 2000, 15:50	10	14	17	21	25	28	32	38
01 Jan 2000, 15:55	10	14	17	21	24	28	32	37
01 Jan 2000, 16:00	10	14	16	21	24	28	32	37
01 Jan 2000, 16:05	10	13	16	21	24	28	31	37
01 Jan 2000, 16:10	10	13	16	21	24	28	31	36
01 Jan 2000, 16:15	10	13	16	21	24	27	31	36

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	10	13	16	20	24	27	31	36
01 Jan 2000, 16:25	10	13	16	20	24	27	31	36
01 Jan 2000, 16:30	10	13	16	20	24	27	31	36
01 Jan 2000, 16:35	10	13	16	20	24	27	31	36
01 Jan 2000, 16:40	10	13	16	20	23	27	31	36
01 Jan 2000, 16:45	10	13	16	20	23	26	30	35
01 Jan 2000, 16:50	9	13	15	19	22	26	29	34
01 Jan 2000, 16:55	9	12	15	19	22	25	28	33
01 Jan 2000, 17:00	9	12	14	18	21	24	28	32
01 Jan 2000, 17:05	9	12	14	18	21	24	27	32
01 Jan 2000, 17:10	9	11	14	18	20	24	27	31
01 Jan 2000, 17:15	9	11	14	17	20	23	26	31
01 Jan 2000, 17:20	8	11	14	17	20	23	26	31
01 Jan 2000, 17:25	8	11	14	17	20	23	26	30
01 Jan 2000, 17:30	8	11	14	17	20	23	26	30
01 Jan 2000, 17:35	8	11	14	17	20	23	26	30
01 Jan 2000, 17:40	8	11	14	17	20	23	26	30
01 Jan 2000, 17:45	8	11	14	17	20	23	26	30
01 Jan 2000, 17:50	8	11	14	17	20	23	26	30
01 Jan 2000, 17:55	8	11	13	17	19	22	26	30
01 Jan 2000, 18:00	8	11	13	16	19	22	25	29
01 Jan 2000, 18:05	8	10	13	16	18	21	24	28
01 Jan 2000, 18:10	8	10	12	15	18	20	23	27
01 Jan 2000, 18:15	7	10	12	15	17	20	23	26
01 Jan 2000, 18:20	7	9	12	14	17	19	22	26
01 Jan 2000, 18:25	7	9	11	14	16	19	22	25
01 Jan 2000, 18:30	7	9	11	14	16	19	21	25
01 Jan 2000, 18:35	7	9	11	14	16	19	21	25
01 Jan 2000, 18:40	7	9	11	14	16	19	21	24
01 Jan 2000, 18:45	7	9	11	14	16	18	21	24
01 Jan 2000, 18:50	7	9	11	14	16	18	21	24
01 Jan 2000, 18:55	7	9	11	14	16	18	21	24
01 Jan 2000, 19:00	7	9	11	14	16	18	21	24
01 Jan 2000, 19:05	7	9	11	14	16	18	21	24
01 Jan 2000, 19:10	7	9	11	14	16	18	21	24
01 Jan 2000, 19:15	7	9	11	14	16	18	21	24
01 Jan 2000, 19:20	7	9	11	14	16	18	21	24
01 Jan 2000, 19:25	7	9	11	14	16	18	21	24
01 Jan 2000, 19:30	7	9	11	14	16	18	21	24
01 Jan 2000, 19:35	7	9	11	14	16	18	21	24

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	7	9	11	14	16	18	21	24
01 Jan 2000, 19:45	7	9	11	14	16	18	21	24
01 Jan 2000, 19:50	7	9	11	14	16	18	21	24
01 Jan 2000, 19:55	7	9	11	13	16	18	20	24
01 Jan 2000, 20:00	6	9	10	13	15	17	20	23
01 Jan 2000, 20:05	6	8	10	13	14	17	19	22
01 Jan 2000, 20:10	6	8	10	12	14	16	18	21
01 Jan 2000, 20:15	6	8	9	11	13	15	17	20
01 Jan 2000, 20:20	5	7	9	11	13	15	17	20
01 Jan 2000, 20:25	5	7	9	11	13	15	16	19
01 Jan 2000, 20:30	5	7	9	11	12	14	16	19
01 Jan 2000, 20:35	5	7	8	11	12	14	16	19
01 Jan 2000, 20:40	5	7	8	10	12	14	16	18
01 Jan 2000, 20:45	5	7	8	10	12	14	16	18
01 Jan 2000, 20:50	5	7	8	10	12	14	16	18
01 Jan 2000, 20:55	5	7	8	10	12	14	16	18
01 Jan 2000, 21:00	5	7	8	10	12	14	16	18
01 Jan 2000, 21:05	5	7	8	10	12	14	16	18
01 Jan 2000, 21:10	5	7	8	10	12	14	16	18
01 Jan 2000, 21:15	5	7	8	10	12	14	16	18
01 Jan 2000, 21:20	5	7	8	10	12	14	16	18
01 Jan 2000, 21:25	5	7	8	10	12	14	16	18
01 Jan 2000, 21:30	5	7	8	10	12	14	16	18
01 Jan 2000, 21:35	5	7	8	10	12	14	16	18
01 Jan 2000, 21:40	5	7	8	10	12	14	16	18
01 Jan 2000, 21:45	5	7	8	10	12	14	16	18
01 Jan 2000, 21:50	5	7	8	10	12	14	16	18
01 Jan 2000, 21:55	5	7	8	10	12	14	16	18
01 Jan 2000, 22:00	5	7	8	10	12	14	16	18
01 Jan 2000, 22:05	5	7	8	10	12	14	16	18
01 Jan 2000, 22:10	5	7	8	10	12	14	16	18
01 Jan 2000, 22:15	5	7	8	10	12	14	16	18
01 Jan 2000, 22:20	5	7	8	10	12	14	16	18
01 Jan 2000, 22:25	5	7	8	10	12	14	16	18
01 Jan 2000, 22:30	5	7	8	10	12	14	16	18
01 Jan 2000, 22:35	5	7	8	10	12	14	16	18
01 Jan 2000, 22:40	5	7	8	10	12	14	16	18
01 Jan 2000, 22:45	5	7	8	10	12	14	16	18
01 Jan 2000, 22:50	5	7	8	10	12	14	16	18
01 Jan 2000, 22:55	5	7	8	10	12	14	16	18

TC48								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	5	7	8	10	12	14	16	18
01 Jan 2000, 23:05	5	7	8	10	12	14	16	18
01 Jan 2000, 23:10	5	7	8	10	12	14	16	18
01 Jan 2000, 23:15	5	7	8	10	12	14	16	18
01 Jan 2000, 23:20	5	7	8	10	12	14	16	18
01 Jan 2000, 23:25	5	7	8	10	12	14	16	18
01 Jan 2000, 23:30	5	7	8	10	12	14	16	18
01 Jan 2000, 23:35	5	7	8	10	12	14	16	18
01 Jan 2000, 23:40	5	7	8	10	12	14	16	18
01 Jan 2000, 23:45	5	7	8	10	12	14	16	18
01 Jan 2000, 23:50	5	7	8	10	12	14	16	18
01 Jan 2000, 23:55	5	7	8	10	12	13	15	18
01 Jan 2000, 24:00	5	6	8	10	11	13	15	17

E.2.12 TC38

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	0	1	1	1	1
01 Jan 2000, 00:15	0	1	1	1	1	1	1	1
01 Jan 2000, 00:20	1	1	1	1	1	1	2	2
01 Jan 2000, 00:25	1	1	1	1	2	2	2	2
01 Jan 2000, 00:30	1	1	1	2	2	2	2	3
01 Jan 2000, 00:35	1	1	2	2	2	2	3	3
01 Jan 2000, 00:40	1	1	2	2	2	3	3	3
01 Jan 2000, 00:45	1	1	2	2	2	3	3	3
01 Jan 2000, 00:50	1	1	2	2	2	3	3	3
01 Jan 2000, 00:55	1	1	2	2	2	3	3	3
01 Jan 2000, 01:00	1	1	2	2	2	3	3	3
01 Jan 2000, 01:05	1	1	2	2	2	3	3	3
01 Jan 2000, 01:10	1	1	2	2	2	3	3	3
01 Jan 2000, 01:15	1	1	2	2	2	3	3	3
01 Jan 2000, 01:20	1	1	2	2	2	3	3	3
01 Jan 2000, 01:25	1	1	2	2	2	3	3	3
01 Jan 2000, 01:30	1	1	2	2	2	3	3	3
01 Jan 2000, 01:35	1	1	2	2	2	3	3	3
01 Jan 2000, 01:40	1	1	2	2	2	3	3	3
01 Jan 2000, 01:45	1	1	2	2	2	3	3	3
01 Jan 2000, 01:50	1	1	2	2	2	3	3	3
01 Jan 2000, 01:55	1	1	2	2	2	3	3	3
01 Jan 2000, 02:00	1	1	2	2	2	3	3	3
01 Jan 2000, 02:05	1	1	2	2	2	3	3	3
01 Jan 2000, 02:10	1	1	2	2	2	3	3	3
01 Jan 2000, 02:15	1	1	2	2	2	3	3	3
01 Jan 2000, 02:20	1	1	2	2	2	3	3	3
01 Jan 2000, 02:25	1	1	2	2	2	3	3	3
01 Jan 2000, 02:30	1	1	2	2	2	3	3	3
01 Jan 2000, 02:35	1	1	2	2	2	3	3	3
01 Jan 2000, 02:40	1	1	2	2	2	3	3	3
01 Jan 2000, 02:45	1	1	2	2	2	3	3	3
01 Jan 2000, 02:50	1	1	2	2	2	3	3	3
01 Jan 2000, 02:55	1	1	2	2	2	3	3	3

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	1	1	2	2	2	3	3	3
01 Jan 2000, 03:05	1	1	2	2	2	3	3	3
01 Jan 2000, 03:10	1	1	2	2	2	3	3	3
01 Jan 2000, 03:15	1	1	2	2	2	3	3	3
01 Jan 2000, 03:20	1	1	2	2	2	3	3	3
01 Jan 2000, 03:25	1	1	2	2	2	3	3	3
01 Jan 2000, 03:30	1	1	2	2	2	3	3	3
01 Jan 2000, 03:35	1	1	2	2	2	3	3	3
01 Jan 2000, 03:40	1	1	2	2	2	3	3	3
01 Jan 2000, 03:45	1	1	2	2	2	3	3	3
01 Jan 2000, 03:50	1	1	2	2	2	3	3	4
01 Jan 2000, 03:55	1	2	2	2	3	3	3	4
01 Jan 2000, 04:00	1	2	2	2	3	3	4	4
01 Jan 2000, 04:05	1	2	2	3	3	3	4	4
01 Jan 2000, 04:10	1	2	2	3	3	3	4	4
01 Jan 2000, 04:15	1	2	2	3	3	3	4	5
01 Jan 2000, 04:20	1	2	2	3	3	3	4	5
01 Jan 2000, 04:25	1	2	2	3	3	4	4	5
01 Jan 2000, 04:30	1	2	2	3	3	4	4	5
01 Jan 2000, 04:35	1	2	2	3	3	4	4	5
01 Jan 2000, 04:40	1	2	2	3	3	4	4	5
01 Jan 2000, 04:45	1	2	2	3	3	4	4	5
01 Jan 2000, 04:50	1	2	2	3	3	4	4	5
01 Jan 2000, 04:55	1	2	2	3	3	4	4	5
01 Jan 2000, 05:00	1	2	2	3	3	4	4	5
01 Jan 2000, 05:05	1	2	2	3	3	4	4	5
01 Jan 2000, 05:10	1	2	2	3	3	4	4	5
01 Jan 2000, 05:15	1	2	2	3	3	4	4	5
01 Jan 2000, 05:20	1	2	2	3	3	4	4	5
01 Jan 2000, 05:25	1	2	2	3	3	4	4	5
01 Jan 2000, 05:30	1	2	2	3	3	4	4	5
01 Jan 2000, 05:35	1	2	2	3	3	4	4	6
01 Jan 2000, 05:40	1	2	2	3	3	4	4	6
01 Jan 2000, 05:45	1	2	2	3	3	4	5	6
01 Jan 2000, 05:50	1	2	2	3	3	4	5	6
01 Jan 2000, 05:55	1	2	2	3	3	4	5	6
01 Jan 2000, 06:00	1	2	2	3	3	4	5	6
01 Jan 2000, 06:05	2	2	2	3	3	4	5	6
01 Jan 2000, 06:10	2	2	2	3	3	4	5	6
01 Jan 2000, 06:15	2	2	3	3	4	5	6	7

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	2	2	3	3	4	5	6	7
01 Jan 2000, 06:25	2	2	3	3	4	5	6	7
01 Jan 2000, 06:30	2	2	3	3	4	5	6	8
01 Jan 2000, 06:35	2	2	3	3	4	5	6	8
01 Jan 2000, 06:40	2	2	3	3	4	5	6	8
01 Jan 2000, 06:45	2	2	3	4	4	5	6	8
01 Jan 2000, 06:50	2	2	3	4	4	5	6	8
01 Jan 2000, 06:55	2	2	3	4	4	5	7	8
01 Jan 2000, 07:00	2	2	3	4	4	5	7	8
01 Jan 2000, 07:05	2	2	3	4	4	6	7	8
01 Jan 2000, 07:10	2	2	3	4	5	6	7	8
01 Jan 2000, 07:15	2	2	3	4	5	6	7	8
01 Jan 2000, 07:20	2	2	3	4	5	6	7	8
01 Jan 2000, 07:25	2	2	3	4	5	6	7	8
01 Jan 2000, 07:30	2	2	3	4	5	6	7	9
01 Jan 2000, 07:35	2	2	3	4	5	6	7	9
01 Jan 2000, 07:40	2	2	3	4	5	6	7	9
01 Jan 2000, 07:45	2	2	3	4	5	6	7	9
01 Jan 2000, 07:50	2	2	3	4	5	6	7	9
01 Jan 2000, 07:55	2	2	3	4	5	6	7	9
01 Jan 2000, 08:00	2	2	3	4	5	6	7	9
01 Jan 2000, 08:05	2	2	3	4	5	6	7	9
01 Jan 2000, 08:10	2	2	3	4	5	7	8	10
01 Jan 2000, 08:15	2	3	3	5	6	7	8	10
01 Jan 2000, 08:20	2	3	4	5	6	7	9	11
01 Jan 2000, 08:25	2	3	4	5	6	8	9	11
01 Jan 2000, 08:30	2	3	4	6	7	8	10	12
01 Jan 2000, 08:35	2	3	4	6	7	9	10	13
01 Jan 2000, 08:40	3	3	4	6	7	9	11	13
01 Jan 2000, 08:45	3	3	4	6	7	9	11	13
01 Jan 2000, 08:50	3	3	5	6	7	9	11	13
01 Jan 2000, 08:55	3	3	5	6	8	9	11	13
01 Jan 2000, 09:00	3	4	5	6	8	9	11	13
01 Jan 2000, 09:05	3	4	5	6	8	9	11	14
01 Jan 2000, 09:10	3	4	5	7	8	10	12	14
01 Jan 2000, 09:15	3	4	5	7	9	10	12	15
01 Jan 2000, 09:20	3	4	5	7	9	11	13	15
01 Jan 2000, 09:25	3	4	5	7	9	11	13	16
01 Jan 2000, 09:30	3	4	6	8	9	11	13	16
01 Jan 2000, 09:35	3	4	6	8	9	11	13	16

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	3	5	6	8	10	12	14	17
01 Jan 2000, 09:45	3	5	6	8	10	12	14	18
01 Jan 2000, 09:50	3	5	6	9	10	13	15	18
01 Jan 2000, 09:55	3	5	6	9	11	13	15	18
01 Jan 2000, 10:00	3	5	7	9	11	13	15	19
01 Jan 2000, 10:05	4	5	7	9	11	13	16	19
01 Jan 2000, 10:10	4	5	7	9	11	14	16	20
01 Jan 2000, 10:15	4	6	7	10	12	14	17	20
01 Jan 2000, 10:20	4	6	8	10	12	15	18	21
01 Jan 2000, 10:25	4	6	8	11	13	16	19	23
01 Jan 2000, 10:30	5	7	9	12	14	17	20	24
01 Jan 2000, 10:35	5	7	9	12	15	18	21	26
01 Jan 2000, 10:40	5	8	10	14	16	20	23	28
01 Jan 2000, 10:45	6	9	11	15	18	21	25	30
01 Jan 2000, 10:50	6	9	12	16	19	23	27	32
01 Jan 2000, 10:55	7	10	13	17	21	25	29	35
01 Jan 2000, 11:00	7	11	14	18	22	27	31	37
01 Jan 2000, 11:05	8	11	15	20	24	28	33	40
01 Jan 2000, 11:10	9	12	16	21	25	30	35	42
01 Jan 2000, 11:15	9	13	17	23	27	32	38	45
01 Jan 2000, 11:20	10	14	18	24	29	34	40	48
01 Jan 2000, 11:25	11	15	20	26	31	37	43	52
01 Jan 2000, 11:30	12	17	21	28	34	40	47	56
01 Jan 2000, 11:35	16	23	29	38	45	54	63	75
01 Jan 2000, 11:40	27	38	49	64	76	91	105	125
01 Jan 2000, 11:45	40	56	71	93	111	131	152	180
01 Jan 2000, 11:50	58	81	103	134	158	187	216	256
01 Jan 2000, 11:55	93	129	161	209	246	290	333	393
01 Jan 2000, 12:00	130	179	224	288	338	396	454	534
01 Jan 2000, 12:05	145	198	246	316	369	432	494	579
01 Jan 2000, 12:10	119	163	202	258	301	352	402	471
01 Jan 2000, 12:15	84	114	141	180	210	245	280	328
01 Jan 2000, 12:20	58	79	97	124	144	168	192	225
01 Jan 2000, 12:25	43	58	71	91	106	123	140	164
01 Jan 2000, 12:30	33	44	55	69	81	94	107	125
01 Jan 2000, 12:35	27	36	44	56	65	76	86	101
01 Jan 2000, 12:40	23	31	38	48	55	64	73	85
01 Jan 2000, 12:45	20	27	33	42	49	57	65	75
01 Jan 2000, 12:50	18	25	30	38	44	51	58	68
01 Jan 2000, 12:55	17	22	27	34	40	46	53	61

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	15	20	25	31	36	42	48	56
01 Jan 2000, 13:05	14	19	23	29	34	39	45	52
01 Jan 2000, 13:10	13	18	22	27	31	36	41	48
01 Jan 2000, 13:15	12	17	20	25	29	34	39	45
01 Jan 2000, 13:20	12	16	19	24	28	32	37	43
01 Jan 2000, 13:25	11	15	18	23	26	31	35	40
01 Jan 2000, 13:30	11	14	17	22	25	29	33	38
01 Jan 2000, 13:35	10	13	16	21	24	28	32	37
01 Jan 2000, 13:40	9	13	15	20	23	26	30	35
01 Jan 2000, 13:45	9	12	15	18	21	25	28	33
01 Jan 2000, 13:50	9	12	14	18	20	24	27	31
01 Jan 2000, 13:55	8	11	13	17	20	23	26	30
01 Jan 2000, 14:00	8	11	13	16	19	22	25	29
01 Jan 2000, 14:05	8	10	13	16	18	21	24	28
01 Jan 2000, 14:10	7	10	12	15	18	20	23	27
01 Jan 2000, 14:15	7	10	12	15	17	20	22	26
01 Jan 2000, 14:20	7	9	11	14	16	19	22	25
01 Jan 2000, 14:25	7	9	11	14	16	18	21	24
01 Jan 2000, 14:30	6	9	10	13	15	18	20	23
01 Jan 2000, 14:35	6	8	10	13	15	17	19	22
01 Jan 2000, 14:40	6	8	10	12	14	16	18	21
01 Jan 2000, 14:45	6	8	9	12	13	16	18	21
01 Jan 2000, 14:50	6	7	9	11	13	15	17	20
01 Jan 2000, 14:55	5	7	9	11	13	15	17	20
01 Jan 2000, 15:00	5	7	9	11	13	15	17	19
01 Jan 2000, 15:05	5	7	9	11	13	15	17	19
01 Jan 2000, 15:10	5	7	9	11	13	15	17	19
01 Jan 2000, 15:15	5	7	9	11	13	15	17	19
01 Jan 2000, 15:20	5	7	9	11	13	14	16	19
01 Jan 2000, 15:25	5	7	8	10	12	14	16	18
01 Jan 2000, 15:30	5	7	8	10	12	13	15	18
01 Jan 2000, 15:35	5	6	8	10	11	13	15	17
01 Jan 2000, 15:40	5	6	8	10	11	13	15	17
01 Jan 2000, 15:45	5	6	8	9	11	13	14	17
01 Jan 2000, 15:50	5	6	8	9	11	13	14	17
01 Jan 2000, 15:55	5	6	8	9	11	13	14	17
01 Jan 2000, 16:00	5	6	7	9	11	13	14	17
01 Jan 2000, 16:05	5	6	7	9	11	13	14	17
01 Jan 2000, 16:10	5	6	7	9	11	13	14	17
01 Jan 2000, 16:15	5	6	7	9	11	13	14	17

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	5	6	7	9	11	13	14	17
01 Jan 2000, 16:25	5	6	8	9	11	13	14	17
01 Jan 2000, 16:30	5	6	8	9	11	13	14	17
01 Jan 2000, 16:35	5	6	7	9	11	12	14	16
01 Jan 2000, 16:40	4	6	7	9	10	12	13	16
01 Jan 2000, 16:45	4	6	7	8	10	11	13	15
01 Jan 2000, 16:50	4	5	7	8	9	11	12	14
01 Jan 2000, 16:55	4	5	6	8	9	11	12	14
01 Jan 2000, 17:00	4	5	6	8	9	11	12	14
01 Jan 2000, 17:05	4	5	6	8	9	11	12	14
01 Jan 2000, 17:10	4	5	6	8	9	11	12	14
01 Jan 2000, 17:15	4	5	6	8	9	11	12	14
01 Jan 2000, 17:20	4	5	6	8	9	11	12	14
01 Jan 2000, 17:25	4	5	6	8	9	11	12	14
01 Jan 2000, 17:30	4	5	6	8	9	11	12	14
01 Jan 2000, 17:35	4	5	6	8	9	11	12	14
01 Jan 2000, 17:40	4	5	6	8	9	11	12	14
01 Jan 2000, 17:45	4	5	6	8	9	11	12	14
01 Jan 2000, 17:50	4	5	6	8	9	10	12	14
01 Jan 2000, 17:55	4	5	6	7	8	10	11	13
01 Jan 2000, 18:00	3	5	6	7	8	9	10	12
01 Jan 2000, 18:05	3	4	5	7	8	9	10	12
01 Jan 2000, 18:10	3	4	5	6	7	9	10	11
01 Jan 2000, 18:15	3	4	5	6	7	9	10	11
01 Jan 2000, 18:20	3	4	5	6	7	8	10	11
01 Jan 2000, 18:25	3	4	5	6	7	8	10	11
01 Jan 2000, 18:30	3	4	5	6	7	8	10	11
01 Jan 2000, 18:35	3	4	5	6	7	8	10	11
01 Jan 2000, 18:40	3	4	5	6	7	8	10	11
01 Jan 2000, 18:45	3	4	5	6	7	8	10	11
01 Jan 2000, 18:50	3	4	5	6	7	8	10	11
01 Jan 2000, 18:55	3	4	5	6	7	8	10	11
01 Jan 2000, 19:00	3	4	5	6	7	8	10	11
01 Jan 2000, 19:05	3	4	5	6	7	8	10	11
01 Jan 2000, 19:10	3	4	5	6	7	8	10	11
01 Jan 2000, 19:15	3	4	5	6	7	8	10	11
01 Jan 2000, 19:20	3	4	5	6	7	8	10	11
01 Jan 2000, 19:25	3	4	5	6	7	8	10	11
01 Jan 2000, 19:30	3	4	5	6	7	8	10	11
01 Jan 2000, 19:35	3	4	5	6	7	8	10	11

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	3	4	5	6	7	8	10	11
01 Jan 2000, 19:45	3	4	5	6	7	8	10	11
01 Jan 2000, 19:50	3	4	5	6	7	8	9	11
01 Jan 2000, 19:55	3	4	5	6	7	8	9	10
01 Jan 2000, 20:00	3	4	4	5	6	7	8	9
01 Jan 2000, 20:05	3	3	4	5	6	7	8	9
01 Jan 2000, 20:10	2	3	4	5	6	7	7	9
01 Jan 2000, 20:15	2	3	4	5	6	6	7	8
01 Jan 2000, 20:20	2	3	4	5	6	6	7	8
01 Jan 2000, 20:25	2	3	4	5	6	6	7	8
01 Jan 2000, 20:30	2	3	4	5	5	6	7	8
01 Jan 2000, 20:35	2	3	4	5	5	6	7	8
01 Jan 2000, 20:40	2	3	4	5	5	6	7	8
01 Jan 2000, 20:45	2	3	4	5	5	6	7	8
01 Jan 2000, 20:50	2	3	4	5	5	6	7	8
01 Jan 2000, 20:55	2	3	4	5	5	6	7	8
01 Jan 2000, 21:00	2	3	4	5	5	6	7	8
01 Jan 2000, 21:05	2	3	4	5	5	6	7	8
01 Jan 2000, 21:10	2	3	4	5	5	6	7	8
01 Jan 2000, 21:15	2	3	4	5	5	6	7	8
01 Jan 2000, 21:20	2	3	4	5	5	6	7	8
01 Jan 2000, 21:25	2	3	4	5	5	6	7	8
01 Jan 2000, 21:30	2	3	4	5	5	6	7	8
01 Jan 2000, 21:35	2	3	4	5	5	6	7	8
01 Jan 2000, 21:40	2	3	4	5	5	6	7	8
01 Jan 2000, 21:45	2	3	4	5	5	6	7	8
01 Jan 2000, 21:50	2	3	4	5	5	6	7	8
01 Jan 2000, 21:55	2	3	4	5	5	6	7	8
01 Jan 2000, 22:00	2	3	4	5	5	6	7	8
01 Jan 2000, 22:05	2	3	4	5	5	6	7	8
01 Jan 2000, 22:10	2	3	4	5	5	6	7	8
01 Jan 2000, 22:15	2	3	4	5	5	6	7	8
01 Jan 2000, 22:20	2	3	4	5	5	6	7	8
01 Jan 2000, 22:25	2	3	4	5	5	6	7	8
01 Jan 2000, 22:30	2	3	4	5	5	6	7	8
01 Jan 2000, 22:35	2	3	4	5	5	6	7	8
01 Jan 2000, 22:40	2	3	4	5	5	6	7	8
01 Jan 2000, 22:45	2	3	4	5	5	6	7	8
01 Jan 2000, 22:50	2	3	4	5	5	6	7	8
01 Jan 2000, 22:55	2	3	4	5	5	6	7	8

TC38								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	2	3	4	5	5	6	7	8
01 Jan 2000, 23:05	2	3	4	5	5	6	7	8
01 Jan 2000, 23:10	2	3	4	5	5	6	7	8
01 Jan 2000, 23:15	2	3	4	5	5	6	7	8
01 Jan 2000, 23:20	2	3	4	5	5	6	7	8
01 Jan 2000, 23:25	2	3	4	5	5	6	7	8
01 Jan 2000, 23:30	2	3	4	5	5	6	7	8
01 Jan 2000, 23:35	2	3	4	5	5	6	7	8
01 Jan 2000, 23:40	2	3	4	5	5	6	7	8
01 Jan 2000, 23:45	2	3	4	5	5	6	7	8
01 Jan 2000, 23:50	2	3	4	5	5	6	7	8
01 Jan 2000, 23:55	2	3	3	4	5	6	6	7
01 Jan 2000, 24:00	2	2	3	4	4	5	6	7

E.2.13 TC19

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	1	1	1	1	1
01 Jan 2000, 00:15	1	1	1	1	1	1	1	2
01 Jan 2000, 00:20	1	1	1	1	1	2	2	2
01 Jan 2000, 00:25	1	1	1	1	2	2	2	3
01 Jan 2000, 00:30	1	1	1	2	2	2	3	3
01 Jan 2000, 00:35	1	1	1	2	2	2	3	3
01 Jan 2000, 00:40	1	1	2	2	2	2	3	3
01 Jan 2000, 00:45	1	1	2	2	2	2	3	3
01 Jan 2000, 00:50	1	1	2	2	2	3	3	3
01 Jan 2000, 00:55	1	1	2	2	2	3	3	3
01 Jan 2000, 01:00	1	1	2	2	2	3	3	3
01 Jan 2000, 01:05	1	1	2	2	2	3	3	3
01 Jan 2000, 01:10	1	1	2	2	2	3	3	3
01 Jan 2000, 01:15	1	1	2	2	2	3	3	3
01 Jan 2000, 01:20	1	1	2	2	2	3	3	3
01 Jan 2000, 01:25	1	1	2	2	2	3	3	3
01 Jan 2000, 01:30	1	1	2	2	2	3	3	3
01 Jan 2000, 01:35	1	1	2	2	2	3	3	3
01 Jan 2000, 01:40	1	1	2	2	2	3	3	3
01 Jan 2000, 01:45	1	1	2	2	2	3	3	3
01 Jan 2000, 01:50	1	1	2	2	2	3	3	3
01 Jan 2000, 01:55	1	1	2	2	2	3	3	3
01 Jan 2000, 02:00	1	1	2	2	2	3	3	3
01 Jan 2000, 02:05	1	1	2	2	2	3	3	3
01 Jan 2000, 02:10	1	1	2	2	2	3	3	3
01 Jan 2000, 02:15	1	1	2	2	2	3	3	3
01 Jan 2000, 02:20	1	1	2	2	2	3	3	3
01 Jan 2000, 02:25	1	1	2	2	2	3	3	3
01 Jan 2000, 02:30	1	1	2	2	2	3	3	3
01 Jan 2000, 02:35	1	1	2	2	2	3	3	3
01 Jan 2000, 02:40	1	1	2	2	2	3	3	3
01 Jan 2000, 02:45	1	1	2	2	2	3	3	3
01 Jan 2000, 02:50	1	1	2	2	2	3	3	3
01 Jan 2000, 02:55	1	1	2	2	2	3	3	3

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	1	1	2	2	2	3	3	3
01 Jan 2000, 03:05	1	1	2	2	2	3	3	3
01 Jan 2000, 03:10	1	1	2	2	2	3	3	3
01 Jan 2000, 03:15	1	1	2	2	2	3	3	3
01 Jan 2000, 03:20	1	1	2	2	2	3	3	3
01 Jan 2000, 03:25	1	1	2	2	2	3	3	3
01 Jan 2000, 03:30	1	1	2	2	2	3	3	3
01 Jan 2000, 03:35	1	1	2	2	2	3	3	3
01 Jan 2000, 03:40	1	1	2	2	2	3	3	3
01 Jan 2000, 03:45	1	1	2	2	2	3	3	3
01 Jan 2000, 03:50	1	1	2	2	2	3	3	3
01 Jan 2000, 03:55	1	2	2	2	3	3	3	4
01 Jan 2000, 04:00	1	2	2	2	3	3	4	4
01 Jan 2000, 04:05	1	2	2	2	3	3	4	4
01 Jan 2000, 04:10	1	2	2	3	3	3	4	4
01 Jan 2000, 04:15	1	2	2	3	3	3	4	4
01 Jan 2000, 04:20	1	2	2	3	3	3	4	4
01 Jan 2000, 04:25	1	2	2	3	3	3	4	4
01 Jan 2000, 04:30	1	2	2	3	3	3	4	4
01 Jan 2000, 04:35	1	2	2	3	3	3	4	4
01 Jan 2000, 04:40	1	2	2	3	3	3	4	4
01 Jan 2000, 04:45	1	2	2	3	3	3	4	4
01 Jan 2000, 04:50	1	2	2	3	3	3	4	4
01 Jan 2000, 04:55	1	2	2	3	3	3	4	4
01 Jan 2000, 05:00	1	2	2	3	3	3	4	4
01 Jan 2000, 05:05	1	2	2	3	3	3	4	4
01 Jan 2000, 05:10	1	2	2	3	3	3	4	5
01 Jan 2000, 05:15	1	2	2	3	3	3	4	5
01 Jan 2000, 05:20	1	2	2	3	3	3	4	5
01 Jan 2000, 05:25	1	2	2	3	3	3	4	5
01 Jan 2000, 05:30	1	2	2	3	3	3	4	5
01 Jan 2000, 05:35	1	2	2	3	3	3	4	5
01 Jan 2000, 05:40	1	2	2	3	3	3	4	5
01 Jan 2000, 05:45	1	2	2	3	3	3	4	5
01 Jan 2000, 05:50	1	2	2	3	3	3	4	5
01 Jan 2000, 05:55	1	2	2	3	3	3	4	5
01 Jan 2000, 06:00	1	2	2	3	3	3	4	5
01 Jan 2000, 06:05	1	2	2	3	3	4	4	5
01 Jan 2000, 06:10	2	2	2	3	3	4	5	6
01 Jan 2000, 06:15	2	2	2	3	3	4	5	6

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	2	2	3	3	4	4	5	7
01 Jan 2000, 06:25	2	2	3	3	4	4	5	7
01 Jan 2000, 06:30	2	2	3	3	4	5	5	7
01 Jan 2000, 06:35	2	2	3	3	4	5	6	7
01 Jan 2000, 06:40	2	2	3	3	4	5	6	7
01 Jan 2000, 06:45	2	2	3	3	4	5	6	7
01 Jan 2000, 06:50	2	2	3	3	4	5	6	7
01 Jan 2000, 06:55	2	2	3	3	4	5	6	7
01 Jan 2000, 07:00	2	2	3	3	4	5	6	7
01 Jan 2000, 07:05	2	2	3	3	4	5	6	7
01 Jan 2000, 07:10	2	2	3	3	4	5	6	8
01 Jan 2000, 07:15	2	2	3	3	4	5	6	8
01 Jan 2000, 07:20	2	2	3	3	4	5	6	7.99
01 Jan 2000, 07:25	2	2	3	3	4	5	6	8.43
01 Jan 2000, 07:30	2	2	3	3	4	5	6	8.59
01 Jan 2000, 07:35	2	2	3	3	4	5	6	9.7
01 Jan 2000, 07:40	2	2	3	3	4	5	6	10.06
01 Jan 2000, 07:45	2	2	3	3	4	5	6	7.75
01 Jan 2000, 07:50	2	2	3	3	4	5	6	8.16
01 Jan 2000, 07:55	2	2	3	4	4	5	7	8.22
01 Jan 2000, 08:00	2	2	3	4	4	5	7	7.91
01 Jan 2000, 08:05	2	2	3	4	5	6	7	7.29
01 Jan 2000, 08:10	2	2	3	4	5	6	7	8.9
01 Jan 2000, 08:15	2	3	3	4	5	6	8	10.13
01 Jan 2000, 08:20	2	3	3	4	5	7	8	9.19
01 Jan 2000, 08:25	2	3	3	5	6	7	9	10.49
01 Jan 2000, 08:30	2	3	4	5	6	8	8.9	10.78
01 Jan 2000, 08:35	2	3	4	5	6	8	10.33	12.07
01 Jan 2000, 08:40	2	3	4	5	7	8	9.35	12.43
01 Jan 2000, 08:45	2	3	4	5	7	8	7.82	11.76
01 Jan 2000, 08:50	2	3	4	6	7	8	12.81	9.92
01 Jan 2000, 08:55	2	3	4	6	7	8	10.17	10.65
01 Jan 2000, 09:00	2	3	4	6	7	9	10.35	9.97
01 Jan 2000, 09:05	2	3	4	6	7	9	11.09	10.99
01 Jan 2000, 09:10	3	3	4	6	8	9	10.39	14.08
01 Jan 2000, 09:15	3	4	5	6	8	10	11.89	11.95
01 Jan 2000, 09:20	3	4	5	7	8	10	11.05	14.94
01 Jan 2000, 09:25	3	4	5	7	8	10	11.8	12.21
01 Jan 2000, 09:30	3	4	5	7	8	10	11.9	12.17
01 Jan 2000, 09:35	3	4	5	7	9	11	10.91	11.1

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	3	4	5	7	9.02	11	10.01	17.52
01 Jan 2000, 09:45	3	4	6	8	9.35	12	14.26	19.62
01 Jan 2000, 09:50	3	4	6	8	8.03	12	14.48	19.78
01 Jan 2000, 09:55	3	4	6	8	12.47	12	14.84	17.78
01 Jan 2000, 10:00	3	4	6	8	9.23	12	11.62	17.07
01 Jan 2000, 10:05	3	5	6	8	9.8	12	15.47	17.54
01 Jan 2000, 10:10	3	5	6	9	12.31	13	15.28	18.53
01 Jan 2000, 10:15	3	5	7	8.99	9.48	14	19.73	19.57
01 Jan 2000, 10:20	4	5	7	10.01	13.02	14	16.72	19.76
01 Jan 2000, 10:25	4	6	8	7.97	11.73	15	20.63	21.01
01 Jan 2000, 10:30	4	6	8	11.5	13.75	16	21.78	22.82
01 Jan 2000, 10:35	4	7	9	7.99	11.93	17	20.77	17.08
01 Jan 2000, 10:40	5	7	10	12.75	14.56	19	19.5	17.99
01 Jan 2000, 10:45	5	8	10	14.64	15.52	21	24.36	44.76
01 Jan 2000, 10:50	6	8	11	14.94	18.34	22	25.48	48.9
01 Jan 2000, 10:55	6	9	12	15.1	17.58	24	22.58	38.1
01 Jan 2000, 11:00	7	10	13	15.85	21.22	26	57	35.32
01 Jan 2000, 11:05	7	11	13.97	17.51	20.93	27	0.91	26.77
01 Jan 2000, 11:10	8	12	14.81	20.19	27.55	30	34.32	37.89
01 Jan 2000, 11:15	8	12	15.22	22.1	21.8	31	37.52	37.21
01 Jan 2000, 11:20	9	13	16.09	18.84	26.59	34	28.8	46.26
01 Jan 2000, 11:25	10	15	20.27	21.82	27.47	37	43.42	18.34
01 Jan 2000, 11:30	11	15.76	18.47	23.49	18.35	40	20.38	48.97
01 Jan 2000, 11:35	16	23.37	28.21	37.97	47.13	58	33.44	54.65
01 Jan 2000, 11:40	29	39.7	52.23	67.77	68.58	102	100.52	115.73
01 Jan 2000, 11:45	40.59	58.42	75.06	88.15	90.16	143	89.24	165.07
01 Jan 2000, 11:50	59.69	83.61	104.17	129.82	133.76	206	169.03	149.14
01 Jan 2000, 11:55	96.61	133.38	153.72	157.61	169.23	327	205.76	141.88
01 Jan 2000, 12:00	136.74	197.78	166.62	192.41	152.27	435	119.24	358.66
01 Jan 2000, 12:05	123.26	121.45	116.31	55.29	-32.05	439	278.66	238.74
01 Jan 2000, 12:10	69.72	58.2	-69.79	-81.26	94.78	321	122.65	697.72
01 Jan 2000, 12:15	-40.76	-134.81	-176.76	-27.3	-104.84	205	625.15	693.39
01 Jan 2000, 12:20	-61.85	-139.92	-2.92	-65.41	343.54	143	602.84	649.85
01 Jan 2000, 12:25	-95.83	-13.58	-33.62	279.93	489.61	104	623.29	538.66
01 Jan 2000, 12:30	-10.14	-0.09	35.62	392.2	495.28	81	553.1	501.15
01 Jan 2000, 12:35	27.53	38.12	143.21	425.49	516.09	68	502.56	490.93
01 Jan 2000, 12:40	46.05	19.26	191.35	412.89	523.36	59	481.99	483.79
01 Jan 2000, 12:45	34.15	52.93	207.33	405.27	517.81	53	471.12	481.05
01 Jan 2000, 12:50	33.42	106.36	213.68	375.02	499.79	49	468.43	476.05
01 Jan 2000, 12:55	80.9	129.29	209.88	343.93	474.67	44	467.08	471.43

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	78.33	137.41	208.78	303.62	443.99	41	468.22	469.58
01 Jan 2000, 13:05	41.19	139.05	213.86	297.63	409	38	476.92	471.45
01 Jan 2000, 13:10	54.15	145.05	214.55	288.07	350.66	36	487.87	474.75
01 Jan 2000, 13:15	44.91	118.66	190.54	257.96	319.56	33	498.15	477.36
01 Jan 2000, 13:20	23.97	61.82	155.39	254.78	314.04	32	503.71	489.47
01 Jan 2000, 13:25	44.55	80.88	169.61	253.37	286.2	30	479.04	505.78
01 Jan 2000, 13:30	26.8	71.79	133.81	218.85	262.39	29	448.84	513.87
01 Jan 2000, 13:35	45.75	58.82	81.89	210.01	244.19	27	389.17	516.98
01 Jan 2000, 13:40	43.8	51.27	50.49	174.44	240.69	26	347.53	490.13
01 Jan 2000, 13:45	7.05	32.27	74.85	154.27	231.39	24	327.05	455.99
01 Jan 2000, 13:50	26.49	-45.11	83.22	143.07	204.97	24	293.89	400.9
01 Jan 2000, 13:55	-6.95	26.69	55.38	114.28	178.01	23	274.78	343.28
01 Jan 2000, 14:00	2.47	29.91	53.88	90.6	162.12	22	262.96	317.57
01 Jan 2000, 14:05	-12.38	-2.43	31.84	56.8	125.83	21	248.39	289.34
01 Jan 2000, 14:10	-0.89	65.81	56.47	42.72	103.68	20	224.79	264.39
01 Jan 2000, 14:15	1.63	-5.74	30.11	37.32	60.47	20	221.2	245.57
01 Jan 2000, 14:20	11.85	-1.99	51.6	56.42	51.75	19	172.47	235.23
01 Jan 2000, 14:25	12.97	28.42	57.54	25.76	74.21	18	128.65	219.5
01 Jan 2000, 14:30	14.75	22	58.32	64.26	55.77	18	99.44	168.47
01 Jan 2000, 14:35	8.81	18.68	-12.43	55.44	46.01	17	89.43	153.94
01 Jan 2000, 14:40	12.41	9.98	-9.71	85.69	37.23	16	78.05	125.32
01 Jan 2000, 14:45	10.73	-6.19	-8.45	-1.45	55.25	15	70.15	74.69
01 Jan 2000, 14:50	10.09	13.28	13.21	45.99	28.41	15	53.41	81.3
01 Jan 2000, 14:55	8.82	12.88	-6.9	-12.58	-18.79	15	54.96	72.46
01 Jan 2000, 15:00	8.67	-0.91	-5.78	5.63	-11.92	15	55.81	77.12
01 Jan 2000, 15:05	8.23	6.3	-4.31	10.98	53.34	15	35.01	39.7
01 Jan 2000, 15:10	5.98	16.07	-2.91	17.44	18.35	15	43.16	28.54
01 Jan 2000, 15:15	5.79	6.23	-2.07	32.93	13.71	15	-14.5	40.28
01 Jan 2000, 15:20	5.83	2.12	-1.32	32.03	14.63	15	5.59	61.77
01 Jan 2000, 15:25	3.98	10.87	-1.58	28.73	15.51	14	17.2	49.4
01 Jan 2000, 15:30	4.13	8.76	0.34	27.86	34.6	13	48.18	58.73
01 Jan 2000, 15:35	4.53	8.8	1.74	26.82	33.45	13	-13.89	46.41
01 Jan 2000, 15:40	4.66	8.6	1.96	25.69	32.72	13	40.66	7.16
01 Jan 2000, 15:45	5.35	8.23	1.41	24.02	32.17	13	0.69	14.88
01 Jan 2000, 15:50	4.78	8.15	1.66	23.05	30.1	13	1.61	15.2
01 Jan 2000, 15:55	6.01	8.1	2.1	22.48	29.61	13	46.46	42.97
01 Jan 2000, 16:00	5.36	4.55	2.37	21.98	28.96	13	-7.37	-13.33
01 Jan 2000, 16:05	5.37	4.78	3.53	21.52	28.32	13	26.68	45.27
01 Jan 2000, 16:10	5.18	6.17	3.74	21.08	27.63	13	-10.01	48.58
01 Jan 2000, 16:15	5.04	6.13	3.96	19.49	26.92	13	26.62	-8.53

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	5.04	6.17	4.19	19.22	26.51	13	1.59	35.76
01 Jan 2000, 16:25	5.04	6.56	4.29	18.29	25.82	13	37.6	1.01
01 Jan 2000, 16:30	5.03	7.25	4.41	17.54	25.54	13	24.09	6.55
01 Jan 2000, 16:35	4.03	6.4	4.61	17.39	25.2	12	-5.89	30.51
01 Jan 2000, 16:40	4.01	6.4	4.64	17.22	23.84	12	24.68	-11.61
01 Jan 2000, 16:45	4.03	5.49	4.65	16.01	23.57	11	3.08	3.91
01 Jan 2000, 16:50	4.03	5.92	3.86	15.84	22.25	11	-8.9	28.73
01 Jan 2000, 16:55	4.02	5.78	3.97	15.67	21.97	11	20.93	32.11
01 Jan 2000, 17:00	4.02	4.97	4.08	15.49	21.71	11	20.53	31.71
01 Jan 2000, 17:05	4.02	5.57	4.19	15.33	21.48	11	22.59	42.51
01 Jan 2000, 17:10	4.02	5.55	4.65	15.13	21.16	11	23.04	30.53
01 Jan 2000, 17:15	4.02	5.66	4.88	14.87	20.78	11	22.56	-5.81
01 Jan 2000, 17:20	4.02	5.74	4.98	14.6	19.55	11	22.23	39.86
01 Jan 2000, 17:25	4.02	5.62	5.08	14.35	19.13	11	19.18	6.54
01 Jan 2000, 17:30	4.01	5.59	5.85	14.11	17.64	11	16.47	-5.9
01 Jan 2000, 17:35	4.01	5.58	5.91	13	17.47	11	16.94	28.2
01 Jan 2000, 17:40	4.01	5.56	5.94	12.86	17.23	11	5.36	35.95
01 Jan 2000, 17:45	4.01	5.55	6.27	12.67	17.02	11	30.62	6.44
01 Jan 2000, 17:50	4.01	5.53	6.24	12.51	16.85	10	30.1	4.55
01 Jan 2000, 17:55	3.01	5.51	6.17	11.32	15.56	10	27.18	6.76
01 Jan 2000, 18:00	3.01	4.5	5.24	11.15	15.36	9	24.68	5.75
01 Jan 2000, 18:05	3.01	4.48	5.27	10.97	15.2	9	15.76	5.82
01 Jan 2000, 18:10	3.01	4.46	5.27	9.84	13.95	9	14.22	2.17
01 Jan 2000, 18:15	3.01	4.45	5.29	9.76	13.72	9	16.25	33.18
01 Jan 2000, 18:20	3.01	4.44	5.32	9.6	13.54	9	-2.68	30.99
01 Jan 2000, 18:25	3.01	4.42	5.65	9.46	13.29	9	-1.2	21.41
01 Jan 2000, 18:30	3.01	4.41	4.9	9.27	13.04	9	-1.53	20.77
01 Jan 2000, 18:35	3.01	4.39	5.58	9.1	11.89	9	5.42	-4.21
01 Jan 2000, 18:40	3.01	4.38	5.23	8.94	11.67	9	5.42	25.87
01 Jan 2000, 18:45	3.01	4.37	5.72	8.23	11.44	9	22.55	5.17
01 Jan 2000, 18:50	3.01	4.37	5.41	7.6	11.23	9	21.46	7.12
01 Jan 2000, 18:55	3	4.36	5.4	7.28	11.01	9	14.16	7.07
01 Jan 2000, 19:00	3	4.35	5.39	7.2	10.88	9	13.56	26.14
01 Jan 2000, 19:05	3.01	4.34	5.38	7.22	10.83	9	14.5	26.77
01 Jan 2000, 19:10	3	4.33	5.37	6.81	10.68	9	13.04	16.99
01 Jan 2000, 19:15	3	4.33	5.36	6.86	10.51	9	2.93	-0.23
01 Jan 2000, 19:20	3	4.32	5.35	6.8	10.35	9	7.43	0.88
01 Jan 2000, 19:25	3.01	4.31	5.34	6.76	10.27	9	7.83	6.41
01 Jan 2000, 19:30	3.03	4.31	5.33	6.75	10.19	9	17.83	7.58
01 Jan 2000, 19:35	3	4.3	5.33	6.71	10.13	9	17.64	25.84

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	3	4.3	5.32	6.69	10.09	9	12.89	24.39
01 Jan 2000, 19:45	3	4.29	5.32	6.66	10.02	9	13.22	16.11
01 Jan 2000, 19:50	3	4.29	5.31	6.65	9.95	8	1.5	0.09
01 Jan 2000, 19:55	3	4.28	4.3	6.64	9.68	8	2.13	0.17
01 Jan 2000, 20:00	3	3.29	4.3	5.63	8.39	7	5.79	5.8
01 Jan 2000, 20:05	2	3.27	4.29	5.59	8.28	7	15.34	4.63
01 Jan 2000, 20:10	2	3.27	4.29	5.56	8.22	7	14.13	23.01
01 Jan 2000, 20:15	2	3.26	4.28	5.55	8.14	7	10	13.92
01 Jan 2000, 20:20	2	3.25	4.28	5.51	8.03	6	9.69	14.34
01 Jan 2000, 20:25	2	3.25	4.27	5.47	7.27	6	0.59	-0.53
01 Jan 2000, 20:30	2	3.24	4.27	4.06	7.16	6	1.49	-0.29
01 Jan 2000, 20:35	2	3.24	4.26	6.19	7.19	6	5.53	5.98
01 Jan 2000, 20:40	2	3.23	4.25	5.96	6.84	6	12.59	6.34
01 Jan 2000, 20:45	2	3.23	4.25	5.28	6.81	6	11.42	16.15
01 Jan 2000, 20:50	2	3.22	4.24	5.27	6.74	6	8.66	15.57
01 Jan 2000, 20:55	2	3.22	4.24	5.27	6.68	6	3.36	11
01 Jan 2000, 21:00	2	3.21	4.23	5.26	6.63	6	3.82	2.3
01 Jan 2000, 21:05	2	3.21	4.23	5.25	6.57	6	6.46	2.48
01 Jan 2000, 21:10	2	3.2	4.22	5.25	3.97	6	10.73	6.77
01 Jan 2000, 21:15	2	3.19	4.22	5.24	5.35	6	8.34	6.45
01 Jan 2000, 21:20	2	3.19	4.21	5.24	7.2	6	8.03	14.3
01 Jan 2000, 21:25	2	3.18	4.21	5.23	5.59	6	4.3	11.04
01 Jan 2000, 21:30	2	3.18	4.2	5.23	6.68	6	6.4	10.84
01 Jan 2000, 21:35	2	3.17	4.2	5.22	6.31	6	10.06	3.12
01 Jan 2000, 21:40	2	3.17	4.19	5.22	6.3	6	10.14	7.38
01 Jan 2000, 21:45	2	3.17	4.18	5.21	6.29	6	8.17	7.2
01 Jan 2000, 21:50	2	3.16	4.18	5.2	6.28	6	3.98	12.64
01 Jan 2000, 21:55	2	3.16	4.17	5.2	6.27	6	4.35	9.83
01 Jan 2000, 22:00	2	3.15	4.17	5.19	6.26	6	4.4	4.06
01 Jan 2000, 22:05	2	3.15	4.17	5.19	6.26	6	4.47	7.04
01 Jan 2000, 22:10	2	3.14	4.16	5.18	6.25	6	5.05	7.09
01 Jan 2000, 22:15	2	3.14	4.16	5.18	6.25	6	6.69	12.88
01 Jan 2000, 22:20	2	3.13	4.15	5.17	6.24	6	9.39	12.52
01 Jan 2000, 22:25	2.01	3.13	4.15	5.17	6.23	6	7.91	9.77
01 Jan 2000, 22:30	2	3.13	4.14	5.16	6.23	6	7.63	3.97
01 Jan 2000, 22:35	2	3.12	4.14	5.16	6.22	6	4.99	6.97
01 Jan 2000, 22:40	2	3.12	4.13	5.15	6.21	6	6.65	6.97
01 Jan 2000, 22:45	2	3.11	4.13	5.15	6.2	6	9.35	7.93
01 Jan 2000, 22:50	2	3.11	4.13	5.15	6.2	6	7.86	7.91
01 Jan 2000, 22:55	2	3.1	4.12	5.14	6.2	6	4.94	8.12

TC19								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	2.01	3.1	4.12	5.14	6.19	6	6.53	14
01 Jan 2000, 23:05	2	3.1	4.11	5.13	6.19	6	9.28	11.03
01 Jan 2000, 23:10	2	3.09	4.11	5.13	6.18	6	9.28	4.88
01 Jan 2000, 23:15	2	3.09	4.1	5.12	6.17	6	7.79	7.93
01 Jan 2000, 23:20	2	3.09	4.1	5.12	6.17	6	4.87	8.08
01 Jan 2000, 23:25	2	3.08	4.1	5.12	6.16	6	6.45	13.97
01 Jan 2000, 23:30	2	3.08	4.09	5.11	6.16	6	9.26	11.01
01 Jan 2000, 23:35	2	3.08	4.09	5.11	6.37	6	7.79	4.87
01 Jan 2000, 23:40	2	3.07	4.09	5.1	6.39	6	7.54	7.91
01 Jan 2000, 23:45	2	3.07	4.08	5.1	6.4	6	4.94	8.05
01 Jan 2000, 23:50	2	3.07	4.08	5.1	4.94	6	6.55	12.46
01 Jan 2000, 23:55	2	3.06	3.08	4.09	5.14	5	8.19	8.63
01 Jan 2000, 24:00	2	2.06	3.07	4.09	4.13	5	6.75	2.17

E.2.14 TC-T30

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
31 Dec 1999, 24:00	0	0	0	0	0	0	0	0
01 Jan 2000, 00:05	0	0	0	0	0	0	0	0
01 Jan 2000, 00:10	0	0	0	0	0	0	0	0
01 Jan 2000, 00:15	0	0	0	0	0	0	0	0
01 Jan 2000, 00:20	0	0	0	0	0	0	1	1
01 Jan 2000, 00:25	0	0	0	1	1	1	1	2
01 Jan 2000, 00:30	0	0	1	1	1	1	2	3
01 Jan 2000, 00:35	0	1	1	2	2	2	2	5
01 Jan 2000, 00:40	1	1	1	2	3	3	4	6
01 Jan 2000, 00:45	1	1	2	3	4	4	5	8
01 Jan 2000, 00:50	1	2	2	4	4	5	6	10
01 Jan 2000, 00:55	1	2	3	5	5	6	7	12
01 Jan 2000, 01:00	2	3	3	6	6	7	8	14
01 Jan 2000, 01:05	2	3	4	7	8	9	10	16
01 Jan 2000, 01:10	2	4	4	7	9	10	11	17
01 Jan 2000, 01:15	3	4	5	8	9	11	12	18
01 Jan 2000, 01:20	3	5	6	9	10	12	14	19
01 Jan 2000, 01:25	3	5	6	10	11	13	15	19
01 Jan 2000, 01:30	4	6	7	10	12	14	16	21
01 Jan 2000, 01:35	4	6	7	11	13	15	17	22
01 Jan 2000, 01:40	4	7	8	12	13	15	18	23
01 Jan 2000, 01:45	5	7	9	12	14	16	19	25
01 Jan 2000, 01:50	5	8	9	13	15	17	20	26
01 Jan 2000, 01:55	5	8	10	14	16	18	21	27
01 Jan 2000, 02:00	6	9	10	15	17	19	22	28
01 Jan 2000, 02:05	6	9	11	15	18	20	23	29
01 Jan 2000, 02:10	6	10	12	16	18	21	23	29
01 Jan 2000, 02:15	7	10	12	16	19	21	24	30
01 Jan 2000, 02:20	7	10	13	17	19	22	25	30
01 Jan 2000, 02:25	8	11	13	17	19	22	25	30
01 Jan 2000, 02:30	8	11	13	17	20	23	25	30
01 Jan 2000, 02:35	8	11	14	17	20	23	26	30
01 Jan 2000, 02:40	8	12	14	18	20	23	26	30
01 Jan 2000, 02:45	9	12	14	18	20	23	26	30
01 Jan 2000, 02:50	9	12	14	18	20	23	26	30
01 Jan 2000, 02:55	9	12	14	18	20	23	26	30

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 03:00	9	12	14	18	20	23	26	30
01 Jan 2000, 03:05	9	12	14	18	20	23	26	30
01 Jan 2000, 03:10	9	12	15	18	20	23	26	30
01 Jan 2000, 03:15	9	12	15	18	20	23	26	30
01 Jan 2000, 03:20	10	12	15	18	20	23	26	30
01 Jan 2000, 03:25	10	12	15	18	20	23	26	30
01 Jan 2000, 03:30	10	12	15	18	20	23	26	30
01 Jan 2000, 03:35	10	12	15	18	20	23	26	30
01 Jan 2000, 03:40	10	12	15	18	20	23	26	30
01 Jan 2000, 03:45	10	12	15	18	20	23	26	30
01 Jan 2000, 03:50	10	12	15	18	20	23	26	30
01 Jan 2000, 03:55	10	12	15	18	20	23	26	30
01 Jan 2000, 04:00	10	12	15	18	21	23	26	30
01 Jan 2000, 04:05	10	12	15	18	21	24	27	31
01 Jan 2000, 04:10	10	13	15	18	21	24	27	31
01 Jan 2000, 04:15	10	13	15	18	21	24	27	32
01 Jan 2000, 04:20	10	13	15	19	21	24	27	32
01 Jan 2000, 04:25	10	13	15	19	21	24	28	33
01 Jan 2000, 04:30	10	13	15	19	22	25	28	34
01 Jan 2000, 04:35	10	13	15	19	22	25	28	34
01 Jan 2000, 04:40	10	13	15	20	22	26	29	35
01 Jan 2000, 04:45	10	13	16	20	23	26	29	35
01 Jan 2000, 04:50	10	13	16	20	23	27	30	36
01 Jan 2000, 04:55	10	14	16	21	24	27	30	36
01 Jan 2000, 05:00	10	14	16	21	24	27	31	37
01 Jan 2000, 05:05	10	14	17	21	24	28	31	37
01 Jan 2000, 05:10	11	14	17	21	24	28	31	38
01 Jan 2000, 05:15	11	14	17	22	25	28	32	38
01 Jan 2000, 05:20	11	15	17	22	25	29	32	39
01 Jan 2000, 05:25	11	15	18	22	25	29	33	40
01 Jan 2000, 05:30	11	15	18	22	26	29	33	40
01 Jan 2000, 05:35	11	15	18	23	26	30	33	41
01 Jan 2000, 05:40	11	15	18	23	26	30	34	41
01 Jan 2000, 05:45	12	15	18	23	26	30	34	42
01 Jan 2000, 05:50	12	16	19	23	27	30	35	42
01 Jan 2000, 05:55	12	16	19	23	27	31	35	43
01 Jan 2000, 06:00	12	16	19	24	27	31	35	44
01 Jan 2000, 06:05	12	16	19	24	27	31	35	44
01 Jan 2000, 06:10	12	16	19	24	27	31	36	45
01 Jan 2000, 06:15	12	16	19	24	27	31	36	46

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 06:20	13	16	19	24	27	32	36	47
01 Jan 2000, 06:25	13	16	20	24	28	32	37	48
01 Jan 2000, 06:30	13	17	20	24	28	32	37	50
01 Jan 2000, 06:35	13	17	20	24	28	33	38	52
01 Jan 2000, 06:40	13	17	20	25	28	33	39	54
01 Jan 2000, 06:45	13	17	20	25	29	33	40	56
01 Jan 2000, 06:50	13	17	20	25	29	34	41	58
01 Jan 2000, 06:55	13	17	20	26	30	35	42	60
01 Jan 2000, 07:00	13	17	21	26	30	35	43	62
01 Jan 2000, 07:05	13	17	21	26	30	36	44	64
01 Jan 2000, 07:10	13	18	21	27	31	37	45	66
01 Jan 2000, 07:15	14	18	21	27	31	38	47	68
01 Jan 2000, 07:20	14	18	22	27	32	38	48	70
01 Jan 2000, 07:25	14	18	22	28	32	39	49	72
01 Jan 2000, 07:30	14	19	22	28	33	40	51	74
01 Jan 2000, 07:35	14	19	22	28	33	41	52	76
01 Jan 2000, 07:40	14	19	22	28	34	42	54	78
01 Jan 2000, 07:45	14	19	23	29	34	43	55	80
01 Jan 2000, 07:50	15	19	23	29	35	44	57	81
01 Jan 2000, 07:55	15	19	23	29	35	45	58	83
01 Jan 2000, 08:00	15	20	23	30	36	46	60	84
01 Jan 2000, 08:05	15	20	23	30	37	47	61	85
01 Jan 2000, 08:10	15	20	24	30	37	49	62	87
01 Jan 2000, 08:15	15	20	24	31	38	50	64	88
01 Jan 2000, 08:20	16	20	24	31	39	51	65	90
01 Jan 2000, 08:25	16	20	24	32	40	52	67	92
01 Jan 2000, 08:30	16	21	25	32	41	54	69	95
01 Jan 2000, 08:35	16	21	25	33	42	55	70	98
01 Jan 2000, 08:40	16	21	25	34	43	57	73	102
01 Jan 2000, 08:45	16	21	26	35	44	59	75	106
01 Jan 2000, 08:50	17	21	26	35	46	61	78	111
01 Jan 2000, 08:55	17	22	26	37	47	63	81	116
01 Jan 2000, 09:00	17	22	27	38	49	66	84	120
01 Jan 2000, 09:05	17	22	27	39	51	68	88	125
01 Jan 2000, 09:10	17	22	27	40	53	71	91	130
01 Jan 2000, 09:15	17	23	28	42	55	74	95	135
01 Jan 2000, 09:20	18	23	29	43	58	77	99	140
01 Jan 2000, 09:25	18	24	29	45	60	80	103	145
01 Jan 2000, 09:30	18	24	30	47	62	83	107	151
01 Jan 2000, 09:35	18	25	31	49	65	87	112	156

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 09:40	19	25	32	51	67	91	116	162
01 Jan 2000, 09:45	19	26	33	53	70	95	121	167
01 Jan 2000, 09:50	20	27	34	55	74	99	126	171
01 Jan 2000, 09:55	20	27	35	58	77	103	131	177
01 Jan 2000, 10:00	20	28	36	60	80	107	135	183
01 Jan 2000, 10:05	21	29	37	63	83	110	139	190
01 Jan 2000, 10:10	21	29	38	65	87	114	145	196
01 Jan 2000, 10:15	22	30	40	68	90	119	150	202
01 Jan 2000, 10:20	22	31	41	71	94	124	156	208
01 Jan 2000, 10:25	23	32	43	73	97	128	161	214
01 Jan 2000, 10:30	23	33	44	77	101	133	167	222
01 Jan 2000, 10:35	24	34	46	80	106	139	173	231
01 Jan 2000, 10:40	24	35	49	84	110	145	181	241
01 Jan 2000, 10:45	25	36	51	88	116	152	190	253
01 Jan 2000, 10:50	26	38	53	93	122	160	200	268
01 Jan 2000, 10:55	27	39	56	98	130	170	211	285
01 Jan 2000, 11:00	28	41	60	105	138	181	224	306
01 Jan 2000, 11:05	29	43	63	112	148	194	240	329
01 Jan 2000, 11:10	30	46	67	121	160	209	258	356
01 Jan 2000, 11:15	31	48	72	131	174	226	278	385
01 Jan 2000, 11:20	32	51	77	143	190	244	301	417
01 Jan 2000, 11:25	33	55	84	156	207	265	327	452
01 Jan 2000, 11:30	35	59	91	172	225	288	355	489
01 Jan 2000, 11:35	36	64	101	189	246	314	388	530
01 Jan 2000, 11:40	38	70	112	211	273	348	433	589
01 Jan 2000, 11:45	41	80	132	244	314	402	509	685
01 Jan 2000, 11:50	48	98	163	293	372	500	666	905
01 Jan 2000, 11:55	58	126	210	378	512	781	1084	1470
01 Jan 2000, 12:00	76	172	301	647	1037	1461	1874	2452
01 Jan 2000, 12:05	108	300	653	1373	1877	2446	2975	3732
01 Jan 2000, 12:10	180	713	1400	2188	2747	3358	3973	4777
01 Jan 2000, 12:15	438	1354	1927	2670	3171	3743	4326	5110
01 Jan 2000, 12:20	886	1578	2043	2630	3048	3532	4015	4675
01 Jan 2000, 12:25	1093	1531	1846	2304	2645	3040	3428	3939
01 Jan 2000, 12:30	1030	1357	1622	1989	2265	2605	2932	3357
01 Jan 2000, 12:35	951	1221	1462	1804	2053	2367	2662	3052
01 Jan 2000, 12:40	870	1146	1380	1729	1987	2292	2572	2946
01 Jan 2000, 12:45	809	1091	1339	1699	1970	2275	2558	2920
01 Jan 2000, 12:50	735	1011	1261	1630	1904	2210	2504	2882
01 Jan 2000, 12:55	641	881	1114	1466	1730	2029	2324	2728

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 13:00	541	731	924	1217	1452	1728	2004	2402
01 Jan 2000, 13:05	455	590	739	960	1144	1375	1611	1958
01 Jan 2000, 13:10	391	491	593	754	896	1066	1244	1511
01 Jan 2000, 13:15	344	428	504	624	728	861	985	1177
01 Jan 2000, 13:20	311	387	456	563	639	742	844	989
01 Jan 2000, 13:25	285	355	418	513	588	675	765	894
01 Jan 2000, 13:30	263	328	385	468	535	616	701	816
01 Jan 2000, 13:35	245	306	359	440	499	569	644	749
01 Jan 2000, 13:40	229	287	337	410	468	534	600	692
01 Jan 2000, 13:45	215	270	317	386	438	501	563	644
01 Jan 2000, 13:50	202	256	300	366	415	473	531	604
01 Jan 2000, 13:55	191	242	284	345	393	448	504	570
01 Jan 2000, 14:00	181	230	270	328	374	426	478	539
01 Jan 2000, 14:05	172	218	257	311	356	406	455	511
01 Jan 2000, 14:10	164	208	245	295	339	387	434	487
01 Jan 2000, 14:15	157	198	234	282	324	371	415	465
01 Jan 2000, 14:20	151	189	223	269	309	356	399	446
01 Jan 2000, 14:25	144	180	214	258	297	342	384	428
01 Jan 2000, 14:30	139	172	206	247	285	328	370	412
01 Jan 2000, 14:35	134	165	198	238	274	316	356	397
01 Jan 2000, 14:40	129	159	190	229	264	304	344	384
01 Jan 2000, 14:45	124	153	183	220	254	293	332	370
01 Jan 2000, 14:50	120	148	176	212	245	283	320	357
01 Jan 2000, 14:55	116	143	170	205	237	273	309	345
01 Jan 2000, 15:00	113	138	165	198	229	264	299	332
01 Jan 2000, 15:05	109	134	159	191	221	255	289	321
01 Jan 2000, 15:10	106	130	155	185	214	247	279	311
01 Jan 2000, 15:15	103	126	150	179	207	239	271	303
01 Jan 2000, 15:20	100	122	146	174	201	232	263	296
01 Jan 2000, 15:25	97	119	142	170	196	227	257	290
01 Jan 2000, 15:30	95	116	138	166	191	221	251	285
01 Jan 2000, 15:35	92	113	134	162	187	217	246	281
01 Jan 2000, 15:40	89	110	131	159	183	213	242	276
01 Jan 2000, 15:45	87	107	128	156	180	209	238	271
01 Jan 2000, 15:50	85	105	126	153	177	205	234	266
01 Jan 2000, 15:55	83	103	123	150	174	202	229	261
01 Jan 2000, 16:00	81	101	121	148	171	198	225	257
01 Jan 2000, 16:05	79	99	119	145	168	195	222	253
01 Jan 2000, 16:10	77	97	117	143	166	192	219	250
01 Jan 2000, 16:15	76	95	115	141	164	190	216	246

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 16:20	75	94	113	140	162	187	213	243
01 Jan 2000, 16:25	73	92	112	138	160	185	210	241
01 Jan 2000, 16:30	72	91	111	137	158	183	208	240
01 Jan 2000, 16:35	71	90	109	135	156	181	206	239
01 Jan 2000, 16:40	70	89	108	134	155	180	205	239
01 Jan 2000, 16:45	69	88	107	133	154	179	204	238
01 Jan 2000, 16:50	67	87	106	132	153	178	203	237
01 Jan 2000, 16:55	66	86	105	131	152	177	202	234
01 Jan 2000, 17:00	65	85	104	130	151	176	201	231
01 Jan 2000, 17:05	65	84	103	129	150	174	198	226
01 Jan 2000, 17:10	64	83	102	128	148	171	195	222
01 Jan 2000, 17:15	63	83	101	126	146	169	192	218
01 Jan 2000, 17:20	62	82	100	124	144	166	189	215
01 Jan 2000, 17:25	62	81	99	122	142	164	186	212
01 Jan 2000, 17:30	61	80	98	121	140	161	183	208
01 Jan 2000, 17:35	61	79	97	119	138	159	180	205
01 Jan 2000, 17:40	60	79	96	118	136	157	178	203
01 Jan 2000, 17:45	60	78	95	116	134	155	175	201
01 Jan 2000, 17:50	59	77	94	115	133	153	174	201
01 Jan 2000, 17:55	58	76	93	114	131	152	172	200
01 Jan 2000, 18:00	58	75	92	113	130	151	171	199
01 Jan 2000, 18:05	57	74	90	111	129	149	170	198
01 Jan 2000, 18:10	56	73	89	110	128	148	169	195
01 Jan 2000, 18:15	56	72	88	109	127	147	167	192
01 Jan 2000, 18:20	55	72	87	108	125	145	165	188
01 Jan 2000, 18:25	54	71	86	107	124	143	162	183
01 Jan 2000, 18:30	54	70	85	105	122	141	159	180
01 Jan 2000, 18:35	53	69	84	104	120	138	156	176
01 Jan 2000, 18:40	53	69	83	102	118	136	154	173
01 Jan 2000, 18:45	52	68	82	100	116	134	151	170
01 Jan 2000, 18:50	52	67	81	99	114	131	148	167
01 Jan 2000, 18:55	51	66	80	98	112	129	146	165
01 Jan 2000, 19:00	50	65	79	96	111	127	144	163
01 Jan 2000, 19:05	50	64	78	95	109	125	142	162
01 Jan 2000, 19:10	49	64	77	94	108	124	140	161
01 Jan 2000, 19:15	49	63	76	92	106	123	139	160
01 Jan 2000, 19:20	48	62	75	91	105	122	138	160
01 Jan 2000, 19:25	48	61	74	91	105	121	138	160
01 Jan 2000, 19:30	47	61	73	90	104	121	137	160
01 Jan 2000, 19:35	47	60	73	90	104	120	137	160

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 19:40	46	59	72	89	104	120	137	160
01 Jan 2000, 19:45	46	59	71	89	103	120	137	160
01 Jan 2000, 19:50	45	58	71	89	103	120	137	160
01 Jan 2000, 19:55	45	58	71	89	103	120	137	160
01 Jan 2000, 20:00	44	57	70	89	103	120	136	159
01 Jan 2000, 20:05	44	57	70	88	102	119	136	158
01 Jan 2000, 20:10	43	56	69	88	102	118	135	156
01 Jan 2000, 20:15	42	56	69	87	101	118	134	153
01 Jan 2000, 20:20	42	56	68	86	100	116	132	149
01 Jan 2000, 20:25	42	55	68	85	99	114	130	145
01 Jan 2000, 20:30	41	55	68	84	97	112	127	142
01 Jan 2000, 20:35	41	55	67	83	95	110	124	139
01 Jan 2000, 20:40	41	54	67	81	94	108	122	136
01 Jan 2000, 20:45	41	54	66	80	92	106	119	133
01 Jan 2000, 20:50	40	53	65	78	90	104	117	130
01 Jan 2000, 20:55	40	53	64	77	89	102	115	127
01 Jan 2000, 21:00	40	52	63	76	87	100	113	125
01 Jan 2000, 21:05	39	51	62	75	86	98	111	124
01 Jan 2000, 21:10	39	51	61	74	84	97	109	122
01 Jan 2000, 21:15	39	50	60	72	83	95	107	122
01 Jan 2000, 21:20	38	49	60	71	82	94	106	121
01 Jan 2000, 21:25	38	49	59	70	81	93	105	121
01 Jan 2000, 21:30	37	48	58	70	80	92	104	120
01 Jan 2000, 21:35	37	47	57	69	79	92	104	120
01 Jan 2000, 21:40	36	46	56	68	79	91	104	120
01 Jan 2000, 21:45	36	46	56	68	78	91	103	120
01 Jan 2000, 21:50	35	45	55	68	78	91	103	120
01 Jan 2000, 21:55	35	45	54	67	78	91	103	120
01 Jan 2000, 22:00	34	44	54	67	78	90	103	120
01 Jan 2000, 22:05	34	44	54	67	78	90	103	120
01 Jan 2000, 22:10	34	44	53	67	78	90	103	120
01 Jan 2000, 22:15	33	43	53	67	78	90	103	120
01 Jan 2000, 22:20	33	43	53	67	78	90	103	120
01 Jan 2000, 22:25	33	43	53	67	78	90	103	120
01 Jan 2000, 22:30	32	43	53	67	78	90	103	120
01 Jan 2000, 22:35	32	43	53	67	78	90	103	120
01 Jan 2000, 22:40	32	43	53	67	78	90	103	120
01 Jan 2000, 22:45	32	43	53	67	78	90	103	120
01 Jan 2000, 22:50	32	43	53	67	78	90	103	120
01 Jan 2000, 22:55	32	43	53	67	78	90	103	120

TC-T30								
Timestamp	Flow (cfs)							
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
01 Jan 2000, 23:00	32	43	53	67	78	90	103	120
01 Jan 2000, 23:05	31	43	53	67	78	90	103	120
01 Jan 2000, 23:10	31	43	53	67	78	90	103	120
01 Jan 2000, 23:15	31	43	53	67	78	90	103	120
01 Jan 2000, 23:20	31	43	53	67	78	90	103	120
01 Jan 2000, 23:25	31	43	53	67	78	90	103	120
01 Jan 2000, 23:30	31	43	53	67	78	91	103	120
01 Jan 2000, 23:35	31	43	53	67	78	91	103	120
01 Jan 2000, 23:40	31	43	53	67	78	91	103	120
01 Jan 2000, 23:45	31	43	53	67	78	91	103	120
01 Jan 2000, 23:50	31	43	53	67	78	91	103	120
01 Jan 2000, 23:55	31	43	53	67	78	90	103	120
01 Jan 2000, 24:00	31	43	53	67	78	90	103	120

APPENDIX E.3 – HEC-FDA H&H INPUT DATA

Introduction

50% Design Inputs

Post-35% Design Inputs

E.3 HEC-FDA H&H INPUT DATA

E.3.1 Introduction

The following tables include H&H data provided for the HEC-FDA levee profile update. This data includes standard deviation calculations, peak flows, and water surface elevations at each index location for the FWOP and with design conditions.

E.3.2 50% Design Inputs

E.3.2.1 Standard Deviation Calculations

Standard deviation was computed by taking the average water surface elevation difference along the entire project length between the +/-30% Manning's N value profiles for each frequency event. This is one of the methods to develop stage-uncertainty in the absence of observed data as stated in EM-1110-2-1619. The computed standard deviation serves as the uncertainty bounds for the stage-discharge rating curve as shown previously in Appendix E.1. This was done for the FWOP and with design conditions. Due to the very large amount of data, the water surface profiles were not included in the appendix. As shown below, the standard deviation for each frequency was computed, but only the max was used for HEC-FDA.

Table 1. Standard Deviations – FWOP Condition.

AEP	Average WSE Difference (feet)	Computed StDev	StDev for FDA (max)
50.0%	1.84	0.460	0.460
20.0%	1.46	0.364	0.460
10.0%	1.72	0.429	0.460
5.0%	1.19	0.298	0.460
2.0%	1.02	0.254	0.460
1.0%	0.85	0.213	0.460
0.5%	0.74	0.185	0.460
0.2%	0.62	0.154	0.460

Table 2. Standard Deviations – With Project Condition.

AEP	Average WSE Difference (feet)	Computed StDev	StDev for FDA (max)
50.0%	1.87	0.468	0.468
20.0%	1.67	0.417	0.468
10.0%	1.66	0.414	0.468
5.0%	1.59	0.397	0.468
2.0%	1.53	0.382	0.468
1.0%	1.32	0.329	0.468
0.5%	0.93	0.234	0.468
0.2%	0.54	0.136	0.468

E.3.2.2 Peak Flows – FWOP Condition

Feasibility River Mile	Channel STA (feet)	Peak Flows (cfs)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	6,974	8,923	10,319	13,152	15,380	17,720	20,038	23,179
2.623	64+14 LB	6,978	8,921	10,320	13,154	15,389	17,718	20,055	23,186
2.623	64+14 RB	6,978	8,921	10,320	13,154	15,389	17,718	20,055	23,186
2.651	62+70 LB	6,982	8,919	10,323	13,172	15,404	17,715	20,072	23,191
2.651	62+70 RB	6,982	8,919	10,323	13,172	15,404	17,715	20,072	23,191
-	58+28 LB	6,991	8,914	10,350	13,202	15,428	17,716	20,095	23,203
-	58+28 RB	6,991	8,914	10,350	13,202	15,428	17,716	20,095	23,203
2.813	54+18 LB	7,000	8,949	10,392	13,241	15,455	17,746	20,114	23,213
2.813	54+18 RB	7,000	8,949	10,392	13,241	15,455	17,746	20,114	23,213
-	50+96 LB	6,915	8,943	10,346	13,164	15,330	17,598	19,933	22,975
-	50+96 RB	6,915	8,943	10,346	13,164	15,330	17,598	19,933	22,975
-	45+86 LB	6,786	8,756	10,375	13,257	15,361	17,426	19,755	22,672
-	45+86 RB	6,786	8,756	10,375	13,257	15,361	17,426	19,755	22,672
3.04	42+30 LB	6,789	8,766	10,464	13,413	15,539	17,484	19,877	22,742
3.04	42+30 RB	6,789	8,766	10,464	13,413	15,539	17,484	19,877	22,742
3.07	40+76 LB	6,788	8,774	10,477	13,445	15,603	17,530	19,909	22,757
3.07	40+76 RB	6,788	8,774	10,477	13,445	15,603	17,530	19,909	22,757
-	34+88 LB	6,292	8,077	9,697	12,693	14,620	16,606	18,670	21,409
-	34+88 RB	6,292	8,077	9,697	12,693	14,620	16,606	18,670	21,409
3.298	28+82 LB	6,290	8,097	9,736	12,752	14,707	16,768	18,618	21,386
3.298	28+82 RB	6,290	8,097	9,736	12,752	14,707	16,768	18,618	21,386
3.345	26+38 LB	6,290	8,093	9,746	12,792	14,751	16,824	18,769	21,388
3.345	26+38 RB	6,290	8,093	9,746	12,792	14,751	16,824	18,769	21,388
3.481	19+20 LB	6,276	8,094	9,762	12,850	14,768	16,862	18,843	21,388
3.481	19+20 RB	6,276	8,094	9,762	12,850	14,768	16,862	18,843	21,388
3.539	16+16 LB	6,284	8,092	9,764	12,850	14,777	16,865	18,865	21,393
3.539	16+16 RB	6,284	8,092	9,764	12,850	14,777	16,865	18,865	21,393
-	13+44 LB	6,276	8,088	9,768	12,845	14,785	16,853	18,701	21,296
-	13+44 RB	6,276	8,088	9,768	12,845	14,785	16,853	18,701	21,296
-	3.726	4,377	5,668	6,619	8,038	9,353	10,768	12,124	14,042
3.665	9+54 LB	6,272	8,108	9,792	12,860	14,789	16,854	18,738	21,436
3.665	9+54 RB	6,272	8,108	9,792	12,860	14,789	16,854	18,738	21,436
-	8+00 LB	2,629	3,612	4,397	5,405	6,318	7,134	8,278	9,337
-	6+00 LB	2,649	3,637	4,445	5,403	6,194	6,974	7,784	8,725
-	4+00 LB	2,669	3,652	4,410	5,439	6,194	6,993	7,824	8,763
-	2+00 LB	2,677	3,662	4,409	5,447	6,197	7,015	7,861	8,752
-	0+40	2,677	3,662	4,409	5,447	6,197	7,015	7,861	8,752

E.3.2.3 Peak Water Surface Elevations – FWOP Condition

Feasibility River Mile	Channel STA (feet)	Peak Water Surface Elevations (feet, NAVD88)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	900.16	901.92	903.08	905.38	906.88	908.13	909.37	910.50
2.623	64+14 LB	901.28	903.27	904.56	907.08	908.49	909.65	910.58	911.53
2.623	64+14 RB	901.12	903.14	904.41	906.87	908.56	909.64	910.53	911.47
2.651	62+70 LB	902.04	904.19	905.68	908.96	910.46	911.44	912.27	913.15
2.651	62+70 RB	901.30	903.27	904.79	908.60	909.99	911.04	911.95	912.74
-	58+28 LB	902.93	905.19	906.52	909.60	910.90	911.82	912.62	913.54
-	58+28 RB	903.05	905.19	906.59	909.52	910.77	911.68	912.47	913.39
2.813	54+18 LB	905.68	907.90	909.14	911.83	913.00	913.85	914.59	915.44
2.813	54+18 RB	905.48	907.81	908.85	911.68	912.94	913.81	914.56	915.42
-	50+96 LB	906.46	908.72	909.93	912.31	913.39	914.22	914.96	915.83
-	50+96 RB	906.65	908.78	909.96	912.28	913.34	914.16	914.91	915.79
-	45+86 LB	907.68	909.74	910.71	912.64	913.59	914.37	915.09	915.93
-	45+86 RB	907.67	909.64	910.59	912.57	913.53	914.31	915.03	915.86
3.04	42+30 LB	908.94	910.91	911.71	913.11	913.97	914.72	915.43	916.25
3.04	42+30 RB	909.05	911.07	911.90	913.37	914.25	915.05	915.78	916.64
3.07	40+76 LB	909.28	911.28	912.12	913.48	914.29	915.76	916.83	917.26
3.07	40+76 RB	909.35	911.38	912.20	913.61	914.45	915.98	917.07	917.50
-	34+88 LB	910.63	912.33	913.18	914.56	915.36	916.52	917.48	918.03
-	34+88 RB	910.48	912.13	913.13	914.46	915.24	916.38	917.33	917.86
3.298	28+82 LB	912.00	913.88	914.90	916.45	917.47	918.36	919.07	919.82
3.298	28+82 RB	912.00	913.88	914.85	916.39	917.33	918.28	918.97	919.72
3.345	26+38 LB	913.72	915.37	916.44	917.91	918.66	919.38	919.98	920.71
3.345	26+38 RB	913.22	914.78	915.77	916.97	917.71	918.40	919.12	919.92
3.481	19+20 LB	919.24	920.48	921.99	923.58	924.17	924.66	925.15	925.73
3.481	19+20 RB	919.25	920.48	922.03	923.39	924.07	924.66	925.12	925.63
3.539	16+16 LB	921.39	922.94	924.75	925.79	926.22	926.60	926.99	928.00
3.539	16+16 RB	921.54	923.14	924.52	925.96	926.42	926.92	927.56	928.91
-	13+44 LB	922.72	924.94	926.29	927.79	928.64	929.3	929.97	931.96
-	13+44 RB	922.83	924.13	925.76	927.71	928.64	929.53	930.42	932.57
-	3.726	931.15	933.29	934.81	935.62	936.02	936.43	936.82	937.4
3.665	9+54 LB	927.55	929.72	931.13	932.66	933.43	934.19	934.8	935.89
3.665	9+54 RB	926.52	928.94	930.82	932.46	933.14	933.96	934.68	935.83
-	8+00 LB	928.43	930.73	932.04	933.34	933.96	934.62	935.35	936.39
-	6+00 LB	928.79	930.94	931.96	933.24	933.89	934.56	935.31	936.37
-	4+00 LB	929.91	931.71	932.69	933.85	934.55	935.27	935.9	936.75
-	2+00 LB	931.06	932.82	933.85	935	935.73	936.37	936.89	937.51
-	0+40	932.31	933.99	935.18	936.26	937.02	937.57	938.01	938.72

E.3.2.4 Peak Flows – 50% Design

Feasibility River Mile	Channel STA (feet)	Peak Flows (cfs)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	6,818	9,184	10,925	14,273	16,499	18,677	20,376	21,861
2.623	64+14 LB	6,823	9,185	10,930	14,268	16,502	18,674	20,405	21,879
2.623	64+14 RB	6,823	9,185	10,930	14,268	16,502	18,674	20,405	21,879
2.651	62+70 LB	6,830	9,186	10,935	14,260	16,505	18,682	20,451	21,886
2.651	62+70 RB	6,830	9,186	10,935	14,260	16,505	18,682	20,451	21,886
-	58+28 LB	6,847	9,192	10,946	14,286	16,520	18,714	20,547	21,904
-	58+28 RB	6,847	9,192	10,946	14,286	16,520	18,714	20,547	21,904
2.813	54+18 LB	6,859	9,195	10,948	14,304	16,531	18,745	20,634	22,004
2.813	54+18 RB	6,859	9,195	10,948	14,304	16,531	18,745	20,634	22,004
-	50+96 LB	6,780	9,059	10,783	14,109	16,296	18,424	20,285	21,962
-	50+96 RB	6,780	9,059	10,783	14,109	16,296	18,424	20,285	21,962
-	45+86 LB	6,662	8,856	10,554	13,857	15,979	18,012	19,798	21,992
-	45+86 RB	6,662	8,856	10,554	13,857	15,979	18,012	19,798	21,992
3.04	42+30 LB	6,667	8,886	10,555	13,869	15,983	18,031	19,836	22,242
3.04	42+30 RB	6,667	8,886	10,555	13,869	15,983	18,031	19,836	22,242
3.07	40+76 LB	6,670	8,898	10,552	13,874	15,985	18,040	19,840	22,266
3.07	40+76 RB	6,670	8,898	10,552	13,874	15,985	18,040	19,840	22,266
-	34+88 LB	6,173	8,191	9,734	12,833	14,765	16,605	18,232	21,267
-	34+88 RB	6,173	8,191	9,734	12,833	14,765	16,605	18,232	21,267
3.298	28+82 LB	6,165	8,153	9,709	12,835	14,774	16,579	18,194	21,288
3.298	28+82 RB	6,165	8,153	9,709	12,835	14,774	16,579	18,194	21,288
3.345	26+38 LB	6,165	8,145	9,699	12,836	14,781	16,566	18,322	21,321
3.345	26+38 RB	6,165	8,145	9,699	12,836	14,781	16,566	18,322	21,321
3.481	19+20 LB	6,156	8,147	9,705	12,870	14,784	16,575	18,377	21,350
3.481	19+20 RB	6,156	8,147	9,705	12,870	14,784	16,575	18,377	21,350
3.539	16+16 LB	6,150	8,148	9,709	12,883	14,779	16,579	18,358	21,362
3.539	16+16 RB	6,150	8,148	9,709	12,883	14,779	16,579	18,358	21,362
-	13+44 LB	6,150	8,151	9,711	12,885	14,772	16,595	18,392	21,372
-	13+44 RB	6,150	8,151	9,711	12,885	14,772	16,595	18,392	21,372
-	3.726	4,240	5,542	6,646	8,086	9,369	10,875	12,649	14,436
3.665	9+54 LB	6,159	8,177	9,770	12,881	14,760	16,605	18,486	21,369
3.665	9+54 RB	6,159	8,177	9,770	12,881	14,760	16,605	18,486	21,369
-	8+00 LB	2,630	3,621	4,318	4,743	5,319	5,889	6,269	7,804
-	6+00 LB	2,646	3,635	4,410	5,330	6,141	6,915	7,774	8,758
-	4+00 LB	2,648	3,648	4,414	5,365	6,157	6,957	7,792	8,798
-	2+00 LB	2,662	3,672	4,515	5,385	6,198	7,047	7,814	8,863
-	0+40	2,662	3,672	4,515	5,385	6,198	7,047	7,814	8,863

E.3.2.5 Peak Water Surface Elevations – 50% Design

Feasibility River Mile	Channel STA (feet)	Peak Water Surface Elevations (feet, NAVD88)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	900.63	903.10	904.70	907.08	908.40	909.58	910.39	911.44
2.623	64+14 LB	901.94	904.12	905.57	907.88	909.20	910.38	911.09	911.48
2.623	64+14 RB	902.42	904.76	906.35	908.37	909.71	911.15	912.29	912.60
2.651	62+70 LB	903.07	905.54	907.17	908.74	910.13	911.68	913.27	913.52
2.651	62+70 RB	902.82	905.18	906.77	908.22	909.55	911.05	9912.53	912.84
-	58+28 LB	903.25	905.71	907.26	909.24	910.73	911.95	913.32	913.55
-	58+28 RB	903.41	905.84	907.28	909.95	910.75	911.83	913.46	913.71
2.813	54+18 LB	905.82	908.37	910.05	912.27	913.74	914.94	916.03	916.31
2.813	54+18 RB	905.64	908.21	909.94	912.14	913.41	914.48	915.45	915.68
-	50+96 LB	906.49	909.11	910.76	913.01	914.57	915.94	917.20	917.56
-	50+96 RB	906.66	909.14	910.70	912.89	914.42	915.77	917.03	917.40
-	45+86 LB	907.29	909.55	911.18	913.37	914.80	916.07	917.26	917.57
-	45+86 RB	907.11	909.82	911.08	913.06	914.44	915.68	916.90	917.67
3.04	42+30 LB	908.58	910.91	912.51	914.70	916.11	917.37	918.46	918.69
3.04	42+30 RB	908.69	911.08	912.68	914.92	916.38	917.69	918.84	919.09
3.07	40+76 LB	908.83	911.27	912.85	915.07	916.89	918.43	919.46	919.77
3.07	40+76 RB	908.95	911.40	912.97	915.22	917.02	918.58	919.67	920.10
-	34+88 LB	910.10	912.17	913.56	915.66	917.28	918.81	919.80	920.09
-	34+88 RB	909.91	911.98	913.32	915.29	916.89	918.41	919.32	919.64
3.298	28+82 LB	911.66	913.83	915.16	917.52	919.04	920.39	921.35	921.56
3.298	28+82 RB	911.59	913.71	915.08	917.41	918.97	920.38	921.38	921.75
3.345	26+38 LB	913.60	915.33	916.69	918.59	919.79	921.02	922.06	922.64
3.345	26+38 RB	913.07	914.64	915.88	917.81	919.04	920.28	921.23	921.70
3.481	19+20 LB	919.60	920.98	921.73	922.91	922.38	923.93	924.90	925.86
3.481	19+20 RB	919.58	920.95	921.79	923.00	923.55	924.09	925.03	926.49
3.539	16+16 LB	921.82	923.37	924.46	926.14	927.00	927.81	928.42	929.25
3.539	16+16 RB	921.85	923.43	924.56	926.36	927.19	928.12	928.86	929.56
-	13+44 LB	923.46	925.14	926.25	927.58	928.18	930.46	931.72	932.88
-	13+44 RB	923.66	925.19	926.37	927.98	928.68	930.96	932.21	933.20
-	3.726	931.80	933.49	934.86	935.86	936.25	936.86	937.54	938.15
3.665	9+54 LB	927.65	929.95	932.10	933.89	934.59	935.66	936.56	937.36
3.665	9+54 RB	927.06	929.25	931.94	934.65	934.77	935.29	936.22	936.87
-	8+00 LB	928.38	930.58	932.73	934.47	935.16	936.16	937.02	937.83
-	6+00 LB	928.61	930.74	932.83	934.59	935.31	936.31	937.17	937.99
-	4+00 LB	929.73	931.54	933.15	934.83	935.53	936.49	937.33	937.90
-	2+00 LB	930.93	932.46	933.78	935.23	935.87	936.75	937.55	938.14
-	0+40	932.19	933.75	935.13	936.52	937.26	938.12	938.99	939.74

E.3.3 Post-35% Design Inputs

E.3.3.1 Standard Deviation Calculations

Standard deviation was computed by taking the average water surface elevation difference along the entire project length between the +/-30% Manning's N value profiles for each frequency event. This is one of the methods to develop stage-uncertainty in the absence of observed data as stated in EM-1110-2-1619. The computed standard deviation serves as the uncertainty bounds for the stage-discharge rating curve as shown previously in Appendix E.1. This was done for the FWOP and with design conditions. Due to the very large amount of data, the water surface profiles were not included in the appendix. As shown below, the standard deviation for each frequency was computed, but only the max was used for HEC-FDA.

Table 3. Standard Deviations – FWOP Condition.

AEP	Average WSE Difference (feet)	Computed StDev	StDev for FDA (max)
50.0%	1.84	0.460	0.460
20.0%	1.46	0.364	0.460
10.0%	1.72	0.429	0.460
5.0%	1.19	0.298	0.460
2.0%	1.02	0.254	0.460
1.0%	0.85	0.213	0.460
0.5%	0.74	0.185	0.460
0.2%	0.62	0.154	0.460

Table 4. Standard Deviations – With Project Condition.

AEP	Average WSE Difference (feet)	Computed StDev	StDev for FDA (max)
50.0%	1.87	0.468	0.468
20.0%	1.67	0.417	0.468
10.0%	1.66	0.414	0.468
5.0%	1.59	0.397	0.468
2.0%	1.53	0.382	0.468
1.0%	1.32	0.329	0.468
0.5%	0.93	0.234	0.468
0.2%	0.54	0.136	0.468

E.3.3.2 Peak Flows – FWOP Condition

Feasibility River Mile	Channel STA (feet)	Peak Flows (cfs)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	1,500	6,974	8,923	10,319	13,152	15,380	17,720	20,038
2.623	64+14 LB	1,500	6,978	8,921	10,320	13,154	15,389	17,718	20,055
2.623	64+14 RB	1,500	6,978	8,921	10,320	13,154	15,389	17,718	20,055
2.651	62+70 LB	1,500	6,982	8,919	10,323	13,172	15,404	17,715	20,072
2.651	62+70 RB	1,500	6,982	8,919	10,323	13,172	15,404	17,715	20,072
-	58+28 LB	1,500	6,991	8,914	10,350	13,202	15,428	17,716	20,095
-	58+28 RB	1,500	6,991	8,914	10,350	13,202	15,428	17,716	20,095
2.813	54+18 LB	1,500	7,000	8,949	10,392	13,241	15,455	17,746	20,114
2.813	54+18 RB	1,500	7,000	8,949	10,392	13,241	15,455	17,746	20,114
-	50+96 LB	1,500	6,915	8,943	10,346	13,164	15,330	17,598	19,933
-	50+96 RB	1,500	6,915	8,943	10,346	13,164	15,330	17,598	19,933
-	45+86 LB	1,500	6,786	8,756	10,375	13,257	15,361	17,426	19,755
-	45+86 RB	1,500	6,786	8,756	10,375	13,257	15,361	17,426	19,755
3.04	42+30 LB	1,500	6,789	8,766	10,464	13,413	15,539	17,484	19,877
3.04	42+30 RB	1,500	6,789	8,766	10,464	13,413	15,539	17,484	19,877
3.07	40+76 LB	1,500	6,788	8,774	10,477	13,445	15,603	17,530	19,909
3.07	40+76 RB	1,500	6,788	8,774	10,477	13,445	15,603	17,530	19,909
-	34+88 LB	1,500	6,292	8,077	9,697	12,693	14,620	16,606	18,670
-	34+88 RB	1,500	6,292	8,077	9,697	12,693	14,620	16,606	18,670
3.298	28+82 LB	1,500	6,290	8,097	9,736	12,752	14,707	16,768	18,618
3.298	28+82 RB	1,500	6,290	8,097	9,736	12,752	14,707	16,768	18,618
3.345	26+38 LB	1,500	6,290	8,093	9,746	12,792	14,751	16,824	18,769
3.345	26+38 RB	1,500	6,290	8,093	9,746	12,792	14,751	16,824	18,769
3.481	19+20 LB	1,500	6,276	8,094	9,762	12,850	14,768	16,862	18,843
3.481	19+20 RB	1,500	6,276	8,094	9,762	12,850	14,768	16,862	18,843
3.539	16+16 LB	1,500	6,284	8,092	9,764	12,850	14,777	16,865	18,865
3.539	16+16 RB	1,500	6,284	8,092	9,764	12,850	14,777	16,865	18,865
-	13+44 LB	1,500	6,276	8,088	9,768	12,845	14,785	16,853	18,701
-	13+44 RB	1,500	6,276	8,088	9,768	12,845	14,785	16,853	18,701
-	3.726	1,500	4,377	5,668	6,619	8,038	9,353	10,768	12,124
3.665	9+54 LB	1,500	6,272	8,108	9,792	12,860	14,789	16,854	18,738
3.665	9+54 RB	1,500	6,272	8,108	9,792	12,860	14,789	16,854	18,738
-	8+00 LB	1,500	2,629	3,612	4,397	5,405	6,318	7,134	8,278
-	6+00 LB	1,500	2,649	3,637	4,445	5,403	6,194	6,974	7,784
-	4+00 LB	1,500	2,669	3,652	4,410	5,439	6,194	6,993	7,824
-	2+00 LB	1,500	2,677	3,662	4,409	5,447	6,197	7,015	7,861
-	0+40	1,500	2,677	3,662	4,409	5,447	6,197	7,015	7,861

E.3.3.3 Peak Water Surface Elevations – FWOP Condition

Feasibility River Mile	Channel STA (feet)	Peak Water Surface Elevations (feet, NAVD88)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	891.58	900.16	901.92	903.08	905.38	906.88	908.13	909.37
2.623	64+14 LB	893.02	901.28	903.27	904.56	907.08	908.49	909.65	910.58
2.623	64+14 RB	893.02	901.12	903.14	904.41	906.87	908.56	909.64	910.53
2.651	62+70 LB	893.48	902.04	904.19	905.68	908.96	910.46	911.44	912.27
2.651	62+70 RB	893.48	901.30	903.27	904.79	908.60	909.99	911.04	911.95
-	58+28 LB	894.92	902.93	905.19	906.52	909.60	910.90	911.82	912.62
-	58+28 RB	894.92	903.05	905.19	906.59	909.52	910.77	911.68	912.47
2.813	54+18 LB	896.98	905.68	907.90	909.14	911.83	913.00	913.85	914.59
2.813	54+18 RB	896.98	905.48	907.81	908.85	911.68	912.94	913.81	914.56
-	50+96 LB	900.73	906.46	908.72	909.93	912.31	913.39	914.22	914.96
-	50+96 RB	900.73	906.65	908.78	909.96	912.28	913.34	914.16	914.91
-	45+86 LB	899.21	907.68	909.74	910.71	912.64	913.59	914.37	915.09
-	45+86 RB	899.21	907.67	909.64	910.59	912.57	913.53	914.31	915.03
3.04	42+30 LB	900.5	908.94	910.91	911.71	913.11	913.97	914.72	915.43
3.04	42+30 RB	900.5	909.05	911.07	911.90	913.37	914.25	915.05	915.78
3.07	40+76 LB	900.79	909.28	911.28	912.12	913.48	914.29	915.76	916.83
3.07	40+76 RB	900.79	909.35	911.38	912.20	913.61	914.45	915.98	917.07
-	34+88 LB	903.05	910.63	912.33	913.18	914.56	915.36	916.52	917.48
-	34+88 RB	903.05	910.48	912.13	913.13	914.46	915.24	916.38	917.33
3.298	28+82 LB	905.6	912.00	913.88	914.90	916.45	917.47	918.36	919.07
3.298	28+82 RB	905.6	912.00	913.88	914.85	916.39	917.33	918.28	918.97
3.345	26+38 LB	908.47	913.72	915.37	916.44	917.91	918.66	919.38	919.98
3.345	26+38 RB	908.47	913.22	914.78	915.77	916.97	917.71	918.40	919.12
3.481	19+20 LB	914.45	919.24	920.48	921.99	923.58	924.17	924.66	925.15
3.481	19+20 RB	914.45	919.25	920.48	922.03	923.39	924.07	924.66	925.12
3.539	16+16 LB	916.19	921.39	922.94	924.75	925.79	926.22	926.60	926.99
3.539	16+16 RB	916.19	921.54	923.14	924.52	925.96	926.42	926.92	927.56
-	13+44 LB	918.12	922.72	924.94	926.29	927.79	928.64	929.3	929.97
-	13+44 RB	918.12	922.83	924.13	925.76	927.71	928.64	929.53	930.42
-	3.726	925.87	931.15	933.29	934.81	935.62	936.02	936.43	936.82
3.665	9+54 LB	921.59	927.55	929.72	931.13	932.66	933.43	934.19	934.8
3.665	9+54 RB	921.59	926.52	928.94	930.82	932.46	933.14	933.96	934.68
-	8+00 LB	926.09	928.43	930.73	932.04	933.34	933.96	934.62	935.35
-	6+00 LB	926.66	928.79	930.94	931.96	933.24	933.89	934.56	935.31
-	4+00 LB	927.78	929.91	931.71	932.69	933.85	934.55	935.27	935.9
-	2+00 LB	929.47	931.06	932.82	933.85	935	935.73	936.37	936.89
-	0+40	930.8	932.31	933.99	935.18	936.26	937.02	937.57	938.01

E.3.3.4 Peak Flows – Post-35% Design

Feasibility River Mile	Channel STA (feet)	Peak Flows (cfs)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	1,500	6,818	9,184	10,925	14,273	16,499	18,677	20,376
2.623	64+14 LB	1,500	6,823	9,185	10,930	14,268	16,502	18,674	20,405
2.623	64+14 RB	1,500	6,823	9,185	10,930	14,268	16,502	18,674	20,405
2.651	62+70 LB	1,500	6,830	9,186	10,935	14,260	16,505	18,682	20,451
2.651	62+70 RB	1,500	6,830	9,186	10,935	14,260	16,505	18,682	20,451
-	58+28 LB	1,500	6,847	9,192	10,946	14,286	16,520	18,714	20,547
-	58+28 RB	1,500	6,847	9,192	10,946	14,286	16,520	18,714	20,547
2.813	54+18 LB	1,500	6,859	9,195	10,948	14,304	16,531	18,745	20,634
2.813	54+18 RB	1,500	6,859	9,195	10,948	14,304	16,531	18,745	20,634
-	50+96 LB	1,500	6,780	9,059	10,783	14,109	16,296	18,424	20,285
-	50+96 RB	1,500	6,780	9,059	10,783	14,109	16,296	18,424	20,285
-	45+86 LB	1,500	6,662	8,856	10,554	13,857	15,979	18,012	19,798
-	45+86 RB	1,500	6,662	8,856	10,554	13,857	15,979	18,012	19,798
3.04	42+30 LB	1,500	6,667	8,886	10,555	13,869	15,983	18,031	19,836
3.04	42+30 RB	1,500	6,667	8,886	10,555	13,869	15,983	18,031	19,836
3.07	40+76 LB	1,500	6,670	8,898	10,552	13,874	15,985	18,040	19,840
3.07	40+76 RB	1,500	6,670	8,898	10,552	13,874	15,985	18,040	19,840
-	34+88 LB	1,500	6,173	8,191	9,734	12,833	14,765	16,605	18,232
-	34+88 RB	1,500	6,173	8,191	9,734	12,833	14,765	16,605	18,232
3.298	28+82 LB	1,500	6,165	8,153	9,709	12,835	14,774	16,579	18,194
3.298	28+82 RB	1,500	6,165	8,153	9,709	12,835	14,774	16,579	18,194
3.345	26+38 LB	1,500	6,165	8,145	9,699	12,836	14,781	16,566	18,322
3.345	26+38 RB	1,500	6,165	8,145	9,699	12,836	14,781	16,566	18,322
3.481	19+20 LB	1,500	6,156	8,147	9,705	12,870	14,784	16,575	18,377
3.481	19+20 RB	1,500	6,156	8,147	9,705	12,870	14,784	16,575	18,377
3.539	16+16 LB	1,500	6,150	8,148	9,709	12,883	14,779	16,579	18,358
3.539	16+16 RB	1,500	6,150	8,148	9,709	12,883	14,779	16,579	18,358
-	13+44 LB	1,500	6,150	8,151	9,711	12,885	14,772	16,595	18,392
-	13+44 RB	1,500	6,150	8,151	9,711	12,885	14,772	16,595	18,392
-	3.726	1,500	4,240	5,542	6,646	8,086	9,369	10,875	12,649
3.665	9+54 LB	1,500	6,159	8,177	9,770	12,881	14,760	16,605	18,486
3.665	9+54 RB	1,500	6,159	8,177	9,770	12,881	14,760	16,605	18,486
-	8+00 LB	1,500	2,630	3,621	4,318	4,743	5,319	5,889	6,269
-	6+00 LB	1,500	2,646	3,635	4,410	5,330	6,141	6,915	7,774
-	4+00 LB	1,500	2,648	3,648	4,414	5,365	6,157	6,957	7,792
-	2+00 LB	1,500	2,662	3,672	4,515	5,385	6,198	7,047	7,814
-	0+40	1,500	2,662	3,672	4,515	5,385	6,198	7,047	7,814

E.3.3.5 Peak Water Surface Elevations – Post-35% Design

Feasibility River Mile	Channel STA (feet)	Peak Water Surface Elevations (feet, NAVD88)							
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
2.594	65+66	891.64	900.63	903.10	904.70	907.08	908.40	909.58	910.39
2.623	64+14 LB	893.37	901.94	904.12	905.57	907.88	909.20	910.38	911.09
2.623	64+14 RB	893.37	902.42	904.76	906.35	908.37	909.71	911.15	912.29
2.651	62+70 LB	893.78	903.07	905.54	907.17	908.74	910.13	911.68	913.27
2.651	62+70 RB	893.78	902.82	905.18	906.77	908.22	909.55	911.05	912.53
-	58+28 LB	894.86	903.25	905.71	907.26	909.24	910.73	911.95	913.32
-	58+28 RB	894.86	903.41	905.84	907.28	909.95	910.75	911.83	913.46
2.813	54+18 LB	896.92	905.82	908.37	910.05	912.27	913.74	914.94	916.03
2.813	54+18 RB	896.92	905.64	908.21	909.94	912.14	913.41	914.48	915.45
-	50+96 LB	901.86	906.49	909.11	910.76	913.01	914.57	915.94	917.20
-	50+96 RB	901.86	906.66	909.14	910.70	912.89	914.42	915.77	917.03
-	45+86 LB	899.05	907.29	909.55	911.18	913.37	914.80	916.07	917.26
-	45+86 RB	899.05	907.11	909.82	911.08	913.06	914.44	915.68	916.90
3.04	42+30 LB	900.45	908.58	910.91	912.51	914.70	916.11	917.37	918.46
3.04	42+30 RB	900.45	908.69	911.08	912.68	914.92	916.38	917.69	918.84
3.07	40+76 LB	900.67	908.83	911.27	912.85	915.07	916.89	918.43	919.46
3.07	40+76 RB	900.67	908.95	911.40	912.97	915.22	917.02	918.58	919.67
-	34+88 LB	903.03	910.10	912.17	913.56	915.66	917.28	918.81	919.80
-	34+88 RB	903.03	909.91	911.98	913.32	915.29	916.89	918.41	919.32
3.298	28+82 LB	905.62	911.66	913.83	915.16	917.52	919.04	920.39	921.35
3.298	28+82 RB	905.62	911.59	913.71	915.08	917.41	918.97	920.38	921.38
3.345	26+38 LB	908.58	913.60	915.33	916.69	918.59	919.79	921.02	922.06
3.345	26+38 RB	908.58	913.07	914.64	915.88	917.81	919.04	920.28	921.23
3.481	19+20 LB	914.52	919.60	920.98	921.73	921.91	922.38	923.93	924.90
3.481	19+20 RB	914.52	919.58	920.95	921.79	923.00	923.55	924.09	925.03
3.539	16+16 LB	916.21	921.82	923.37	924.46	926.14	927.00	927.81	928.42
3.539	16+16 RB	916.21	921.85	923.43	924.56	926.36	927.19	928.12	928.86
-	13+44 LB	918.25	923.46	925.14	926.25	927.58	928.18	930.46	931.72
-	13+44 RB	918.25	923.66	925.19	926.37	927.98	928.68	930.96	932.21
-	3.726	925.91	931.80	933.49	934.86	935.86	936.25	936.86	937.54
3.665	9+54 LB	921.6	927.65	929.95	932.10	933.89	934.59	935.66	936.56
3.665	9+54 RB	921.6	927.06	929.25	931.94	934.65	934.77	935.29	936.22
-	8+00 LB	926.12	928.38	930.58	932.73	934.47	935.16	936.16	937.02
-	6+00 LB	926.59	928.61	930.74	932.83	934.59	935.31	936.31	937.17
-	4+00 LB	927.76	929.73	931.54	933.15	934.83	935.53	936.49	937.33
-	2+00 LB	929.15	930.93	932.46	933.78	935.23	935.87	936.75	937.55
-	0+40	923.41	932.19	933.75	935.13	936.52	937.26	938.12	938.99

APPENDIX E.4 – RIPRAP SIZING INPUT DATA

Design Criteria

Upstream Reference Cross-Section

Initial Computed Riprap Sizes

Cross Section Riprap Design

E.4 RIPRAP SIZING INPUT DATA

E.4.1 Design Criteria

Riprap sizing was computed in the HEC-RAS riprap sizing tool which uses design requirements outlined in EM 1110-2-1601. Riprap size is a function of channel/overbank geometry, stone unit weight, velocities, and safety factors.

E.4.2 Upstream Reference Cross-Section

Upstream reference cross-section data was pulled from the Turkey Creek channel upstream (south) of Shawnee Mission Parkway, outside of the project area.

Table E.1. Upstream Reference Cross Section Input Data

Parameter	Value
Average Channel Velocity	9.7 ft/s
Hydraulic Depth	10.3 ft
Channel Width	80 ft

E.4.3 Initial Computed Riprap Sizes

Initial riprap sizes were developed with the HEC-RAS tool for various sections along the left and right banks. Riprap sizes were consolidated to just three separate diameters to simplify the design as shown in Section 4.5.2 of the main report. Table E.2 and Table E.3 provide the initial computed riprap sizes for the left and right bank, respectively

Table E.2. Computed Left Bank Maximum Riprap Sizes

Channel STA	Velocity (ft/s)	D100 (in) (Gradation)
0+00-9+00 (Tribs LB & RB)	12	21 (EM 1601-5)
9+00-14+00	15	33 (EM 1601-9)
15+25-19+00	10	15 (EM 1601-3)
21+00-25+00	9	15 (EM 1601-3)
33+00-36+00	9	15 (EM 1601-3)
63+00-End	15	24 (EM 1601-6)

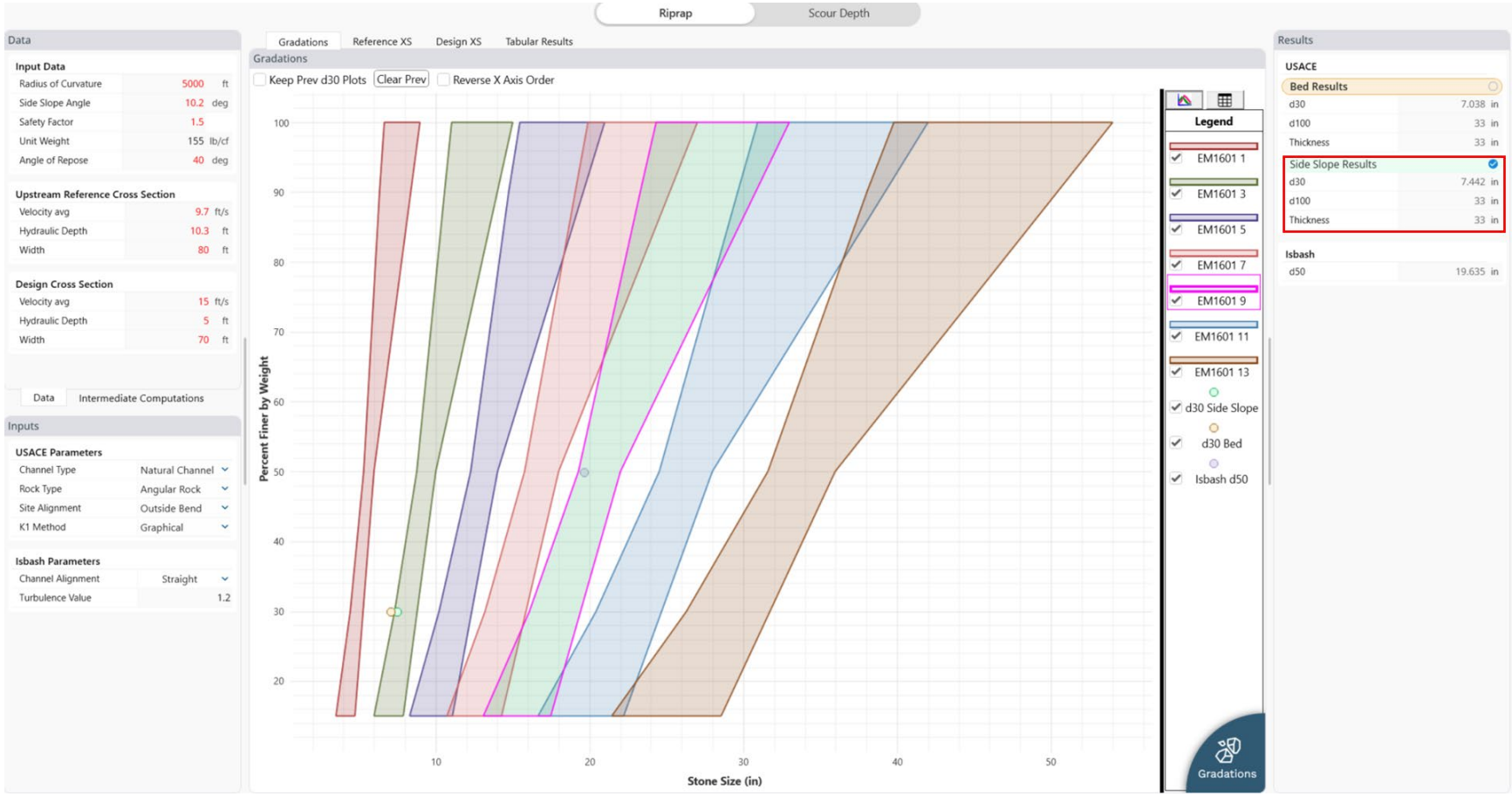
Table E.3. Computed Right Bank Maximum Riprap Sizes

Channel STA	Velocity (ft/s)	D100 (in) (Gradation)
15+25-19+00	10	15 (EM 1601-3)
19+00-26+00	9	15 (EM 1601-3)
30+00-33+00	9	30 (EM 1601-8)
33+00-36+00	12	21 (EM 1601-5)
36+00-42+00	9	21 (EM 1601-5)
42+00-48+00	12	21 (EM 1601-5)
53+00-57+00	12	21 (EM 1601-5)
57+00-60+00	15	33 EM (1601-9)
60+00-64+00	12	21 (EM 1601-5)
64+00-End	10	42 (EM 1601-11)

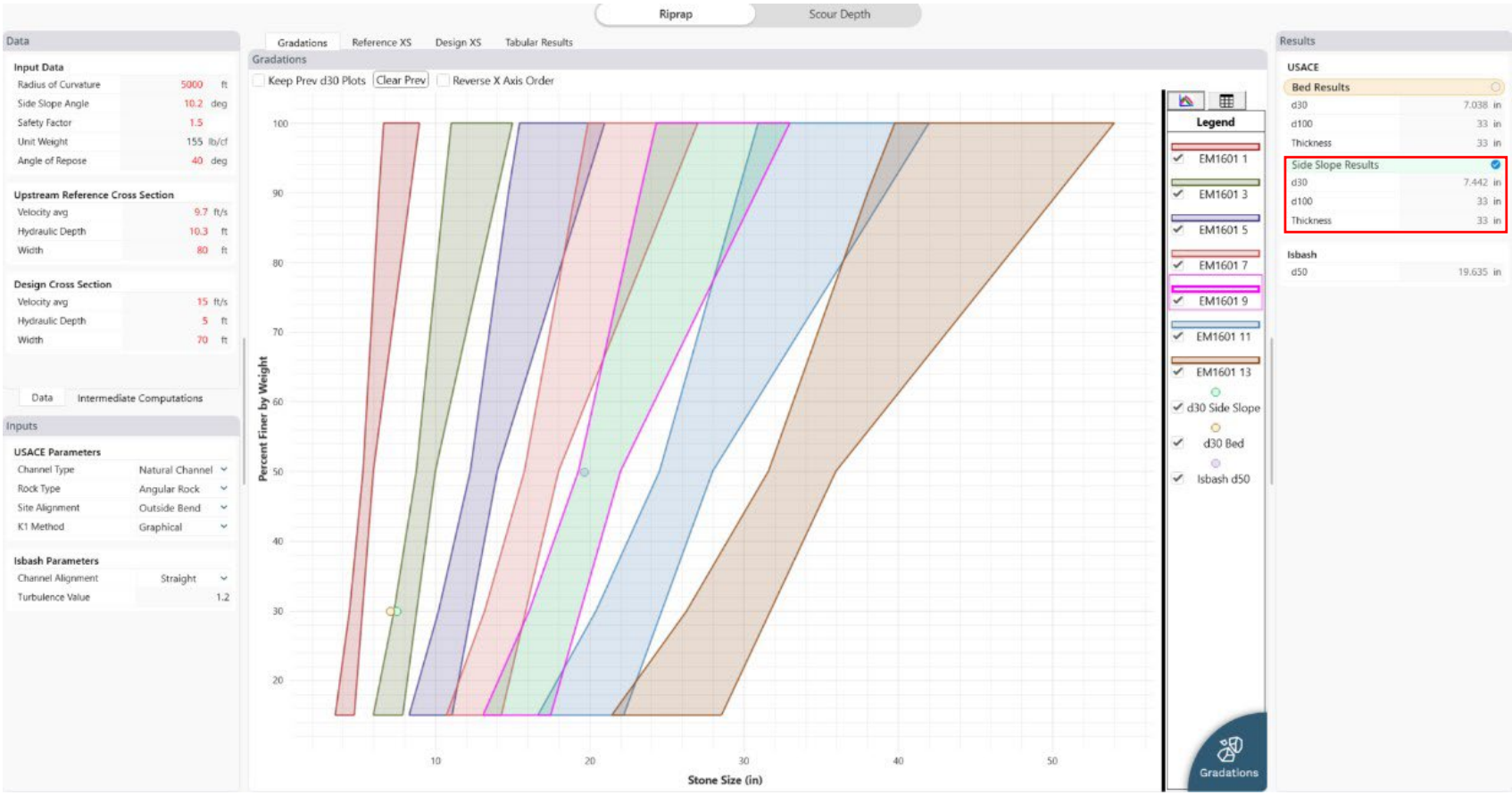
E.4.4 Cross Section Riprap Design

The following sections provide the riprap design information and calculations performed for each cross section. These figures detail the input parameters on the left side pane, and the selected gradation in the center that intersects with the plotted riprap sizing methods. On the right pane details that selected riprap gradations maximum diameter (D_{100}) value in inches. Each figure represents a riprap sizing for a given reach based on unique channel characteristics and velocities.

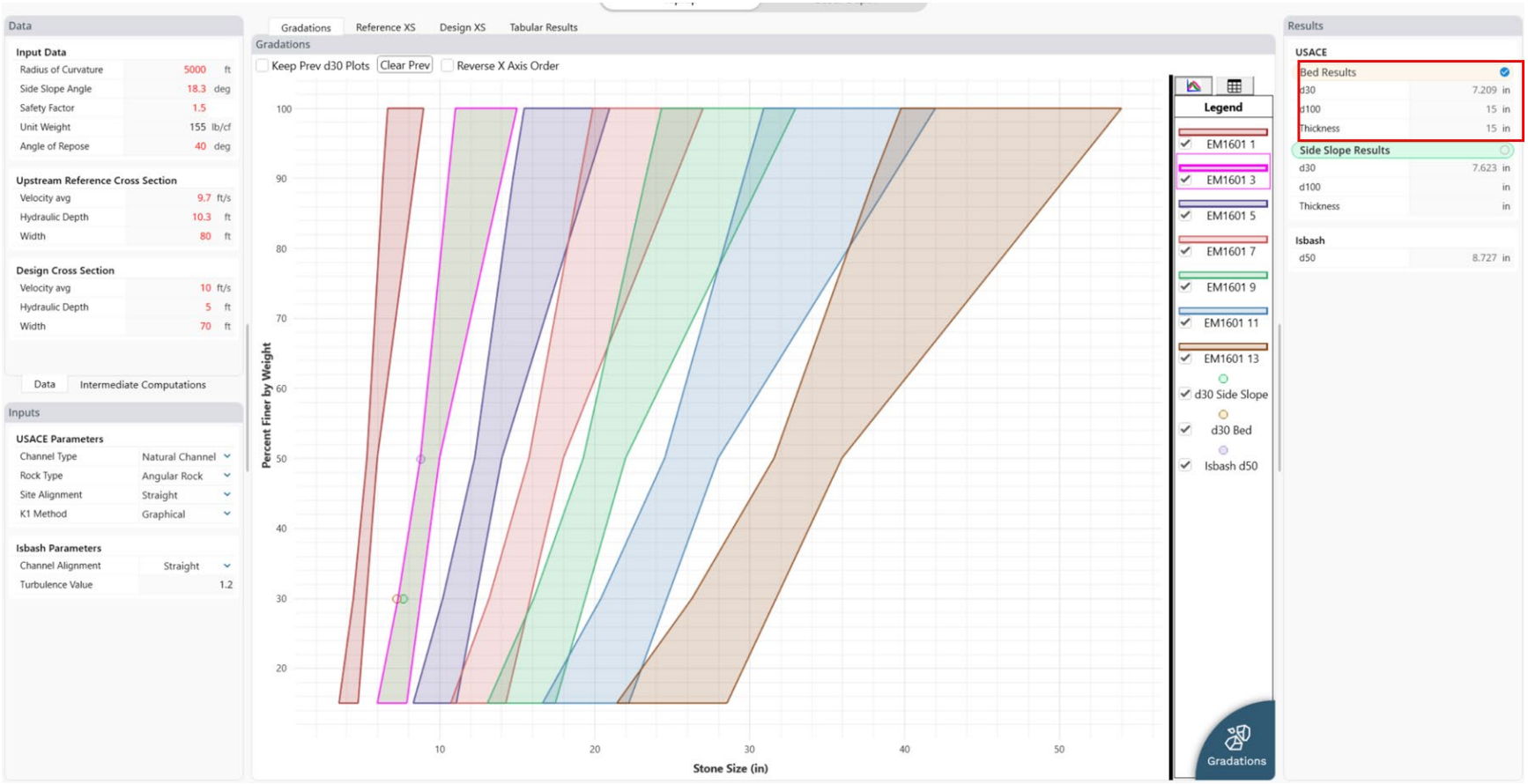
E.4.4.1 Trib (Left and Right Bank) (Channel STA 0+00-9+00)



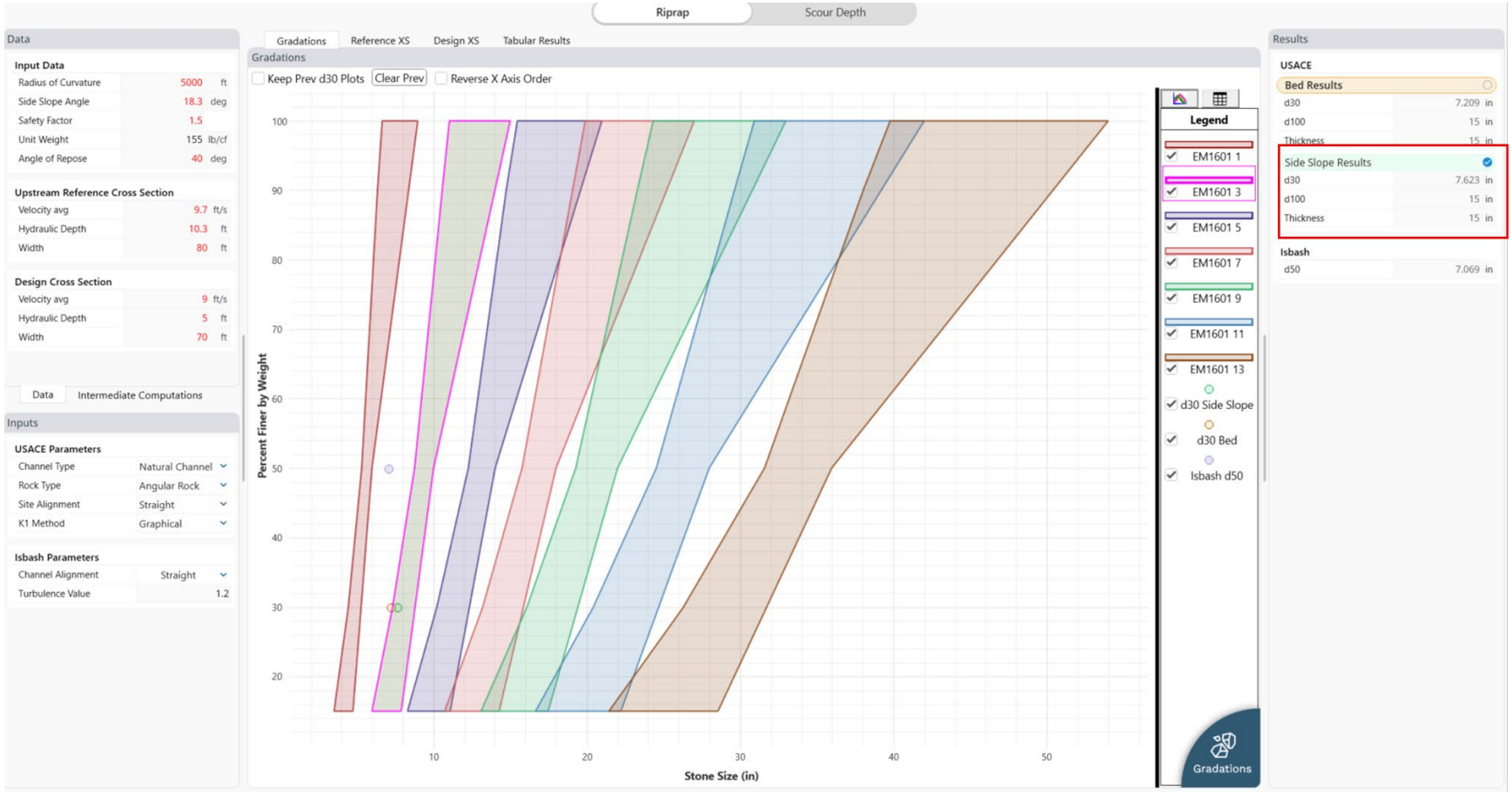
E.4.4.2 Left Bank (Channel STA 9+00-14+00)



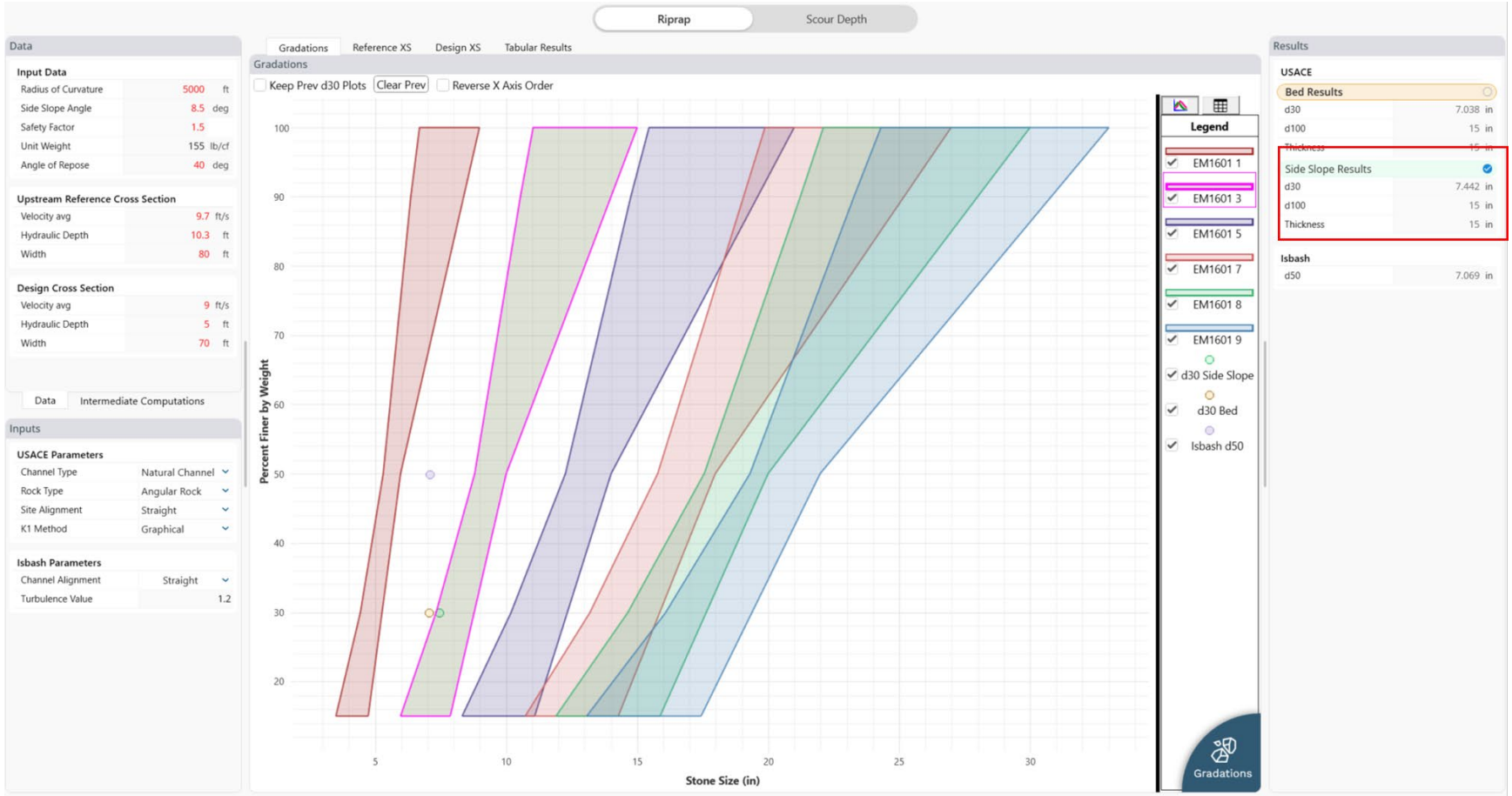
E.4.4.3 Left Bank (Channel STA 15+25-19+00)



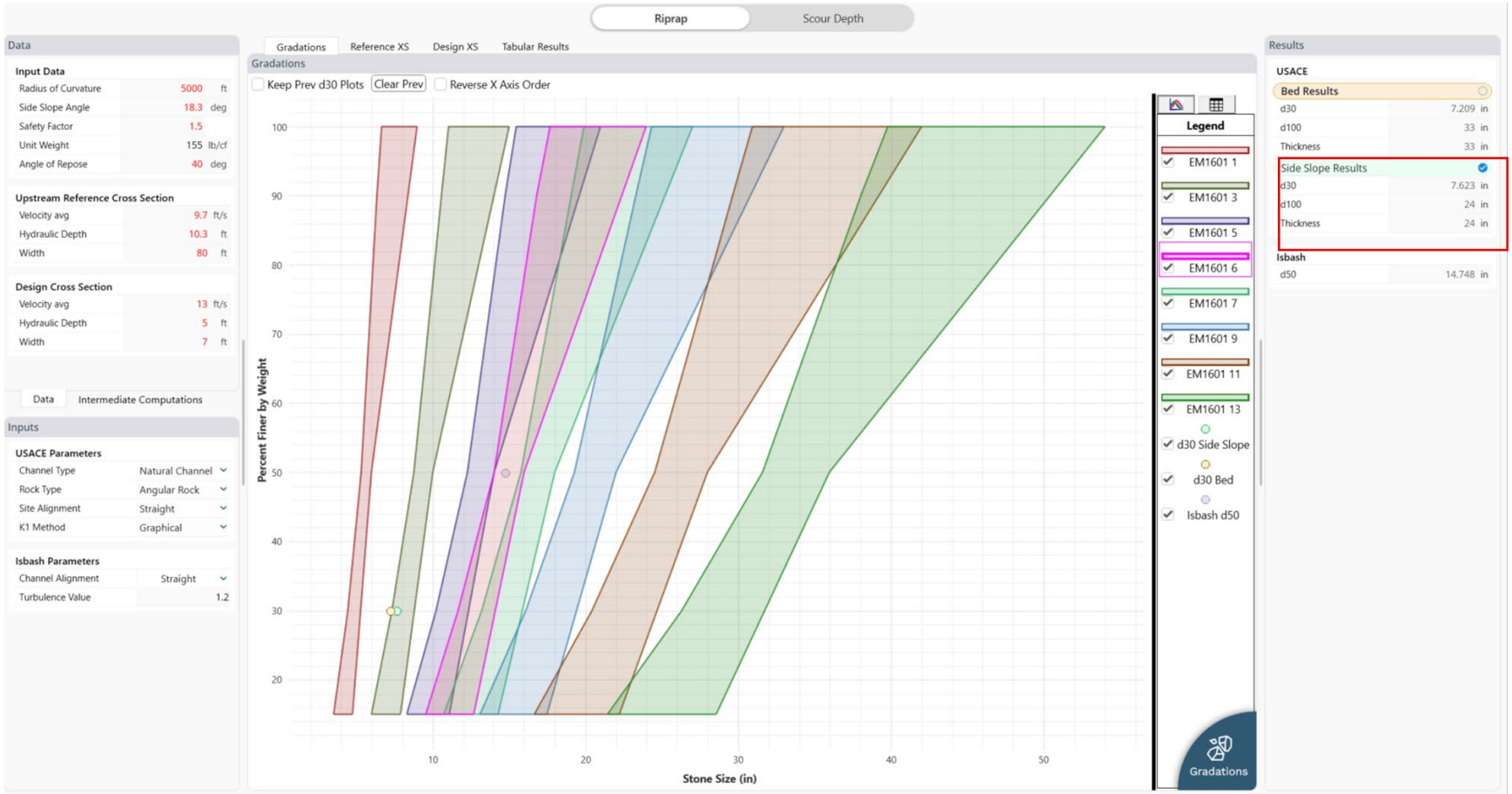
E.4.4.4 Left Bank (Channel STA 21+00-25+00)



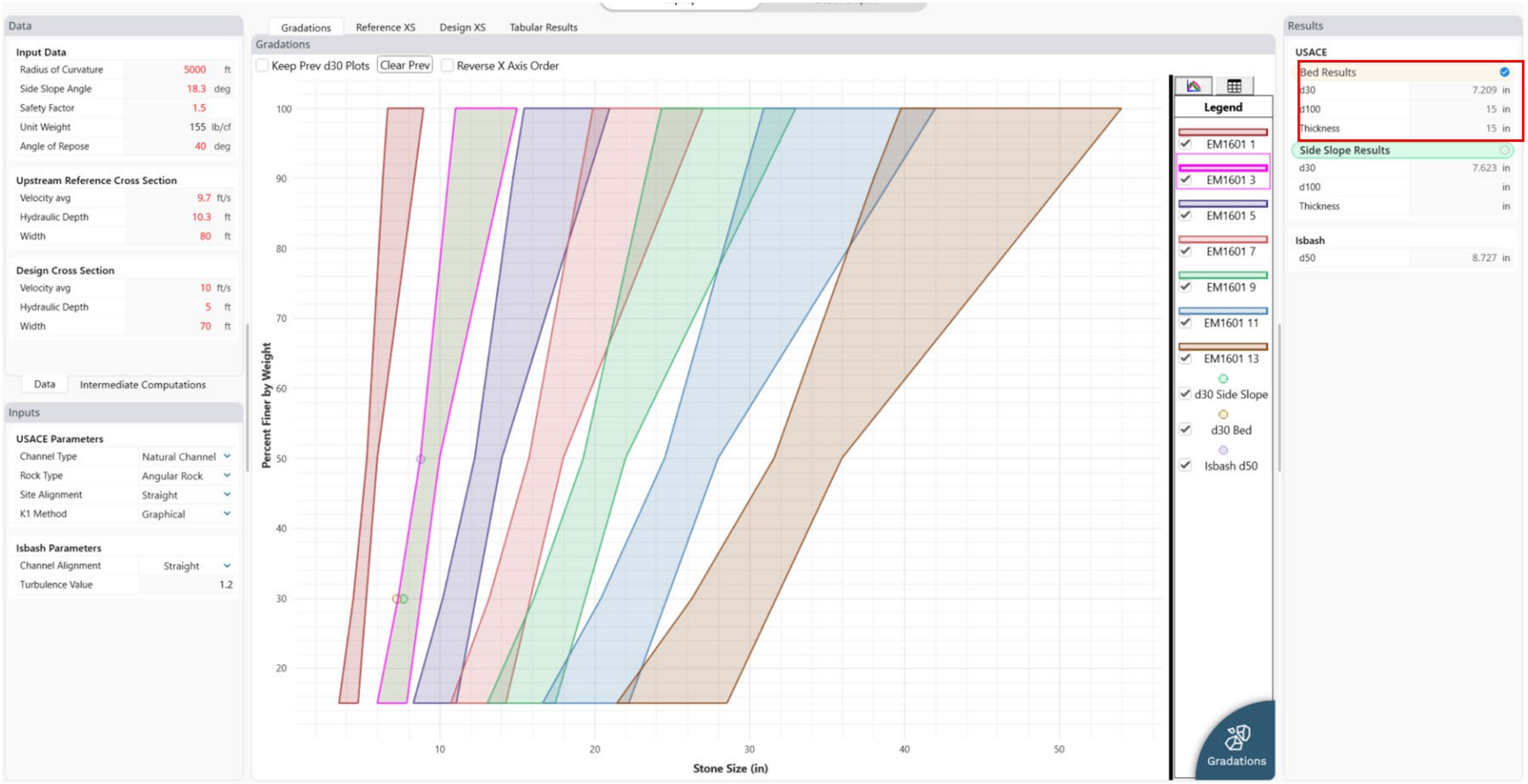
E.4.4.5 Left Bank (Channel STA 33+00-36+00)



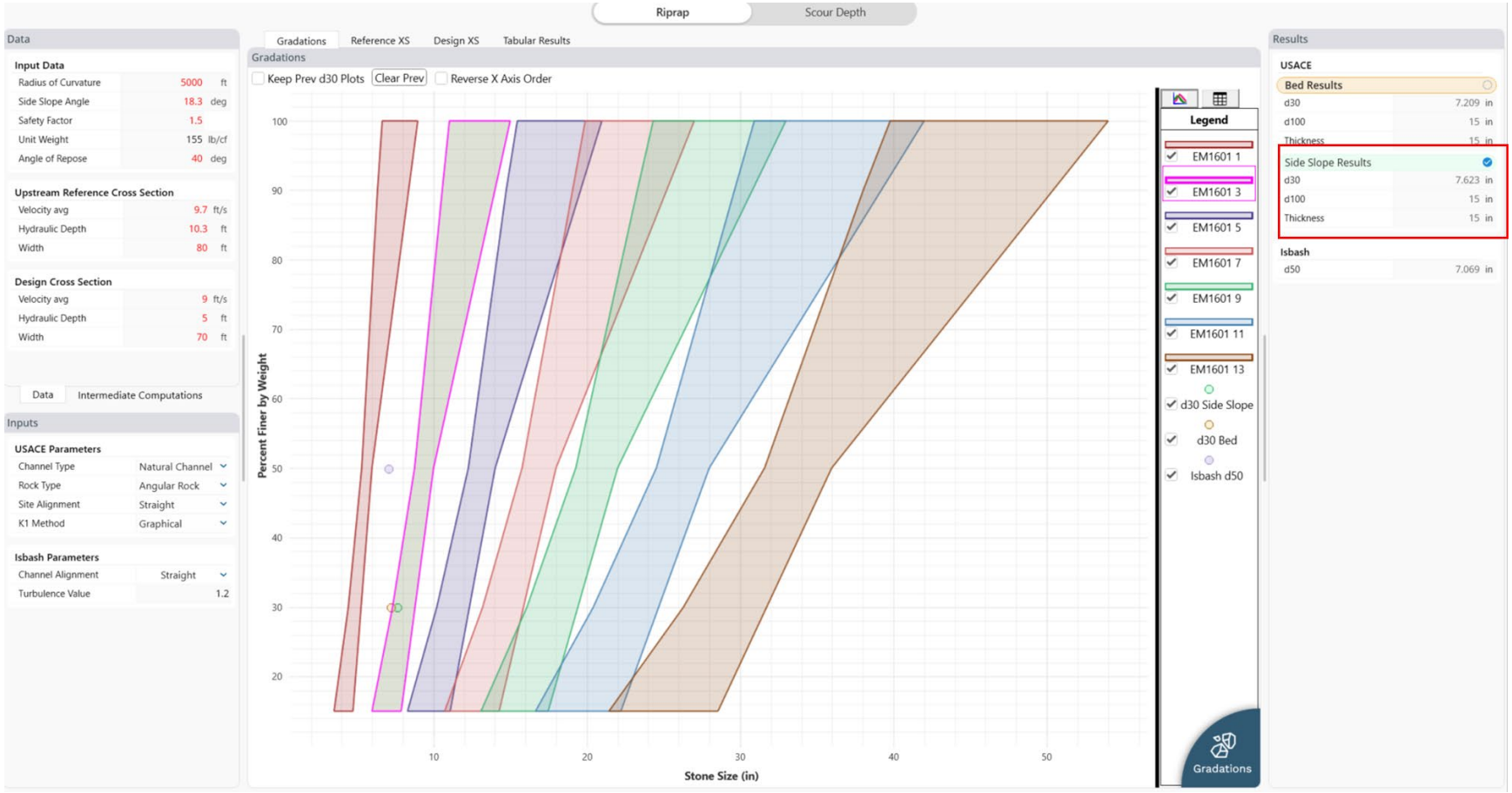
E.4.4.6 Left Bank (Channel STA 63+00-End)



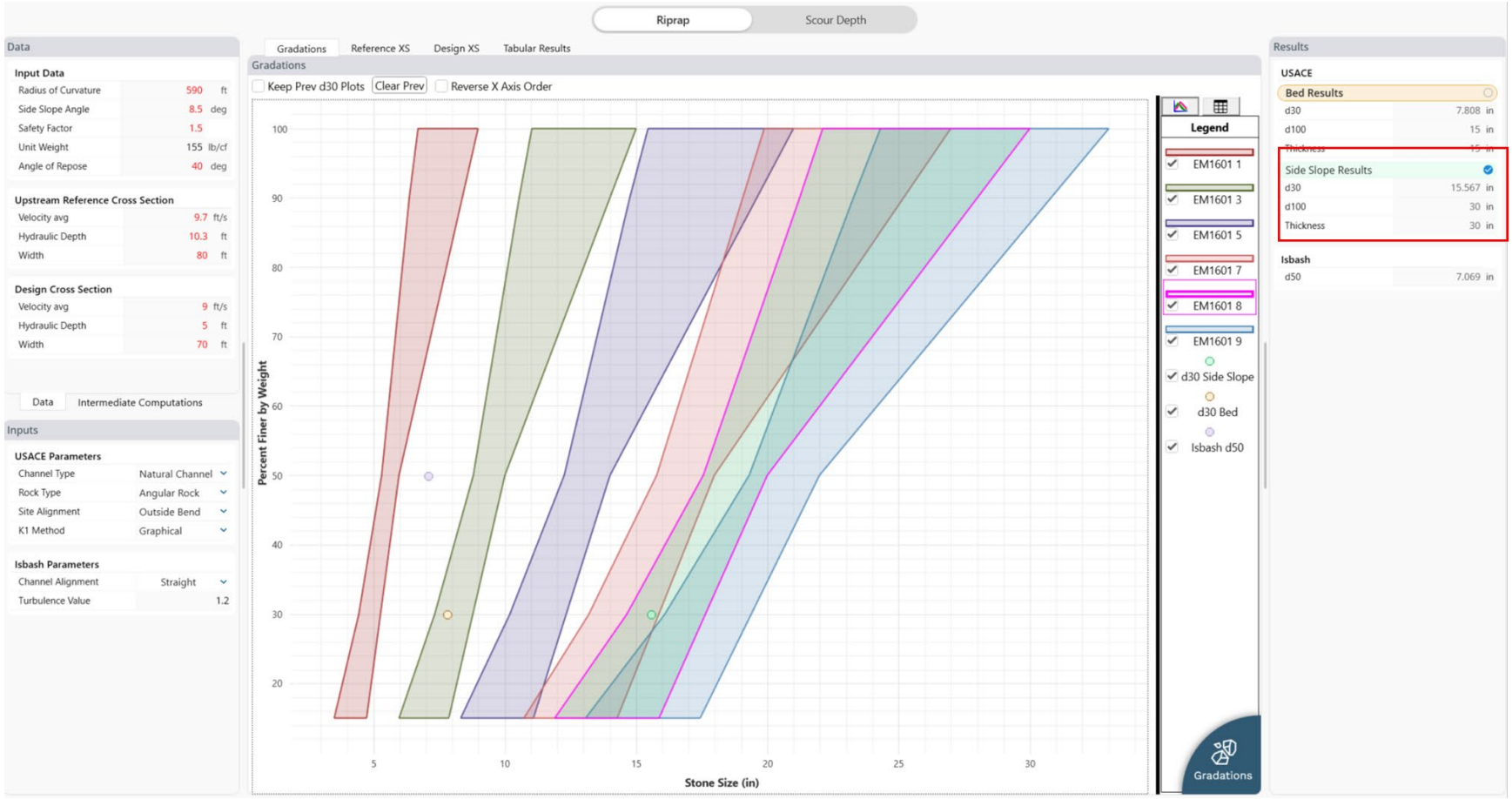
E.4.4.7 Right Bank (Channel STA 15+25-19+00)



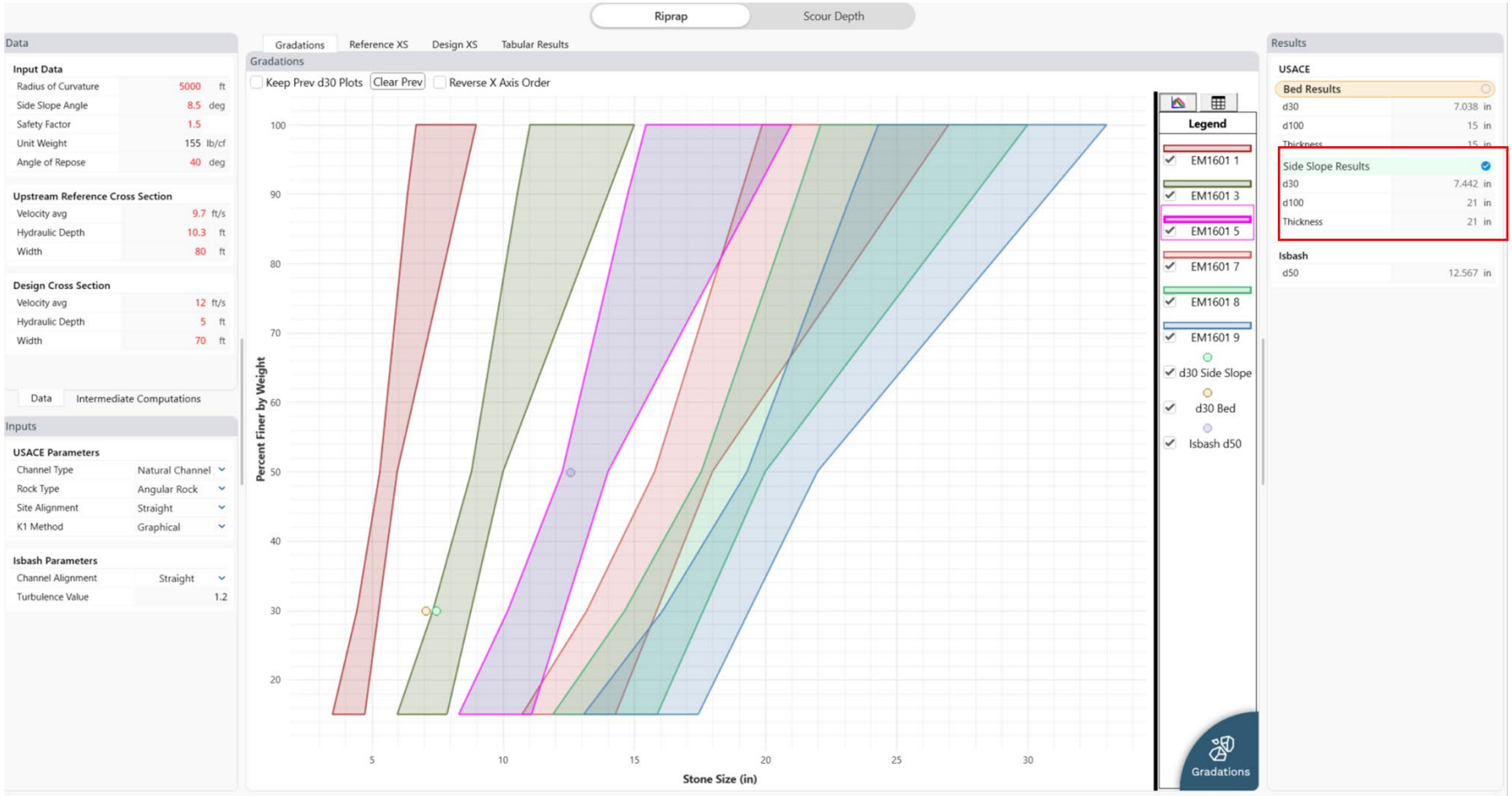
E.4.4.8 Right Bank (Channel STA 19+00-26+00)



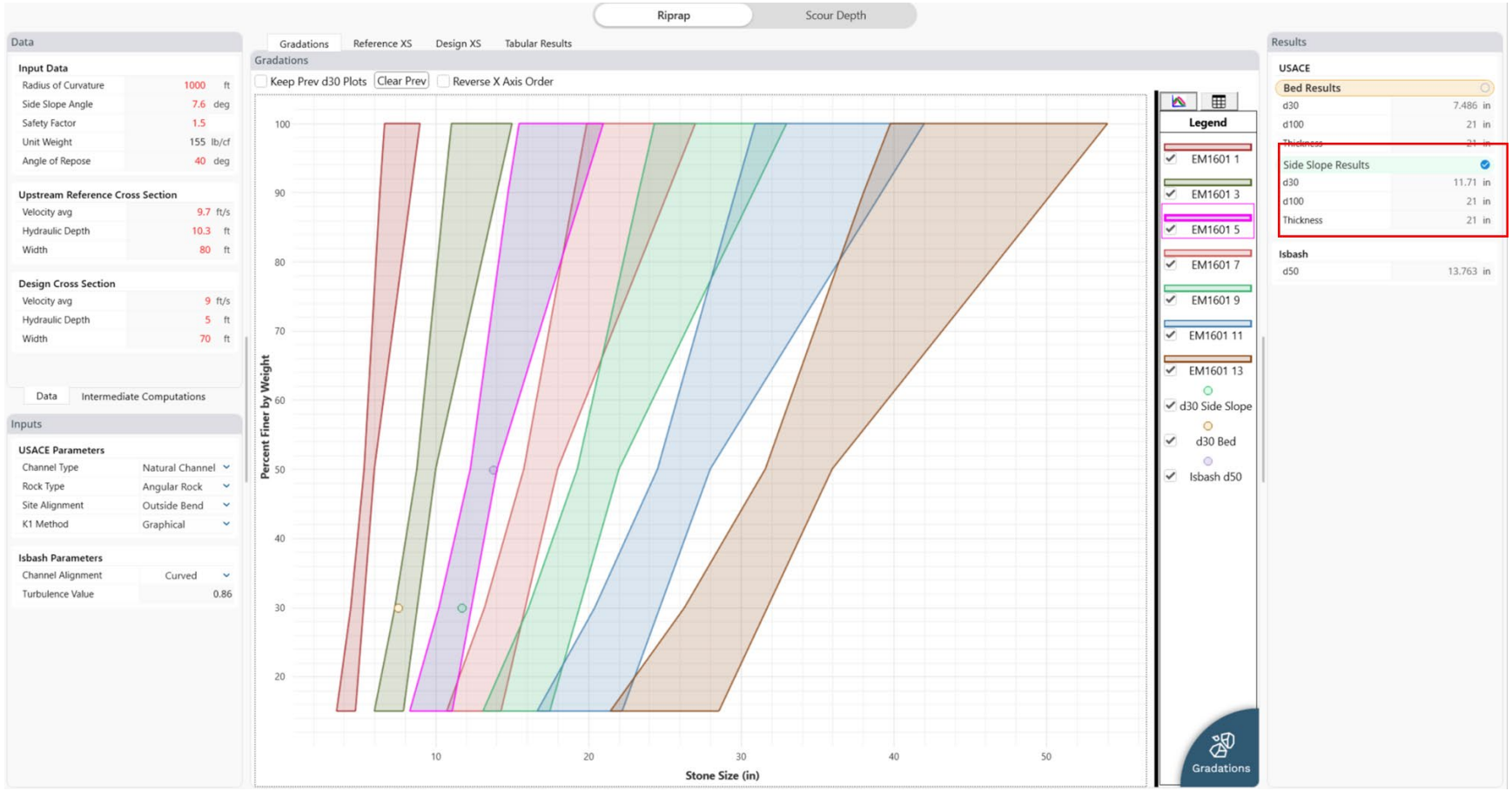
E.4.4.9 Right Bank (Channel STA 30+00-33+00)



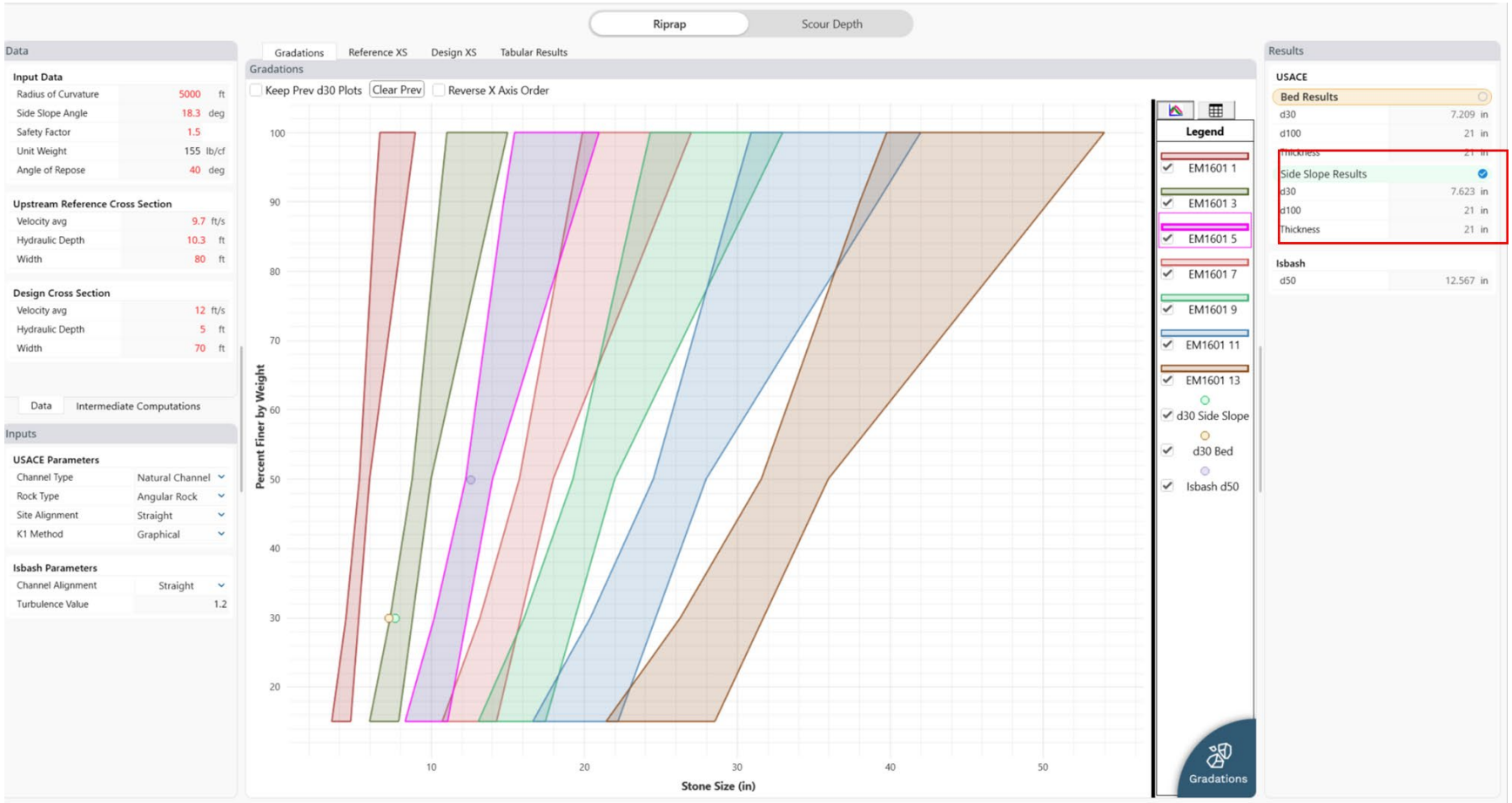
E.4.4.10 Right Bank (Channel STA 33+00-36+00)



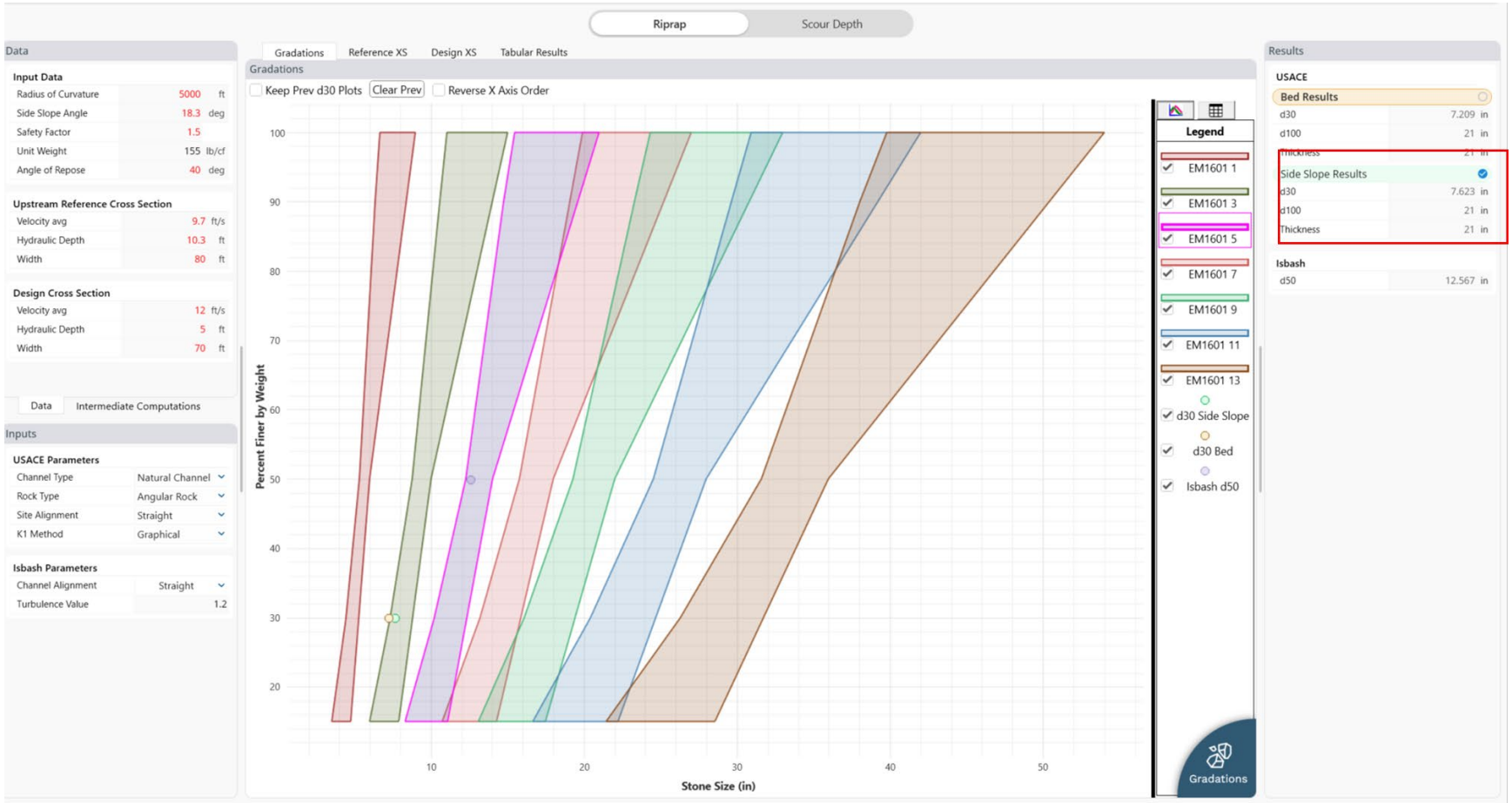
E.4.4.11 Right Bank (Channel STA 36+00-42+00)



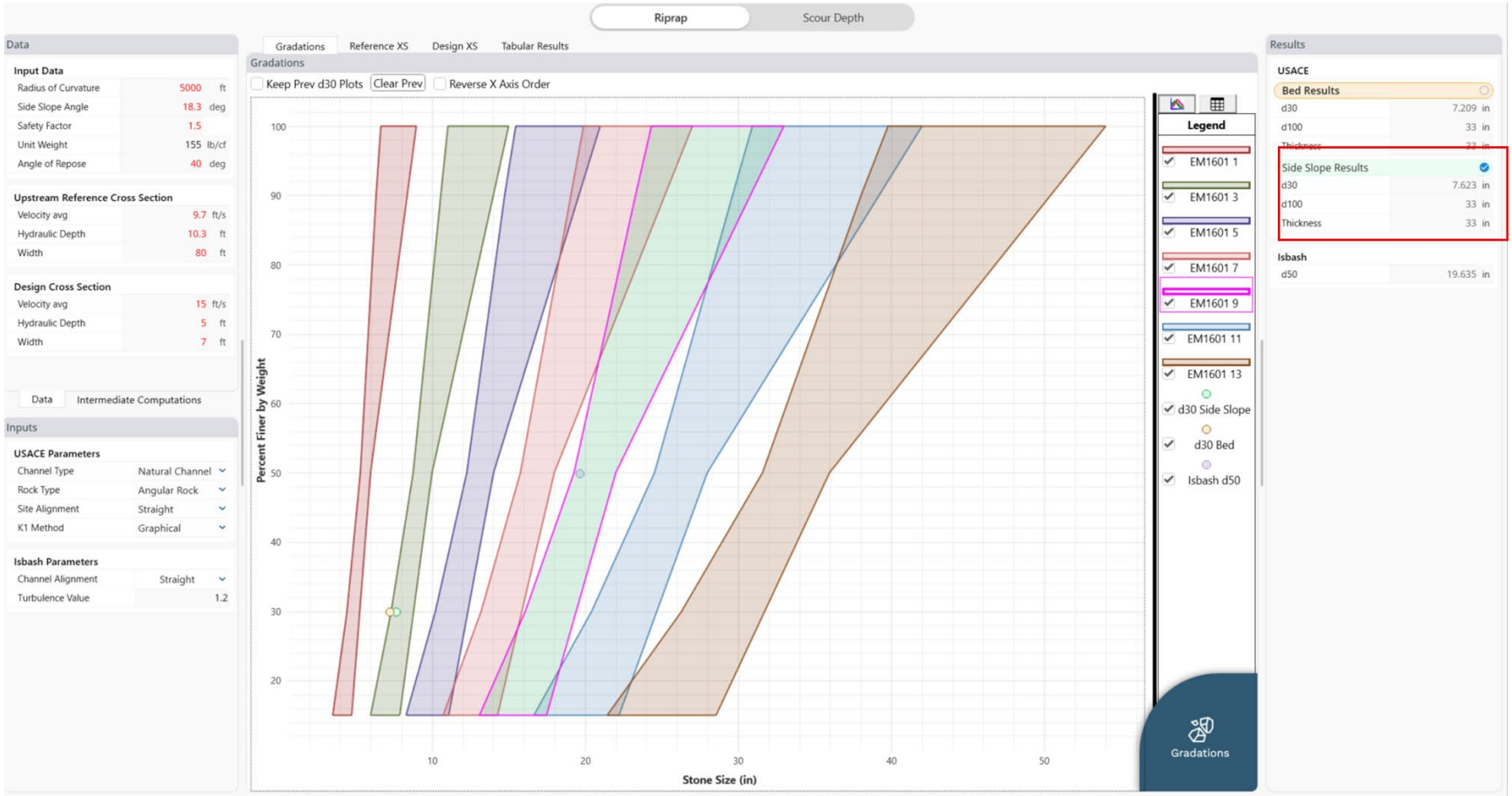
E.4.4.12 Right Bank (Channel STA 42+00-48+00)



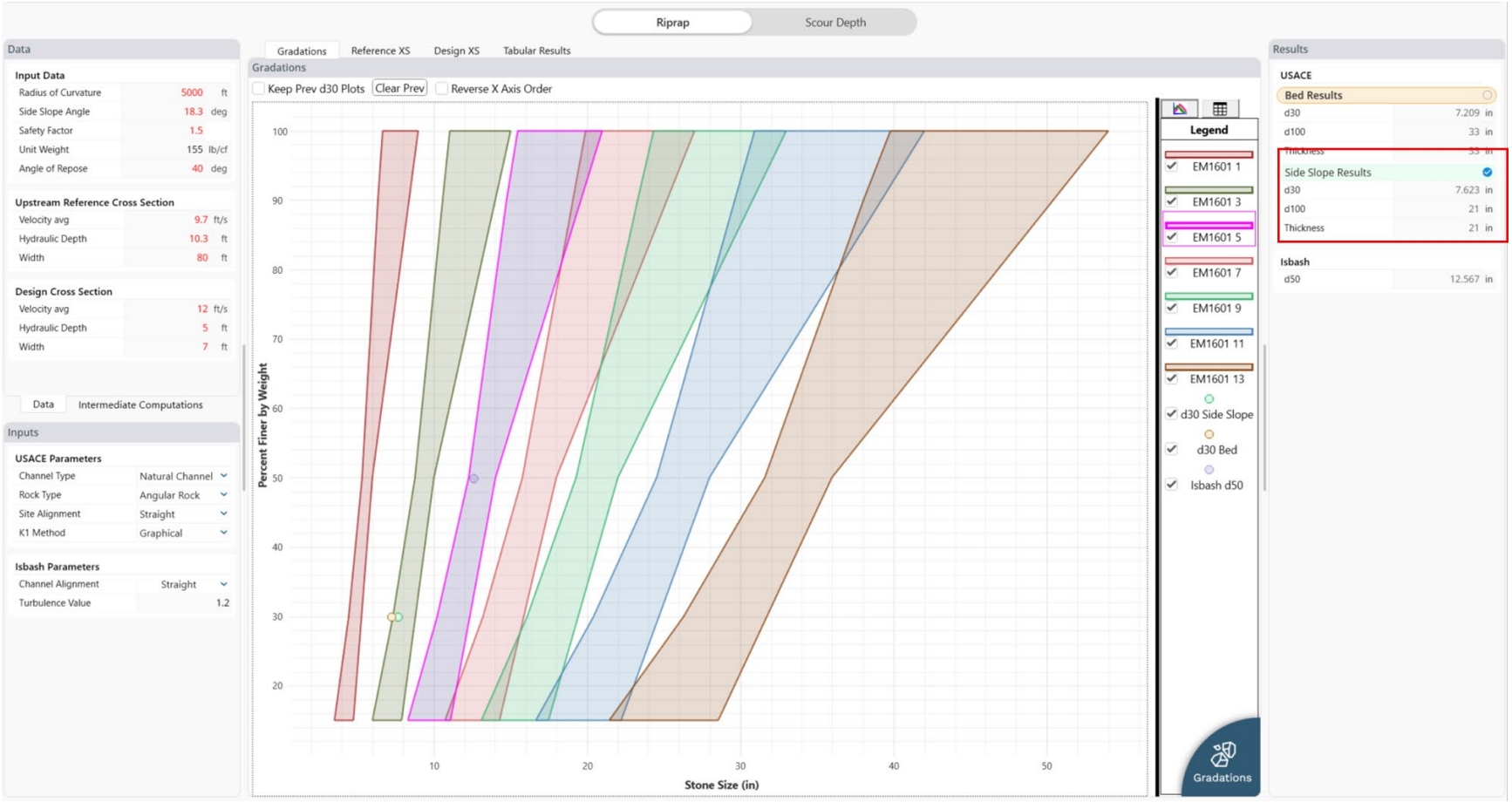
E.4.4.13 Right Bank (Channel STA 53+00-57+00)



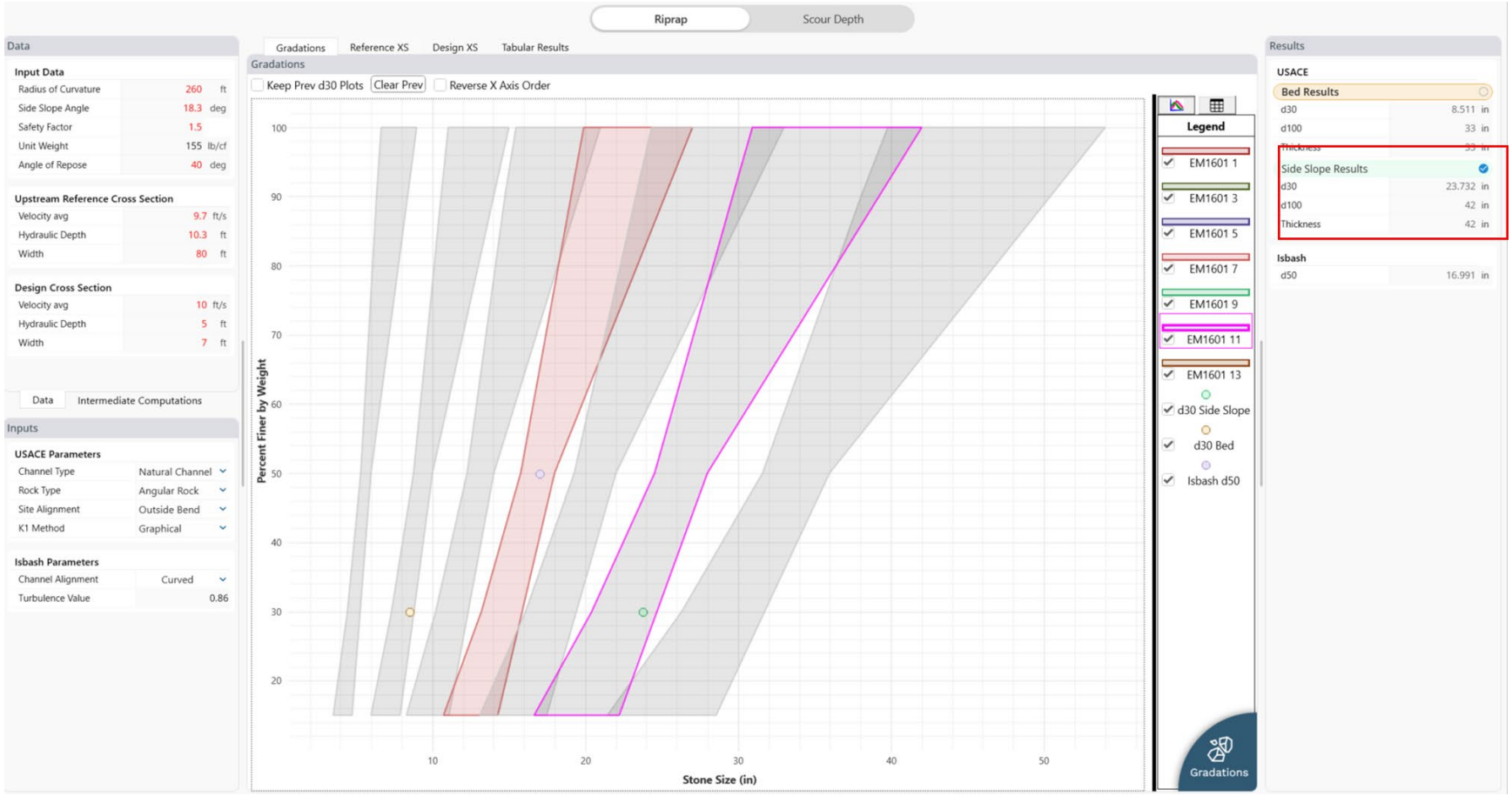
E.4.4.14 Right Bank (Channel STA 57+00-60+00)



E.4.4.15 Right Bank (Channel STA 60+00-64+00)



E.4.4.16 Right Bank (Channel STA 63+00-End)



APPENDIX E.5 – BRIDGE SCOUR H&H INPUT DATA

Introduction

Shawnee Mission Parkway Pedestrian Bridge

Merriam Market Place Pedestrian Bridge

E.5 BRIDGE SCOUR H&H INPUT DATA

E.5.1 Introduction

H&H data provided to the structural PDT member to perform a bridge scour analysis in accordance with Hydrologic Engineering Circular No. 18 – Evaluating Scour at Bridges from the Federal Highway Administration. Data was pulled to analyze the two pedestrian bridges in the project extents just downstream of Shawnee Mission Parkway and at Merriam Marketplace. Scour analyses for Johnson Drive and Merriam Drive bridges was not performed in-house as it was part of the scope for HNTB who are contracted to do the bridge designs.

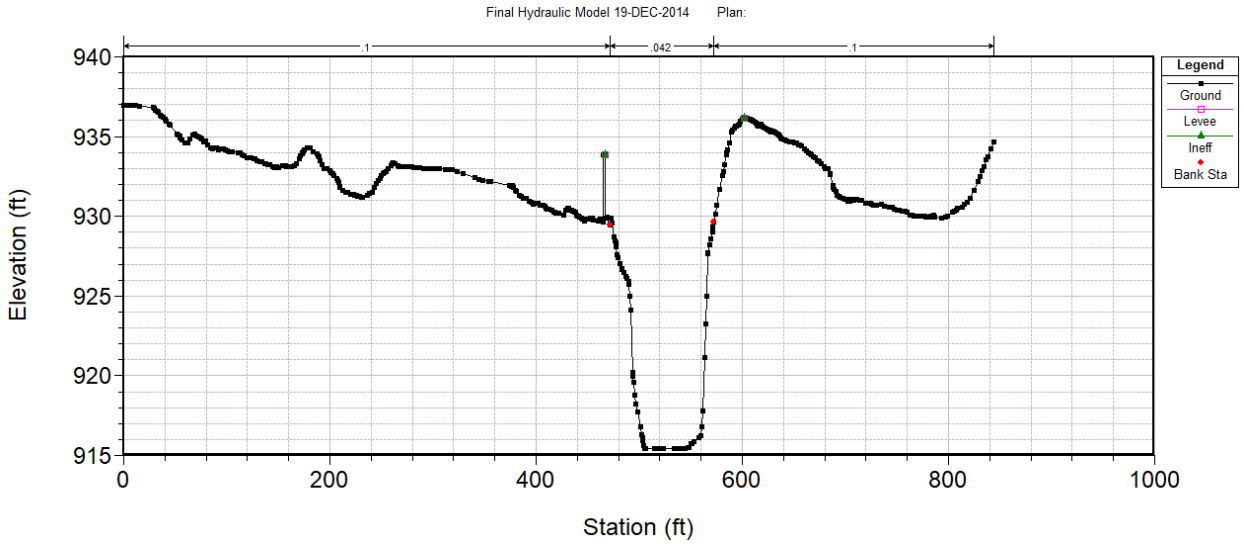
Detailed hydraulic output tables were pulled from the Stantec 2D both pedestrian bridges. All results are based on the 1% AEP design event. Each bridge required 4 cross section outputs including:

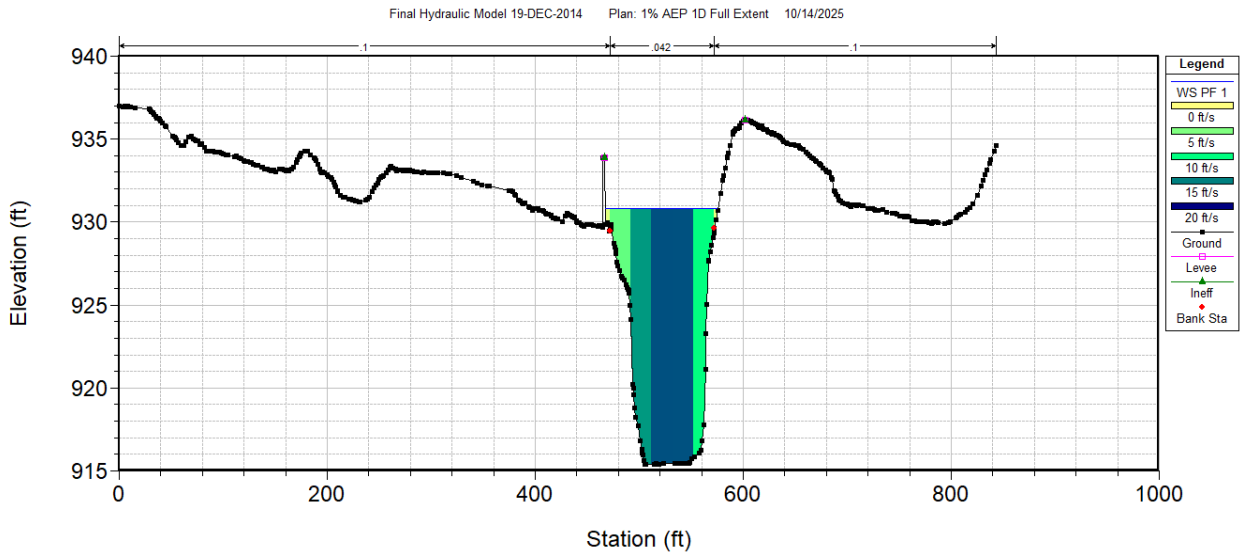
- Upstream approach cross-section
- Upstream bridge face
- Downstream bridge face
- Downstream exit cross-section

E.5.2 Shawnee Mission Parkway Pedestrian Bridge

Bridge Opening Width (ft)		102
Floodway Width (ft)		206
Bottom Deck Elevation (ft)	**Max**	928.58
	Min	926.20

E.5.2.1 Upstream Approach Cross-Section





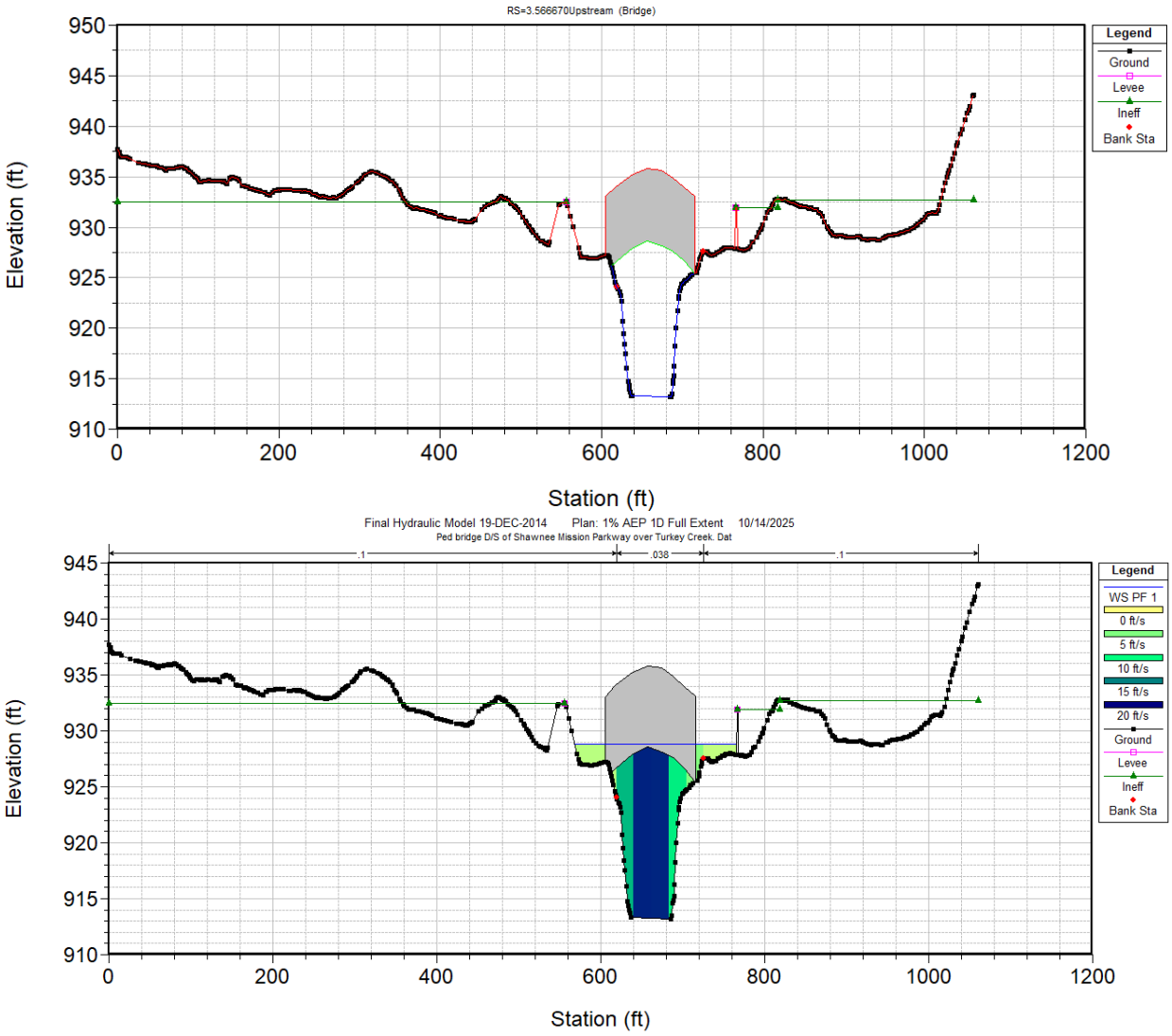
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 3.607 Profile: PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	466.88	471.93	6	4.84	5.51	0.03	1.05	1.17	0.41	0.48
2	Chan	471.93	492.02	450	72.99	21.48	2.71	3.63	6.17	1.57	9.67
3	Chan	492.02	512.11	3837	274.07	23.58	23.11	13.64	14	5.36	75.08
4	Chan	512.11	532.21	5220	309.22	20.09	31.44	15.39	16.88	7.1	119.86
5	Chan	532.21	552.3	5178	307.84	20.11	31.19	15.32	16.82	7.06	118.8
6	Chan	552.3	572.39	1908	191.19	27.33	11.49	9.52	9.98	3.23	32.21
7	ROB	572.39	601.51	2	2.14	3.61	0.01	0.63	0.9	0.27	0.25

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 3.607 Profile: PF 1

E.G. Elev (ft)	934.02	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	3.2	**Wt. n-Val.**	0.1	0.042	0.1
W.S. Elev (ft)	930.82	**Reach Len. (ft)**	159.5	195.5	214.4
Crit W.S. (ft)		**Flow Area (sq ft)**	4.84	1155.3	2.14
E.G. Slope (ft/ft)	0.00739	**Area (sq ft)**	4.84	1155.3	2.14
Q Total (cfs)	16600	**Flow (cfs)**	5.67	16592.4	1.93
Top Width (ft)	108.48	**Top Width (ft)**	4.62	100.46	3.41
Vel Total (ft/s)	14.28	**Avg. Vel. (ft/s)**	1.17	14.36	0.9
Max Chl Dpth (ft)	15.41	**Hydr. Depth (ft)**	1.05	11.5	0.63
Conv. Total (cfs)	193095.5	**Conv. (cfs)**	66	193007.1	22.5
Length Wtd. (ft)		**Wetted Per. (ft)**	5.51	112.59	3.61
Min Ch El (ft)	915.41	**Shear (lb/sq ft)**	0.41	4.73	0.27
Alpha	1.01	**Stream Power (lb/ft s)**	0.48	68	0.25
Frctn Loss (ft)		**Cum Volume (acre-ft)**	87.48	279.29	13.39
C & E Loss (ft)		**Cum SA (acres)**	15.6	17.09	3.01

E.5.2.2 Upstream Bridge Face



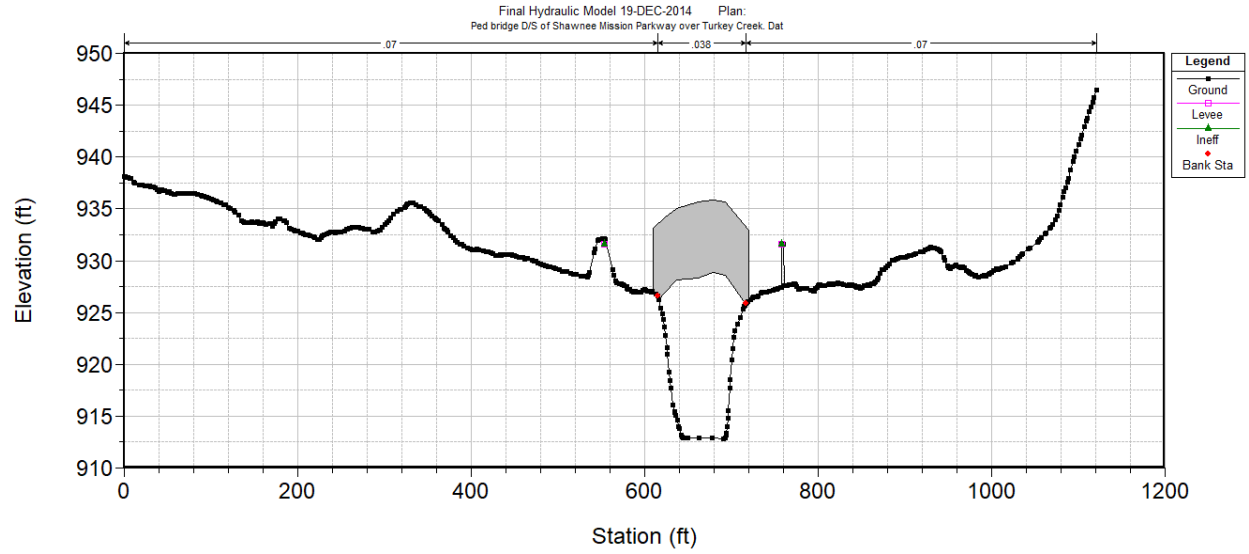
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 3.566670BR U Profile: PF 1

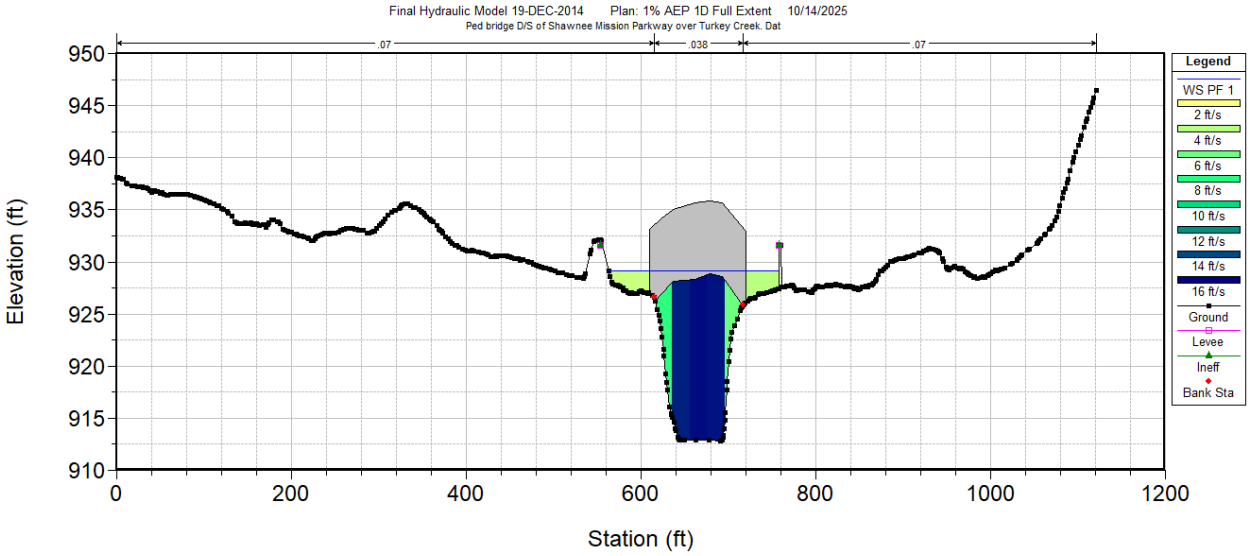
	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	556	618.97	195	71.16	52.63	1.17	1.64	2.74	1.72	4.7
2	Chan	618.97	640.36	2531	198.4	46.64	15.24	9.28	12.76	5.4	68.92
3	Chan	640.36	661.75	6018	322.34	42.79	36.25	15.07	18.67	9.57	178.62
4	Chan	661.75	683.14	5928	319.45	42.79	35.71	14.93	18.56	9.48	175.97
5	Chan	683.14	704.53	1653	155.06	47.69	9.96	7.25	10.66	4.13	44.03
6	Chan	704.53	725.92	166	34.69	35.66	1	1.63	4.77	1.24	5.9
7	ROB	725.92	766.14	110	46.83	40.23	0.66	1.19	2.35	1.48	3.47

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 3.566670BR U Profile: PF 1

E.G. Elev (ft)	932.66	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	3.82	**Wt. n-Val.**	0.1	0.038	0.1
W.S. Elev (ft)	928.84	**Reach Len. (ft)**	10	10	10
Crit W.S. (ft)	926.58	**Flow Area (sq ft)**	71.16	1029.94	46.83
E.G. Slope (ft/ft)	0.020345	**Area (sq ft)**	71.16	1029.94	46.83
Q Total (cfs)	16600	**Flow (cfs)**	194.69	16295.48	109.84
Top Width (ft)	86.77	**Top Width (ft)**	36.62	10.73	39.42
Vel Total (ft/s)	14.46	**Avg. Vel. (ft/s)**	2.74	15.82	2.35
Max Chl Dpth (ft)	15.65	**Hydr. Depth (ft)**	1.94	96	1.19
Conv. Total (cfs)	116380.6	**Conv. (cfs)**	1364.9	114245.7	770
Length Wtd. (ft)	10	**Wetted Per. (ft)**	52.63	215.57	40.23
Min Ch El (ft)	913.19	**Shear (lb/sq ft)**	1.72	6.07	1.48
Alpha	1.18	**Stream Power (lb/ft s)**	4.7	96.01	3.47
Frctn Loss (ft)	0.16	**Cum Volume (acre-ft)**	86.98	273.21	12.99
C & E Loss (ft)	0.44	**Cum SA (acres)**	15.47	16.61	2.89

E.5.2.3 Downstream Bridge Face





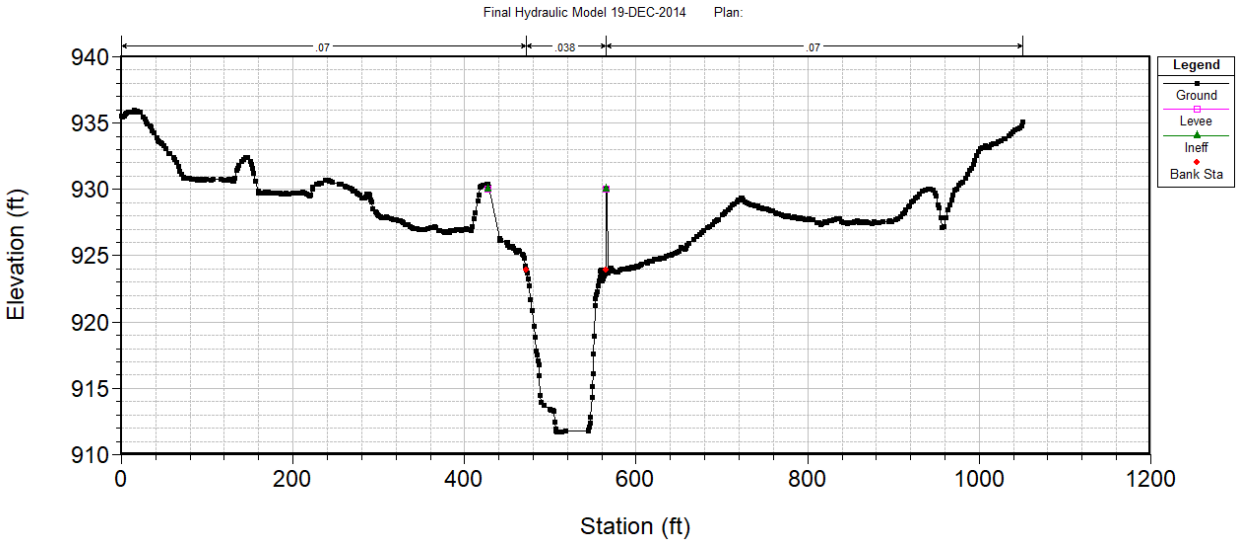
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 3.566670BR D Profile: PF 1

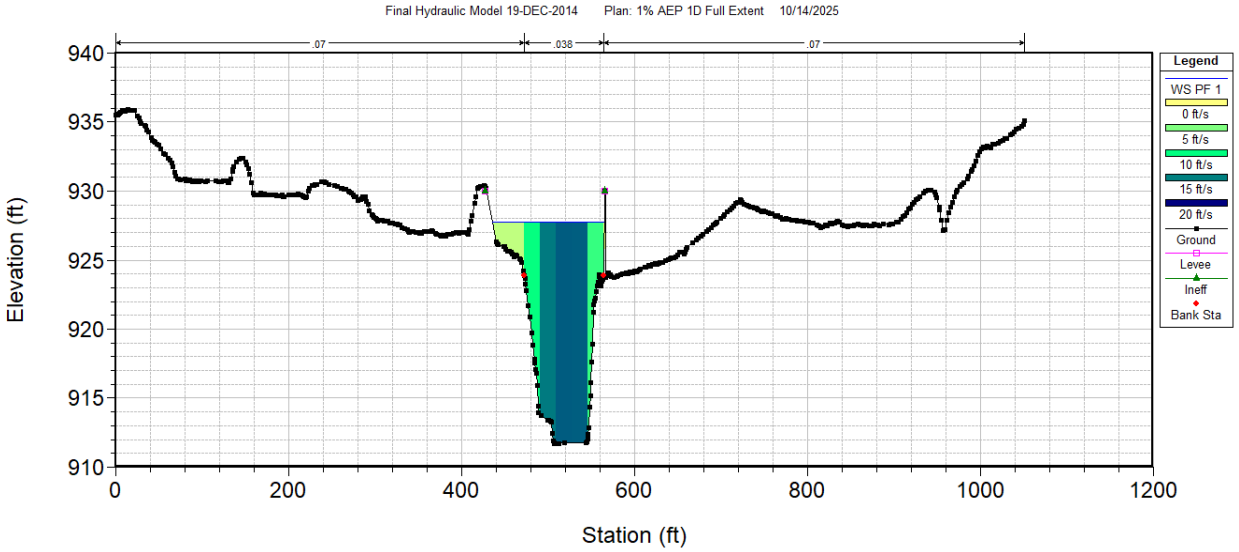
	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	553.98	614.73	285.83	82.58	49.42	1.72	1.76	3.46	1.4	4.84
2	Chan	614.73	635.2	912.5	116.16	42.05	5.5	6.01	7.86	2.31	18.16
3	Chan	635.2	655.68	4609.34	304.69	41.27	27.77	14.88	15.13	6.18	93.47
4	Chan	655.68	676.15	5020.97	319.74	40.95	30.25	15.62	15.7	6.53	102.61
5	Chan	676.15	696.63	4953.57	319.82	41.82	29.84	15.62	15.49	6.4	99.14
6	Chan	696.63	717.1	480.19	80.07	43.45	2.89	4.07	6	1.54	9.25
7	ROB	717.1	758.29	337.6	86.45	43.16	2.03	2.24	3.91	1.68	6.55

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 3.566670BR D Profile: PF 1

E.G. Elev (ft)	932.06	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	2.94	**Wt. n-Val.**	0.07	0.038	0.07
W.S. Elev (ft)	929.11	**Reach Len. (ft)**	8.54	8.54	8.54
Crit W.S. (ft)	925.96	**Flow Area (sq ft)**	82.58	1140.48	86.45
E.G. Slope (ft/ft)	0.013406	**Area (sq ft)**	82.58	1140.48	86.45
Q Total (cfs)	16600	**Flow (cfs)**	285.83	15976.57	337.6
Top Width (ft)	85.56	**Top Width (ft)**	47.01		38.55
Vel Total (ft/s)	12.68	**Avg. Vel. (ft/s)**	3.46	14.01	3.91
Max Chl Dpth (ft)	16.27	**Hydr. Depth (ft)**	1.76		2.24
Conv. Total (cfs)	143371.5	**Conv. (cfs)**	2468.7	137987	2915.8
Length Wtd. (ft)	8.54	**Wetted Per. (ft)**	49.42	209.55	43.16
Min Ch El (ft)	912.84	**Shear (lb/sq ft)**	1.4	4.56	1.68
Alpha	1.18	**Stream Power (lb/ft s)**	4.84	63.81	6.55
Frctn Loss (ft)	0.06	**Cum Volume (acre-ft)**	86.97	272.96	12.97
C & E Loss (ft)	0.29	**Cum SA (acres)**	15.46	16.61	2.89

E.5.2.4 Downstream Exit Cross-Section





Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.836 Profile: PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	427.41	472.33	186.4	75.58	36.9	1.12	2.08	2.47	0.66	1.64
2	Chan	472.33	490.78	1586.02	160.85	21.51	9.55	8.72	9.86	2.42	23.89
3	Chan	490.78	509.23	4122.31	270.65	18.85	24.83	14.67	15.23	4.65	70.86
4	Chan	509.23	527.69	4852.83	295.96	18.45	29.23	16.04	16.4	5.2	85.21
5	Chan	527.69	546.14	4813.68	295.31	18.57	29	16	16.3	5.15	83.96
6	Chan	546.14	564.59	1038.34	130.93	24.27	6.26	7.1	7.93	1.75	13.86
7	ROB	564.59	565.08	0.43	0.81	3.97	0	2.4	0.53	0.07	0.04

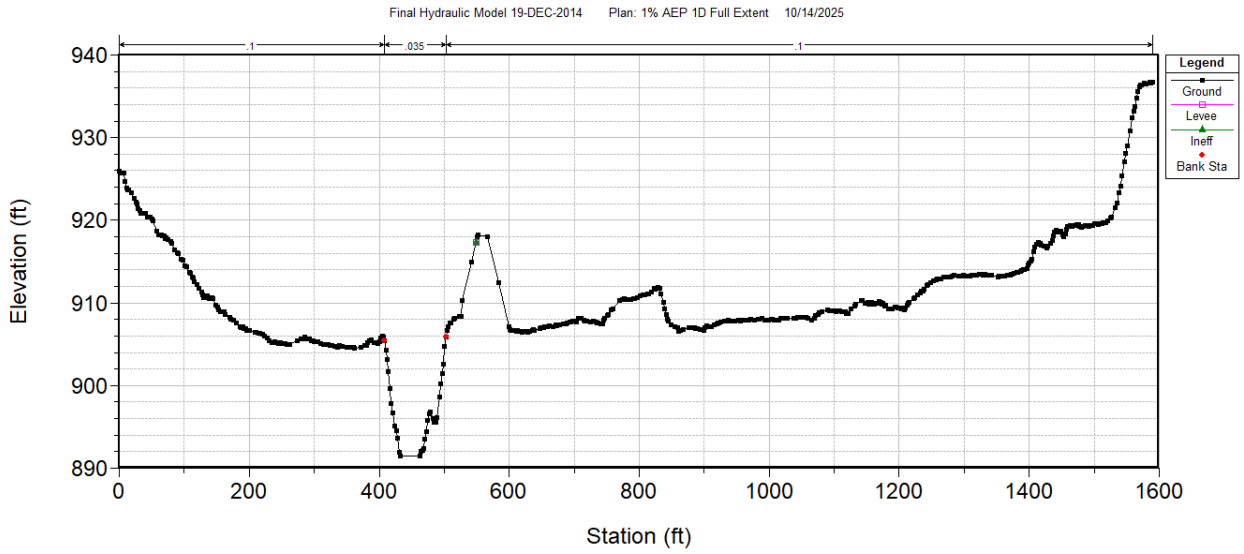
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.836 Profile: PF 1

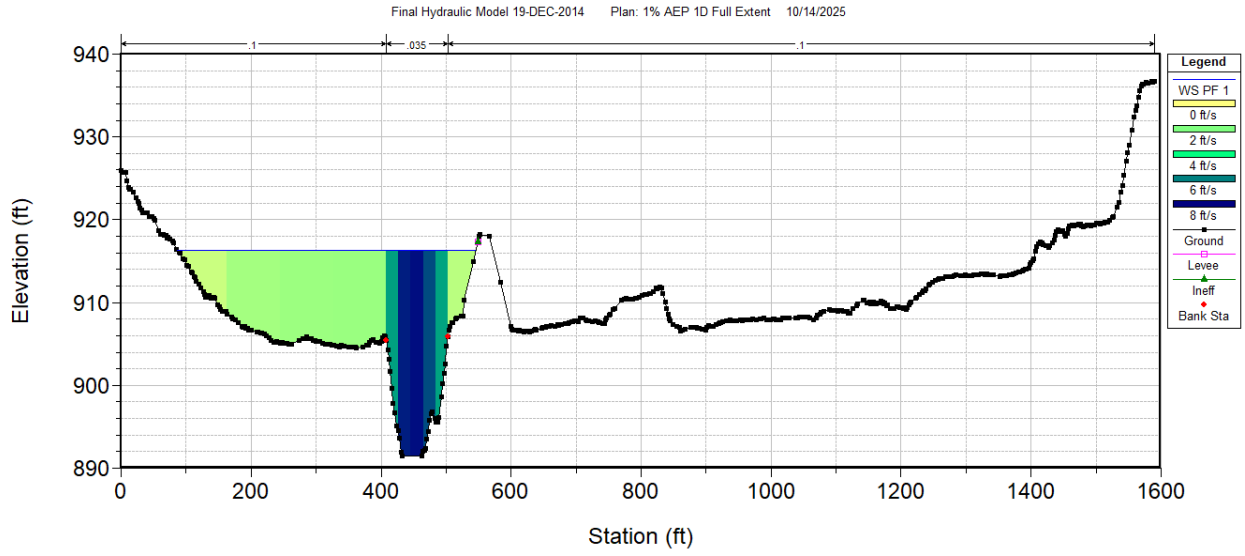
E.G. Elev (ft)	930.89	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	3.11	**Wt. n-Val.**	0.07	0.038	0.07
W.S. Elev (ft)	927.78	**Reach Len. (ft)**	283.5	306.5	307.5
Crit W.S. (ft)	925.34	**Flow Area (sq ft)**	75.58	1153.7	0.81
E.G. Slope (ft/ft)	0.005189	**Area (sq ft)**	75.58	1153.7	0.81
Q Total (cfs)	16600	**Flow (cfs)**	186.4	16413.17	0.43
Top Width (ft)	129.01	**Top Width (ft)**	36.41	92.26	0.34
Vel Total (ft/s)	13.49	**Avg. Vel. (ft/s)**	2.47	14.23	0.53
Max Chl Dpth (ft)	16.06	**Hydr. Depth (ft)**	2.08	12.5	2.4
Conv. Total (cfs)	230433.5	**Conv. (cfs)**	2587.5	227840	6
Length Wtd. (ft)	306.05	**Wetted Per. (ft)**	36.9	101.65	3.97
Min Ch El (ft)	911.72	**Shear (lb/sq ft)**	0.66	3.68	0.07
Alpha	1.1	**Stream Power (lb/ft s)**	1.64	52.31	0.04
Frctn Loss (ft)	2.07	**Cum Volume (acre-ft)**	86.65	268.98	12.8
C & E Loss (ft)	0.09	**Cum SA (acres)**	15.3	16.3	2.82

E.5.3 Merriam Marketplace Pedestrian Bridge

Bridge Opening Width (ft)		133
Floodway Width (ft)		400
Bottom Deck Elevation (ft)	**Max**	913.95
	Min	911.53

E.5.3.1 Upstream Approach Cross-Section





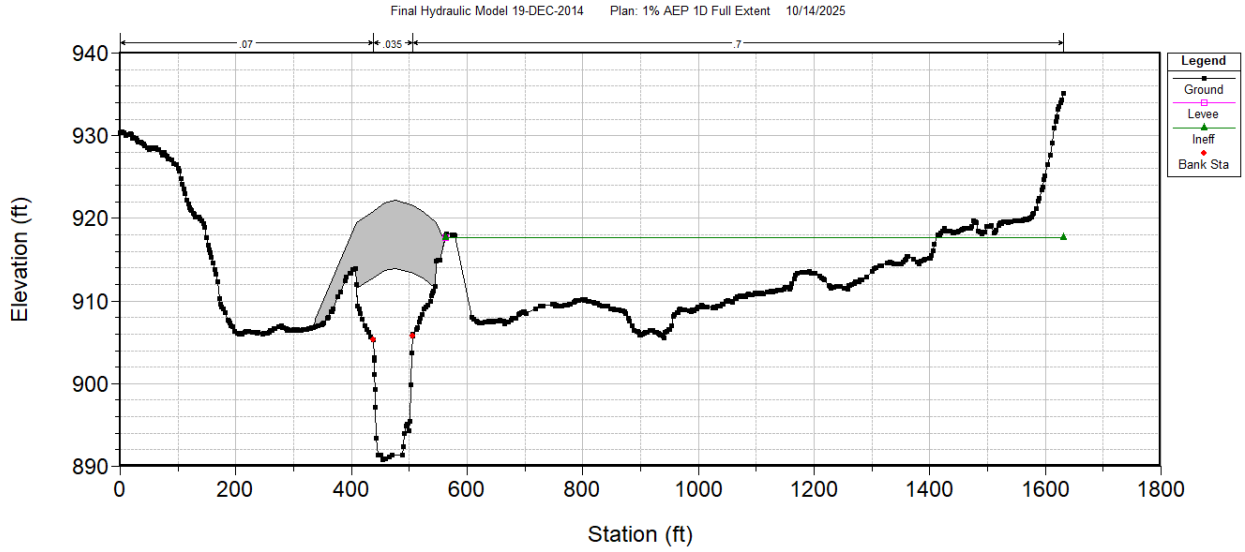
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.906 Profile: PF 1

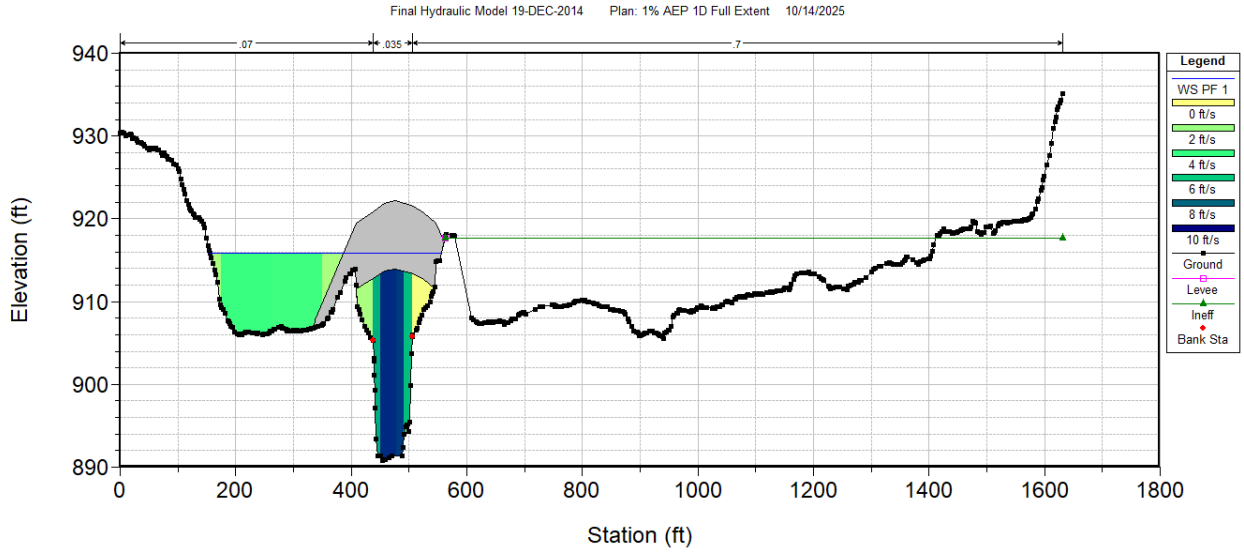
	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	81.68	163.36	274.89	332.56	77.79	1.53	4.31	0.83	0.13	0.11
2	LOB	163.36	245.04	1132.93	793.72	81.82	6.29	9.72	1.43	0.3	0.43
3	LOB	245.04	326.72	1413.25	905.92	81.73	7.85	11.09	1.56	0.34	0.54
4	LOB	326.72	408.4	1480.74	932.4	81.9	8.23	11.42	1.59	0.35	0.56
5	Chan	408.4	427.26	1776.67	323.84	22.23	9.87	17.17	5.49	0.45	2.48
6	Chan	427.26	446.12	3492.56	462.66	19.68	19.4	24.53	7.55	0.73	5.5
7	Chan	446.12	464.98	3648.7	467.75	18.94	20.27	24.8	7.8	0.77	5.97
8	Chan	464.98	483.84	2769.82	406.03	20.1	15.39	21.53	6.82	0.63	4.27
9	Chan	483.84	502.7	1720.48	316.85	22.09	9.56	16.8	5.43	0.44	2.42
10	ROB	502.7	548.95	289.96	268.7	45.68	1.61	6.2	1.08	0.18	0.2

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.906 Profile: PF 1

E.G. Elev (ft)	916.89	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	0.54	**Wt. n-Val.**	0.1	0.035	0.1
W.S. Elev (ft)	916.34	**Reach Len. (ft)**	146.8	122.1	85.4
Crit W.S. (ft)	906.99	**Flow Area (sq ft)**	2964.59	1977.13	268.7
E.G. Slope (ft/ft)	0.000497	**Area (sq ft)**	2964.59	1977.13	268.7
Q Total (cfs)	18000	**Flow (cfs)**	4301.81	13408.23	289.96
Top Width (ft)	459.89	**Top Width (ft)**	322.25	94.3	43.34
Vel Total (ft/s)	3.45	**Avg. Vel. (ft/s)**	1.45	6.78	1.08
Max Chl Dpth (ft)	24.84	**Hydr. Depth (ft)**	9.2	20.97	6.2
Conv. Total (cfs)	807634.9	**Conv. (cfs)**	193016.2	601608.6	13010.2
Length Wtd. (ft)	130.62	**Wetted Per. (ft)**	323.23	103.04	45.68
Min Ch El (ft)	891.5	**Shear (lb/sq ft)**	0.28	0.6	0.18
Alpha	2.91	**Stream Power (lb/ft s)**	0.41	4.04	0.2
Frctn Loss (ft)	0.04	**Cum Volume (acre-ft)**	62.67	153.43	8.32
C & E Loss (ft)	0.11	**Cum SA (acres)**	11.05	9.23	1.78

E.5.3.2 Upstream Bridge Face





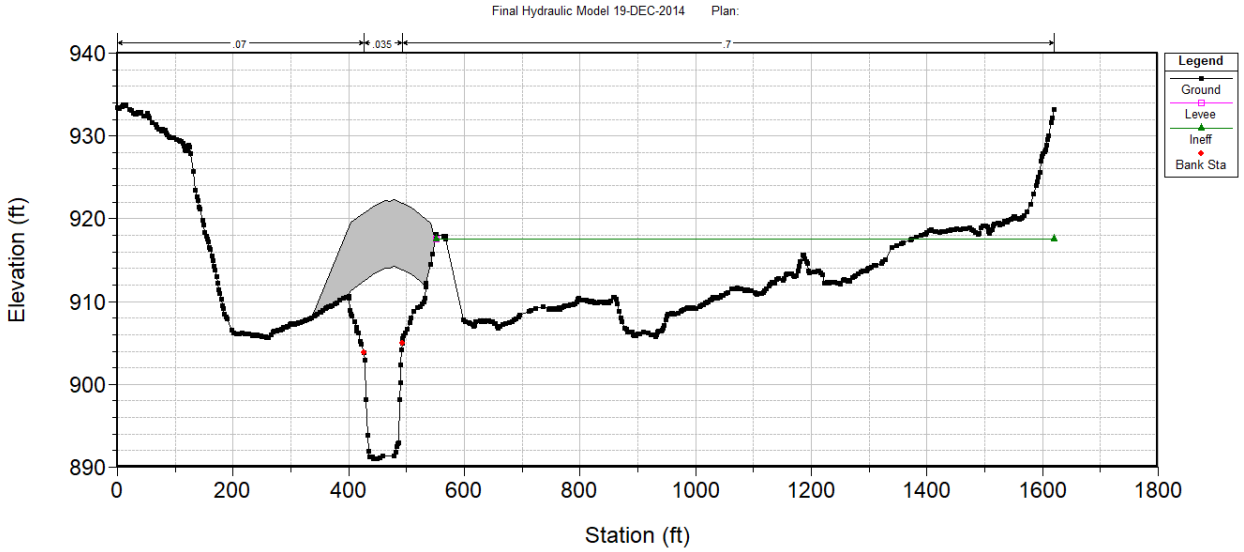
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.867445BR U Profile: PF 1

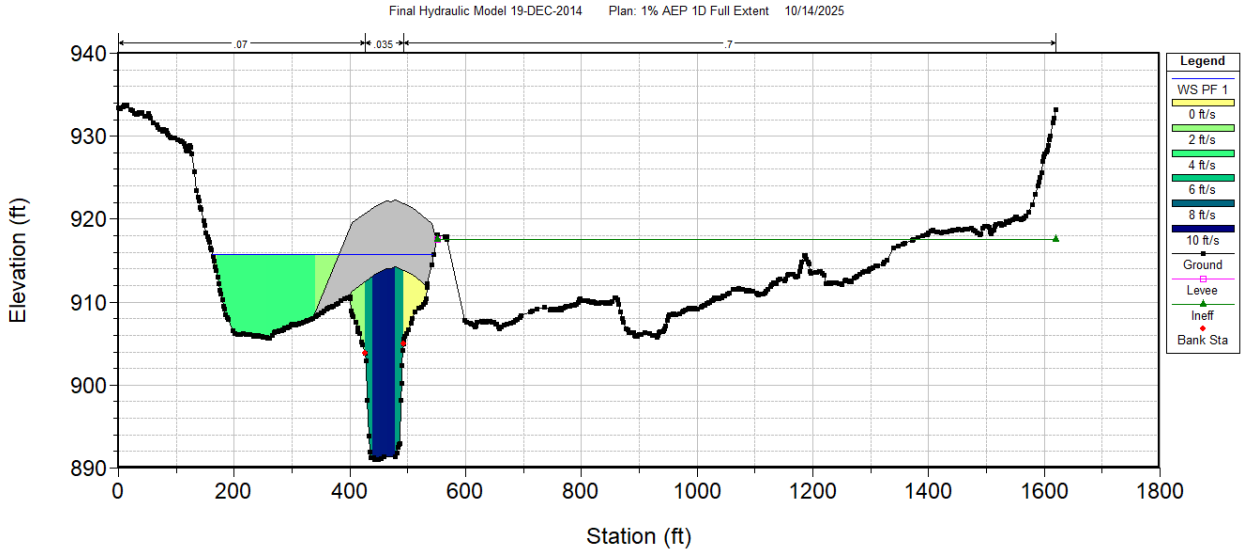
	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	87.45	174.89	98.8	57.03	20.56	0.55	2.94	1.73	0.34	0.59
2	LOB	174.89	262.34	3150.65	813.4	87.73	17.5	9.3	3.87	1.13	4.39
3	LOB	262.34	349.78	2946.86	782.49	88.03	16.37	8.95	3.77	1.09	4.1
4	LOB	349.78	437.23	415.06	248.9	95.03	2.31	3.82	1.67	0.32	0.53
5	Chan	437.23	450.92	1608.46	245.06	35.7	8.94	17.9	6.56	0.84	5.51
6	Chan	450.92	464.61	2853.08	311.24	27.47	15.85	22.73	9.17	1.39	12.71
7	Chan	464.61	478.31	2834.86	309.7	27.39	15.75	22.62	9.15	1.38	12.66
8	Chan	478.31	492	2675.06	302.65	28.21	14.86	22.1	8.84	1.31	11.6
9	Chan	492	505.69	1394.23	222.57	34.77	7.75	16.26	6.26	0.78	4.91
10	ROB	505.69	563.41	22.94	154	77.16	0.13	4.02	0.15	0.24	0.04

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.867445BR U Profile: PF 1

E.G. Elev (ft)	916.62	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	0.72	**Wt. n-Val.**	0.07	0.035	0.7
W.S. Elev (ft)	915.9	**Reach Len. (ft)**	11	11	11
Crit W.S. (ft)	908.02	**Flow Area (sq ft)**	1901.83	1391.22	154
E.G. Slope (ft/ft)	0.00196	**Area (sq ft)**	1901.83	1391.22	154
Q Total (cfs)	18000	**Flow (cfs)**	6611.37	11365.69	22.94
Top Width (ft)	231.46	**Top Width (ft)**	231.46		
Vel Total (ft/s)	5.22	**Avg. Vel. (ft/s)**	3.48	8.17	0.15
Max Chl Dpth (ft)	25.13	**Hydr. Depth (ft)**	8.22		
Conv. Total (cfs)	406566.3	**Conv. (cfs)**	149331.2	256716.9	518.2
Length Wtd. (ft)	11	**Wetted Per. (ft)**	291.36	153.53	77.16
Min Ch El (ft)	890.77	**Shear (lb/sq ft)**	0.8	1.11	0.24
Alpha	1.71	**Stream Power (lb/ft s)**	2.78	9.06	0.04
Frctn Loss (ft)	0.02	**Cum Volume (acre-ft)**	41.25	144.4	7.32
C & E Loss (ft)	0.03	**Cum SA (acres)**	8.91	8.82	1.61

E.5.3.3 Downstream Bridge Face





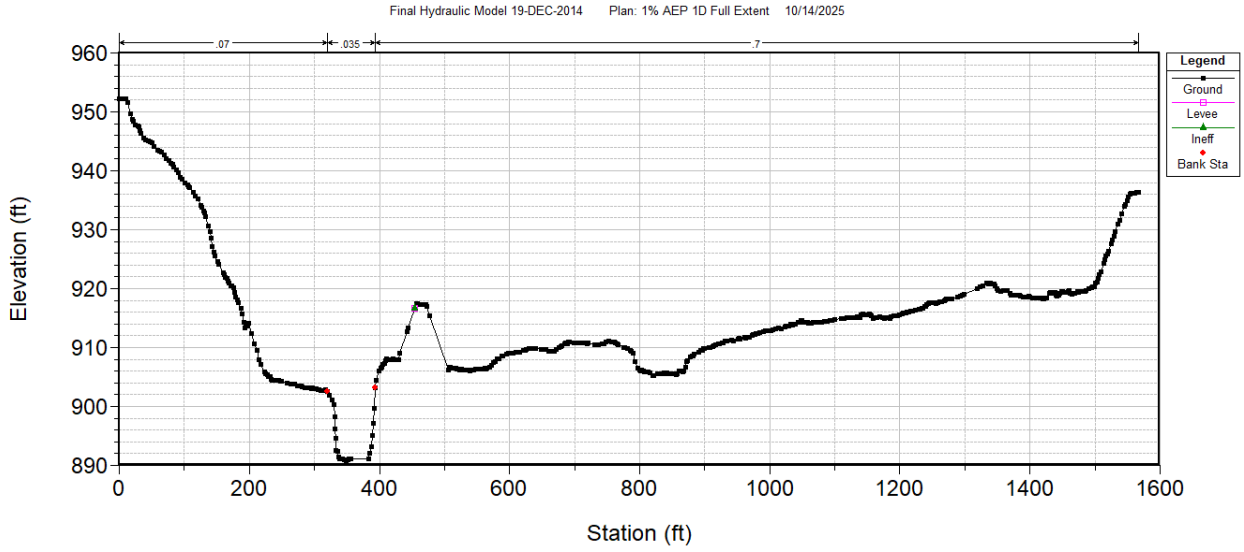
Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.867445BR D Profile: PF 1

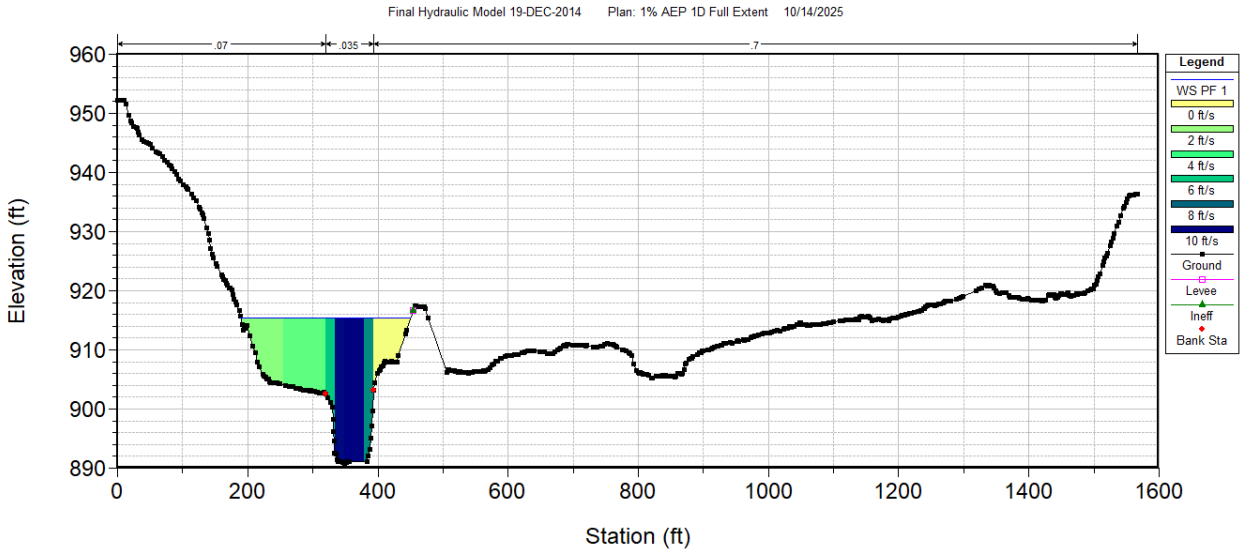
	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	85.29	170.58	9.67	9.67	8.53	0.05	1.18	1	0.15	0.15
2	LOB	170.58	255.87	2858.26	740.98	86.29	15.88	8.69	3.86	1.13	4.36
3	LOB	255.87	341.16	2874.67	740.67	85.46	15.97	8.68	3.88	1.14	4.42
4	LOB	341.16	426.45	487.92	267.32	95.65	2.71	4	1.83	0.37	0.67
5	Chan	426.45	439.62	1551.39	227.94	33.33	8.62	17.31	6.81	0.9	6.12
6	Chan	439.62	452.79	2801.29	295.88	26.37	15.56	22.47	9.47	1.48	13.97
7	Chan	452.79	465.95	2846.53	298.61	26.34	15.81	22.68	9.53	1.49	14.21
8	Chan	465.95	479.12	2868.07	299.96	26.34	15.93	22.78	9.56	1.5	14.32
9	Chan	479.12	492.29	1669.86	242.18	34.73	9.28	18.39	6.9	0.92	6.32
10	ROB	492.29	551.21	32.35	191.37	83.71	0.18	4.63	0.17	0.3	0.05

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.867445BR D Profile: PF 1

E.G. Elev (ft)	916.57	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	0.82	**Wt. n-Val.**	0.07	0.035	0.7
W.S. Elev (ft)	915.75	**Reach Len. (ft)**	9.31	9.31	9.31
Crit W.S. (ft)	907.33	**Flow Area (sq ft)**	1758.65	1364.58	191.37
E.G. Slope (ft/ft)	0.002106	**Area (sq ft)**	1758.65	1364.58	191.37
Q Total (cfs)	18000	**Flow (cfs)**	6230.52	11737.13	32.35
Top Width (ft)	218.75	**Top Width (ft)**	218.75		
Vel Total (ft/s)	5.43	**Avg. Vel. (ft/s)**	3.54	8.6	0.17
Max Chl Dpth (ft)	24.78	**Hydr. Depth (ft)**	8.04		
Conv. Total (cfs)	392205.4	**Conv. (cfs)**	135757.9	255742.6	704.9
Length Wtd. (ft)	9.31	**Wetted Per. (ft)**	275.93	147.12	83.71
Min Ch El (ft)	890.97	**Shear (lb/sq ft)**	0.84	1.22	0.3
Alpha	1.78	**Stream Power (lb/ft s)**	2.97	10.49	0.05
Frctn Loss (ft)	0.01	**Cum Volume (acre-ft)**	40.78	144.05	7.28
C & E Loss (ft)	0.01	**Cum SA (acres)**	8.91	8.82	1.61

E.5.3.4 Downstream Exit Cross-Section





Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.836 Profile: PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)	Shear (lb/sq ft)	Power (lb/ft s)
1	LOB	127.8	191.7	0.54	1.35	2.5	0	0.62	0.4	0.03	0.01
2	LOB	191.7	255.6	1153.86	498.77	65.8	6.41	7.81	2.31	0.41	0.95
3	LOB	255.6	319.5	2445.73	773.9	63.95	13.59	12.11	3.16	0.65	2.06
4	Chan	319.5	334.3	1380.36	228.4	19.98	7.67	15.43	6.04	0.62	3.73
5	Chan	334.3	349.1	3498.61	356.27	15.04	19.44	24.07	9.82	1.28	12.55
6	Chan	349.1	363.9	3587.34	359.39	14.81	19.93	24.28	9.98	1.31	13.08
7	Chan	363.9	378.7	3572.52	358.42	14.8	19.85	24.22	9.97	1.31	13.03
8	Chan	378.7	393.5	2282.31	315.32	21.04	12.68	21.31	7.24	0.81	5.85
9	ROB	393.5	454.68	78.75	371.38	59.31	0.44	6.5	0.21	0.34	0.07

Plan: 1% AEP 1D Full Extent River 1 Below Trib RS: 2.836 Profile: PF 1

E.G. Elev (ft)	916.37	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (ft)	0.99	**Wt. n-Val.**	0.07	0.035	0.7
W.S. Elev (ft)	915.38	**Reach Len. (ft)**	159.7	119.3	94.9
Crit W.S. (ft)	906.65	**Flow Area (sq ft)**	1274.02	1617.81	371.38
E.G. Slope (ft/ft)	0.000865	**Area (sq ft)**	1274.02	1617.81	371.38
Q Total (cfs)	18000	**Flow (cfs)**	3600.12	14321.13	78.75
Top Width (ft)	261.08	**Top Width (ft)**	129.98	74	57.11
Vel Total (ft/s)	5.52	**Avg. Vel. (ft/s)**	2.83	8.85	0.21
Max Chl Dpth (ft)	24.51	**Hydr. Depth (ft)**	9.8	21.86	6.5
Conv. Total (cfs)	612187.8	**Conv. (cfs)**	122441.6	487067.8	2678.2
Length Wtd. (ft)	124.69	**Wetted Per. (ft)**	132.25	85.67	59.31
Min Ch El (ft)	890.87	**Shear (lb/sq ft)**	0.52	1.02	0.34
Alpha	2.1	**Stream Power (lb/ft s)**	1.47	9.02	0.07
Frctn Loss (ft)	0.13	**Cum Volume (acre-ft)**	34.02	138.29	5.95
C & E Loss (ft)	0.06	**Cum SA (acres)**	8.05	8.57	1.41

APPENDIX E.6 – HYDRAULIC MODEL OUTPUTS

Introduction

Upstream Tributary (Channel STA 6+00)

Shawnee Mission Parkway Pedestrian Bridge (Channel STA 14+50)

Johnson Drive Bridge (Channel STA 41+00)

Merriam Marketplace Pedestrian Bridge (Channel STA 51+50)

Merriam Drive Bridge (Channel STA 63+50)

E.6 HYDRAULIC MODEL OUTPUTS

E.6.1 Introduction

The following sections provides detailed model outputs not already covered in Appendices E.3 through E.5. This includes flow hydrographs at key locations and depth grids for the FWOP and with project conditions. It should be noted that the exact water surface elevations vary along the channel cross section due to superelevation, but the plots are meant to show the change between FWOP and 50% design conditions as well as the duration of high water.

E.6.2 Upstream Tributary (Channel STA 6+00)

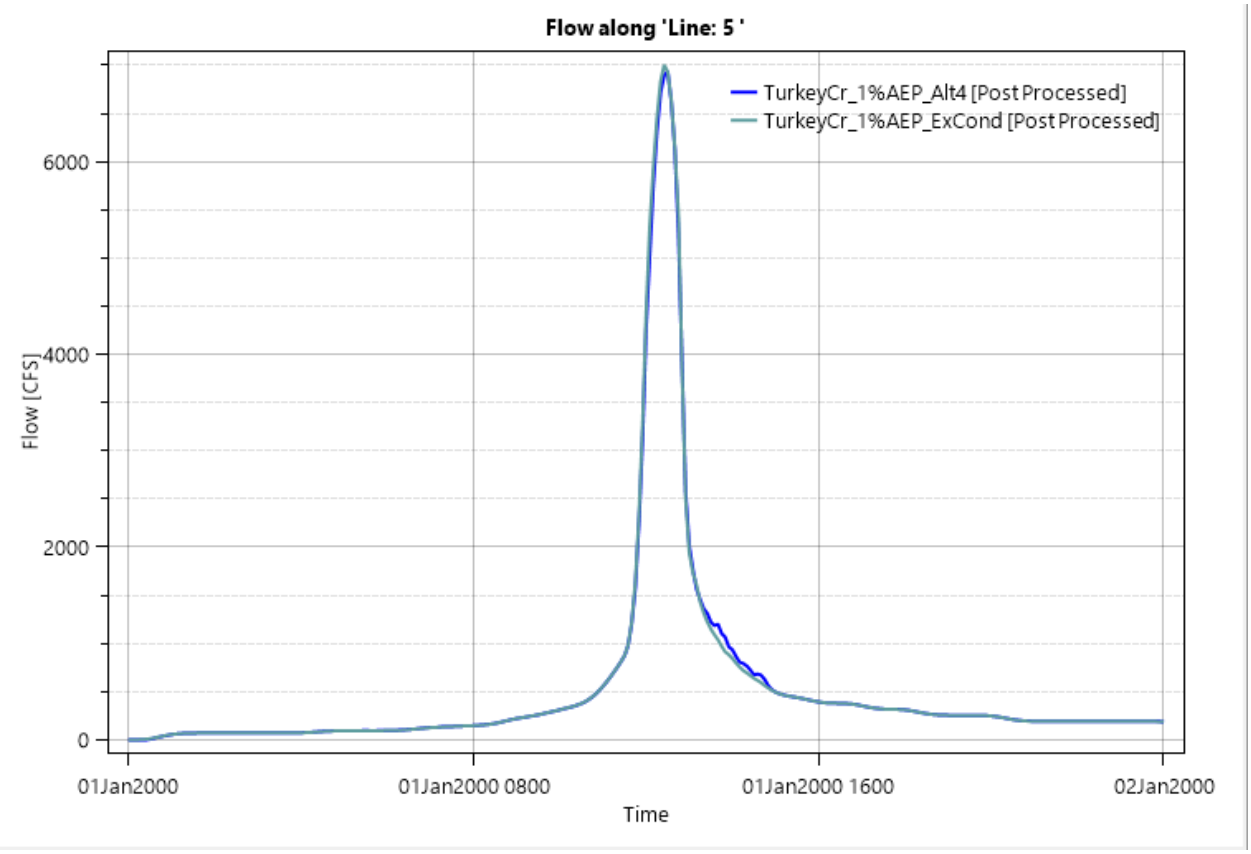


Figure 1. Flow hydrograph for upstream tributary (Channel STA 6+00) for FWOP and 50% Design

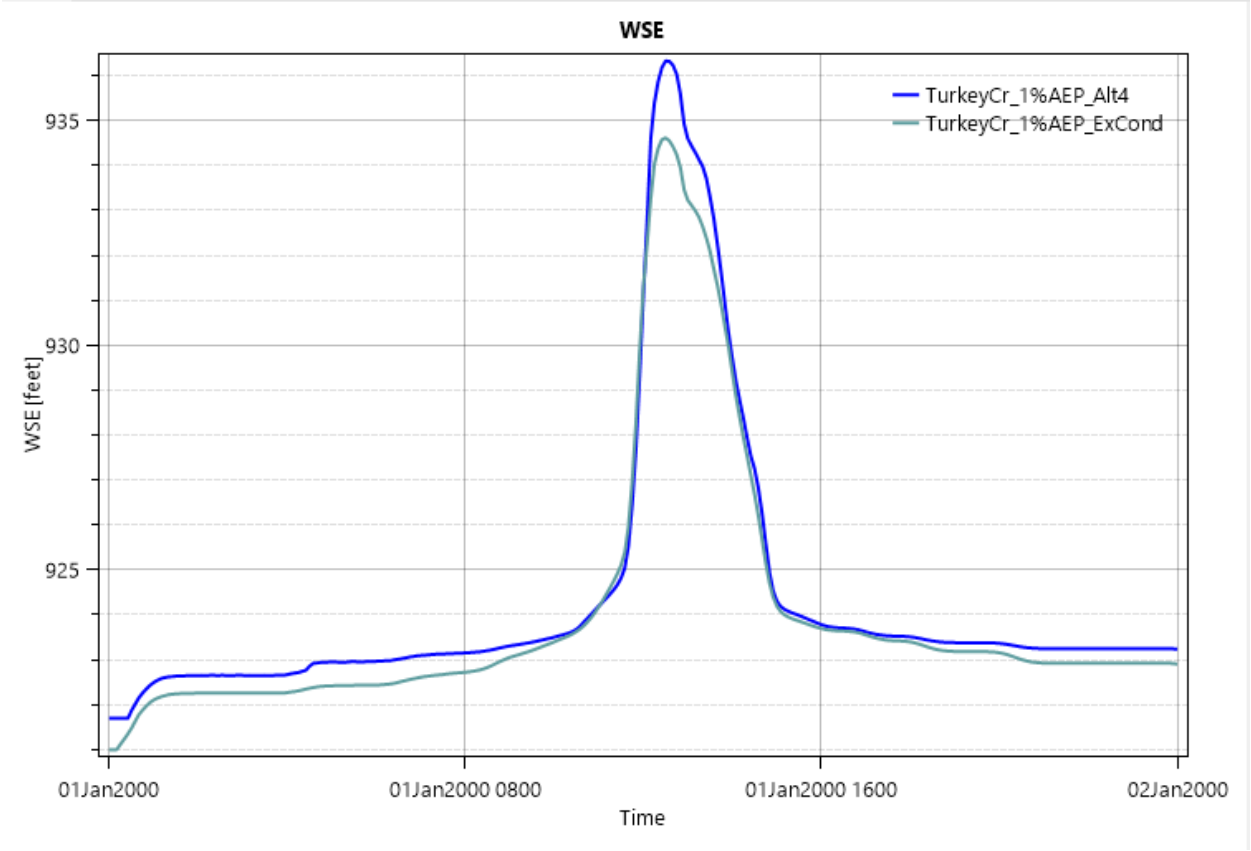


Figure 2. WSE hydrograph for upstream tributary (Channel STA 6+00) for FWOP and 50% Design

E.6.3 Shawnee Mission Parkway Pedestrian Bridge (Channel STA 14+50)

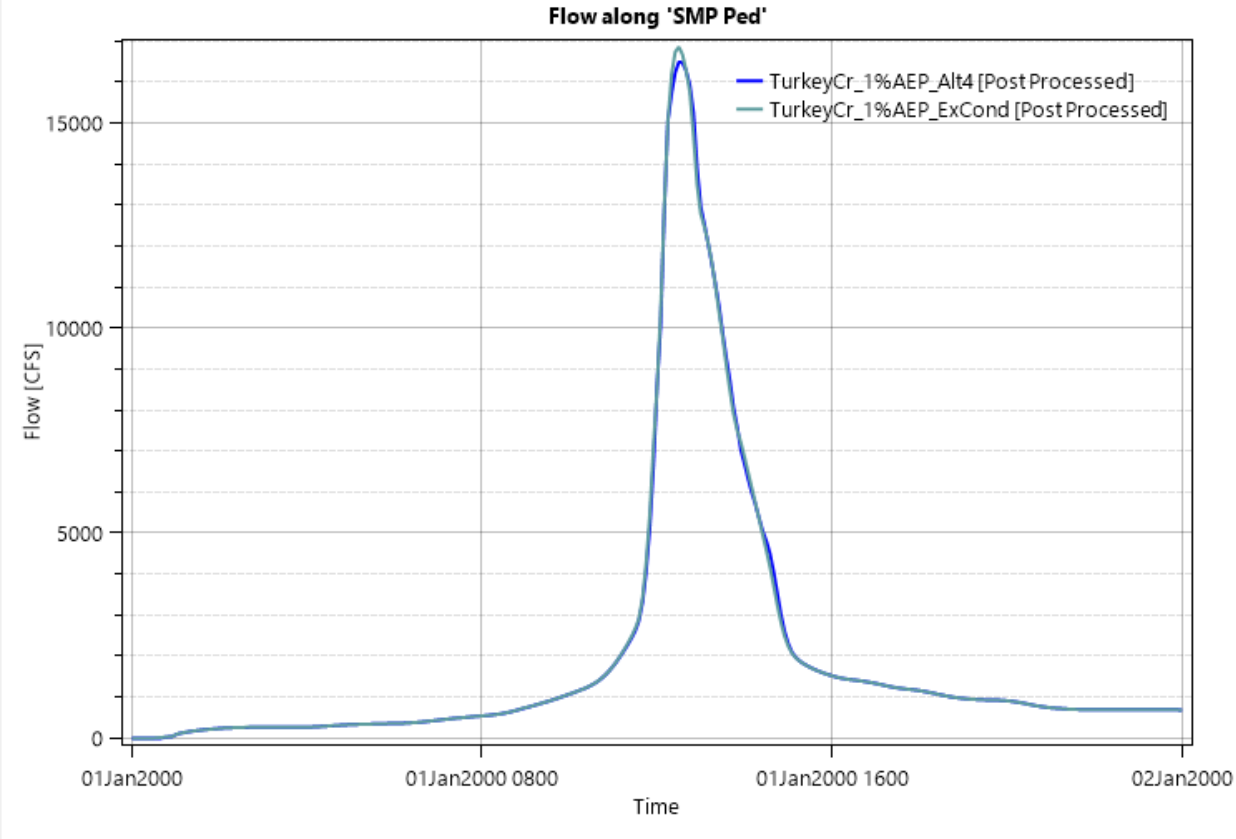


Figure 3. Flow hydrograph for SMP Pedestrian Bridge (Channel STA 14+50) for FWOP and 50% Design

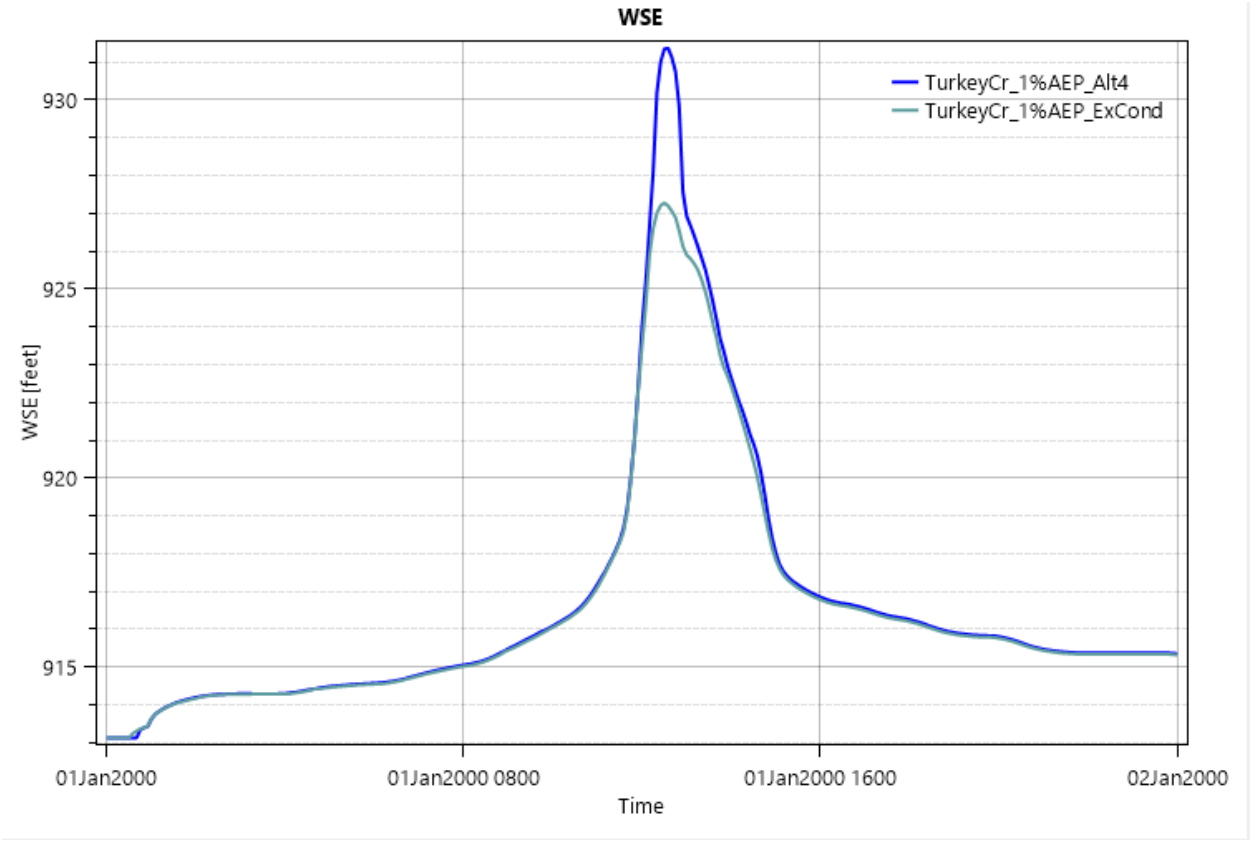


Figure 4. WSE hydrograph for SMP Pedestrian Bridge (Channel STA 14+50) for FWOP and 50% Design

E.6.4 Johnson Drive Bridge (Channel STA 41+00)

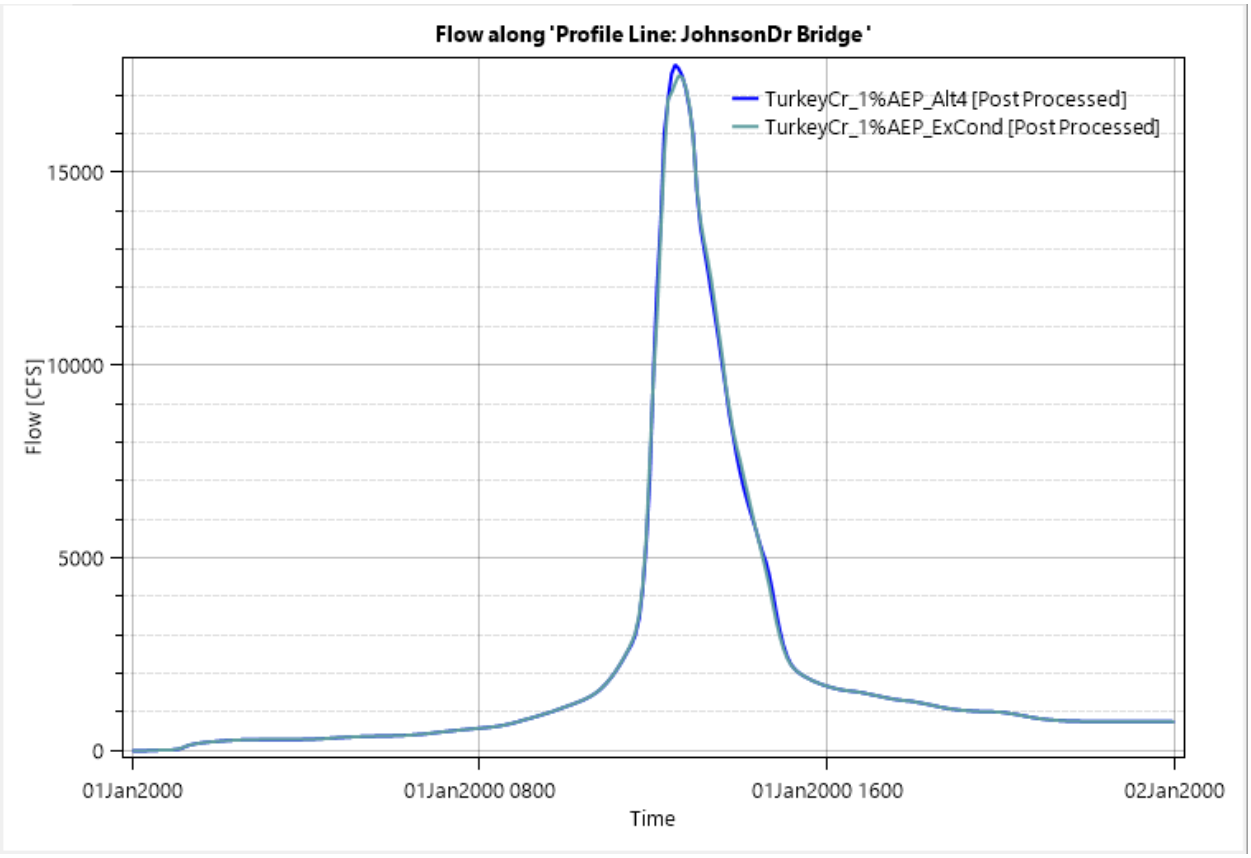


Figure 5. Flow hydrograph for Johnson Drive Bridge (Channel STA 41+00) for FWOP and 50% Design

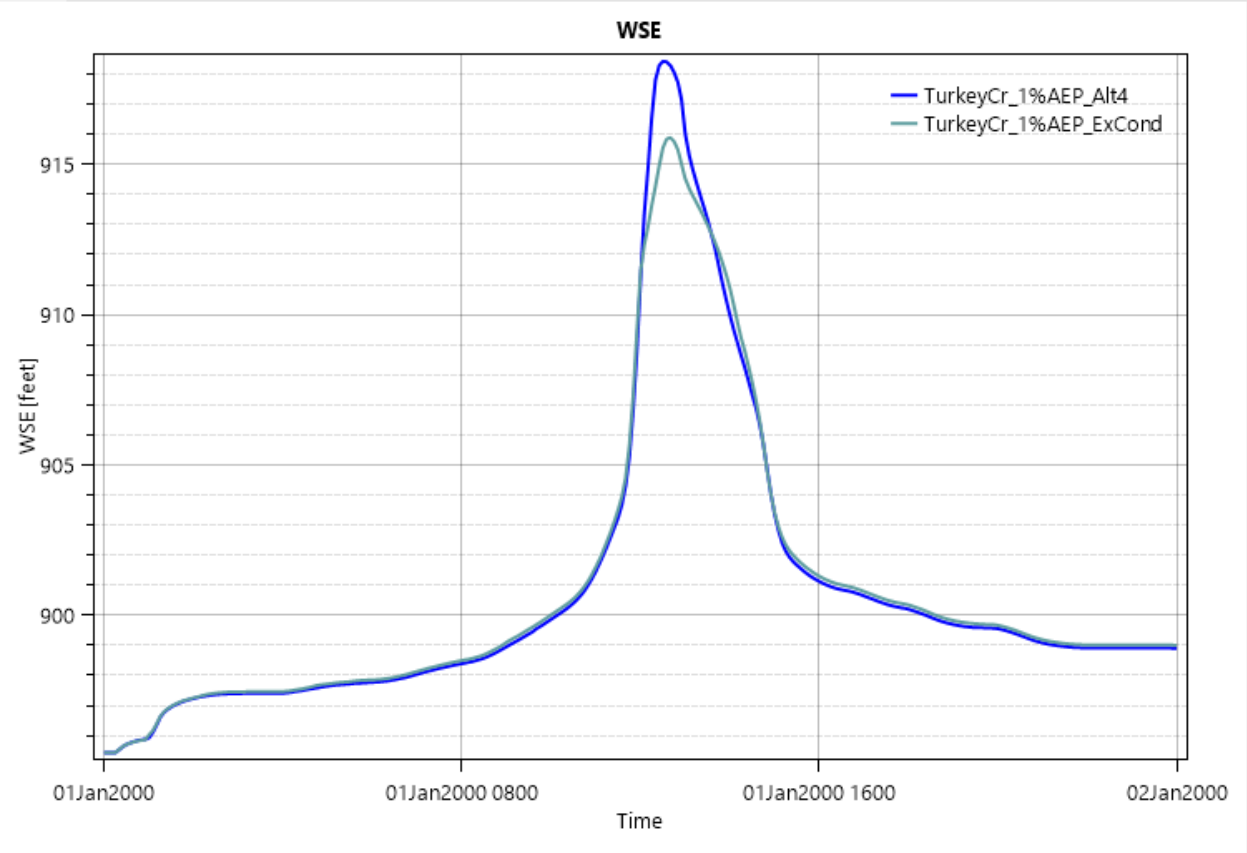


Figure 6. WSE hydrograph for Johnson Drive Bridge (Channel STA 41+00) for FWOP and 50% Design

E.6.5 Merriam Marketplace Pedestrian Bridge (Channel STA 51+50)

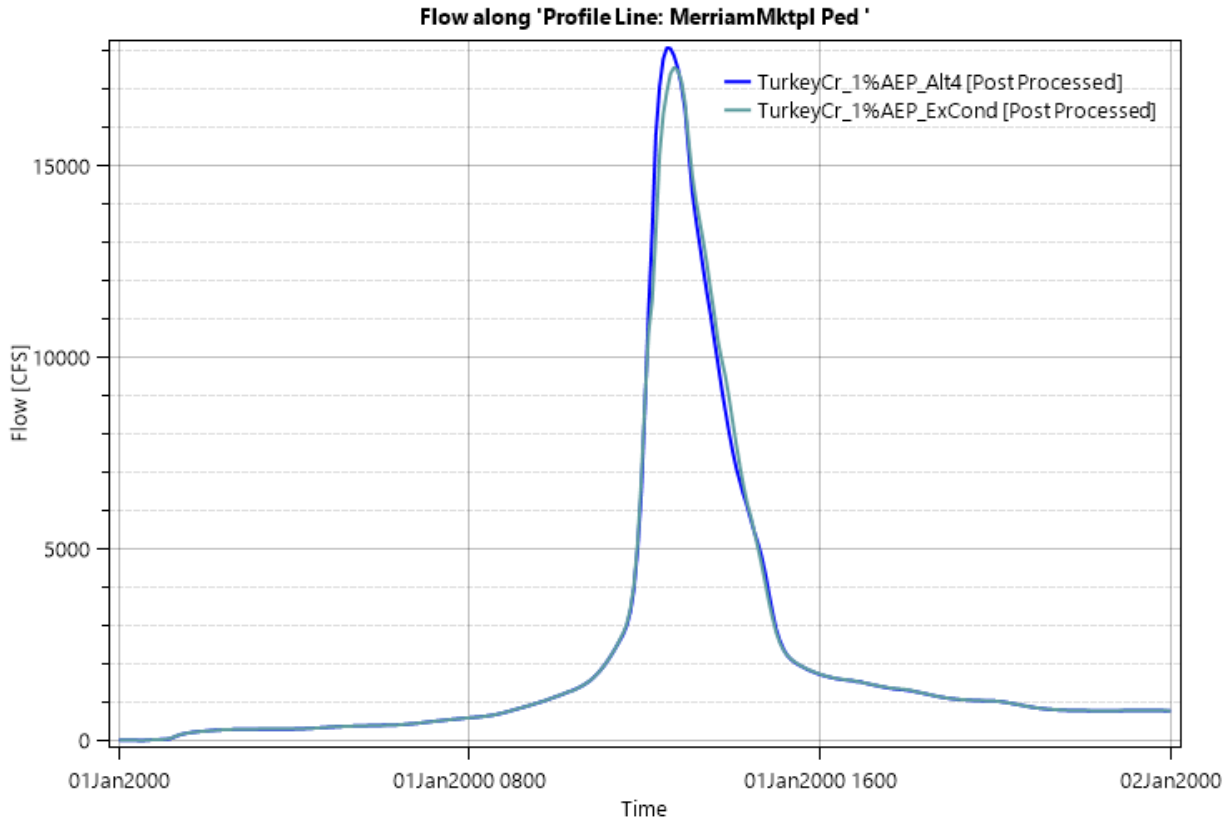


Figure 7. Flow hydrograph for Merriam Marketplace Bridge (Channel STA 51+50) for FWOP and 50% Design

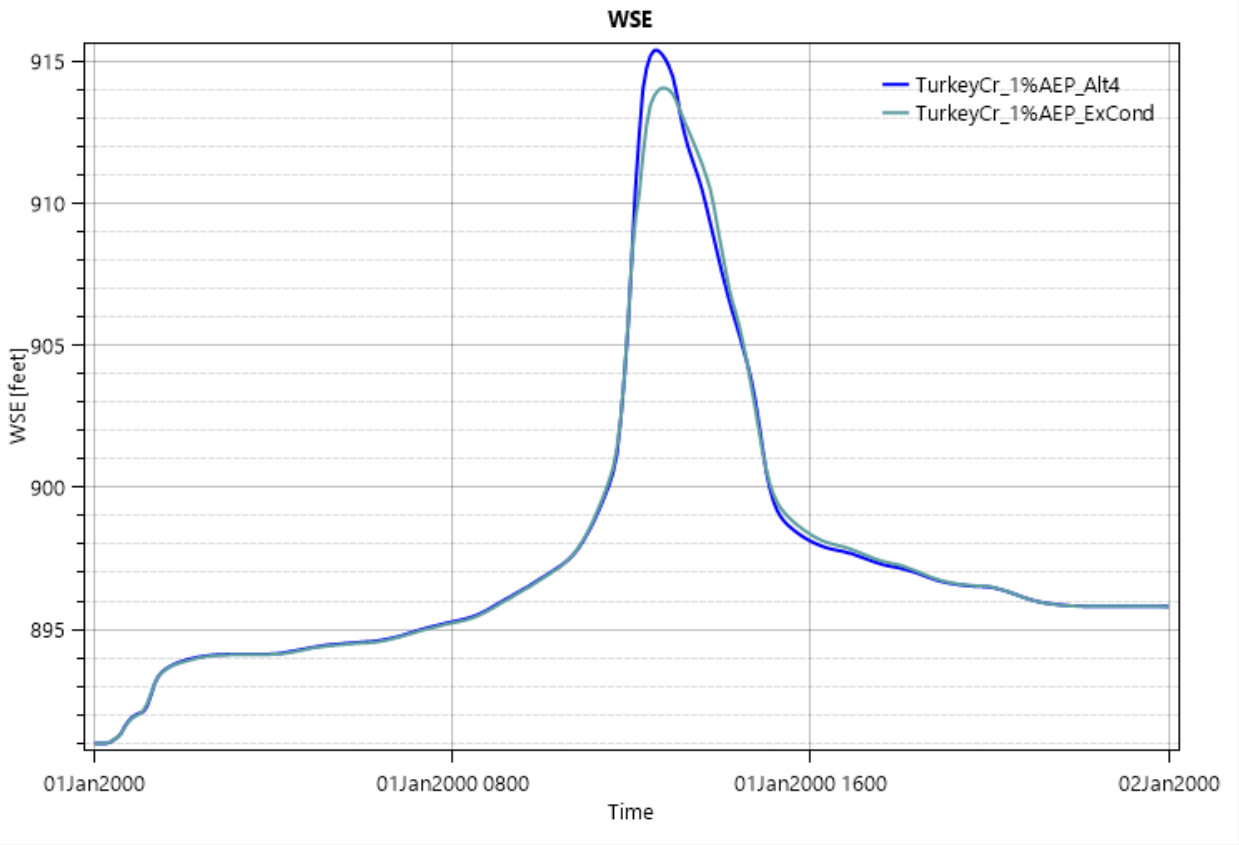


Figure 8. WSE hydrograph for Merriam Marketplace Bridge (Channel STA 51+50) for FWOP and 50% Design

E.6.6 Merriam Drive Bridge (Channel STA 63+50)

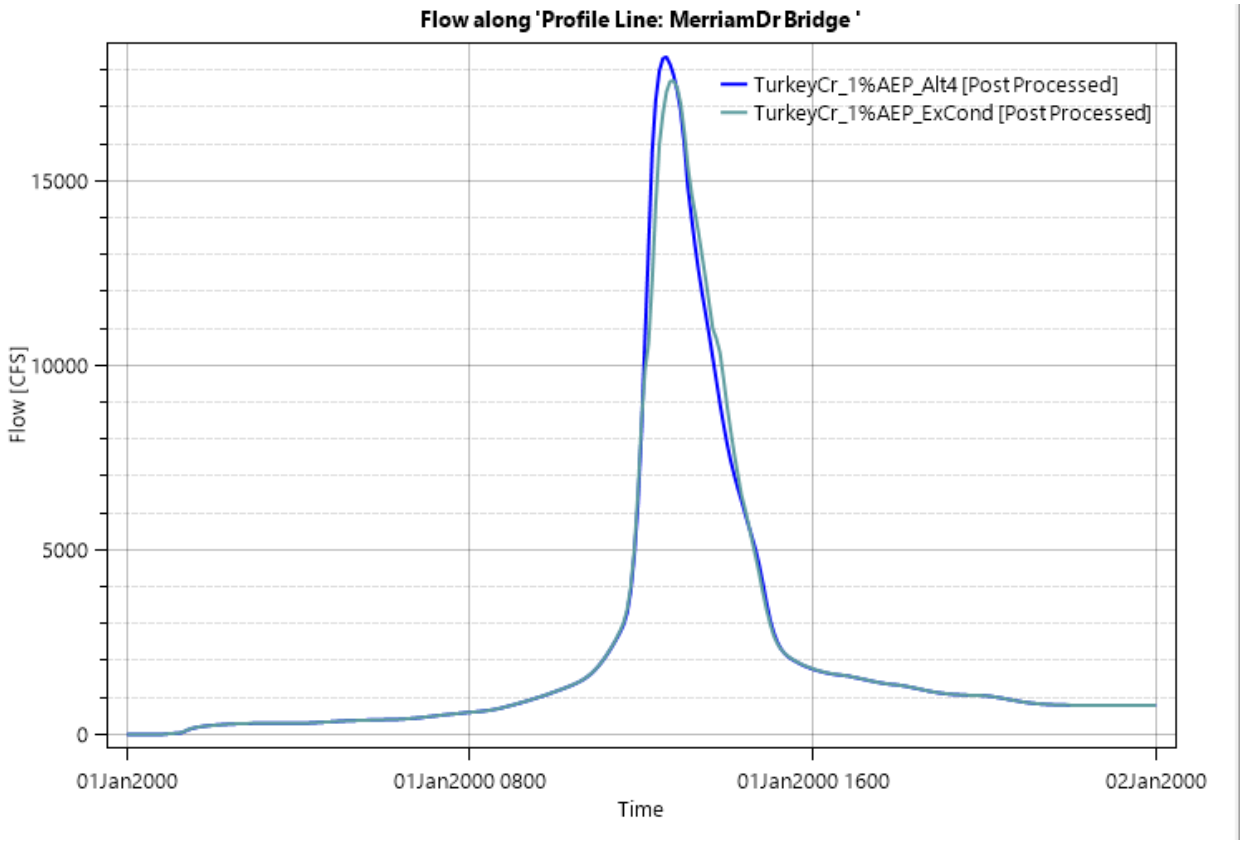


Figure 9. Flow hydrograph for Merriam Bridge (Channel STA 63+50) for FWOP and 50% Design

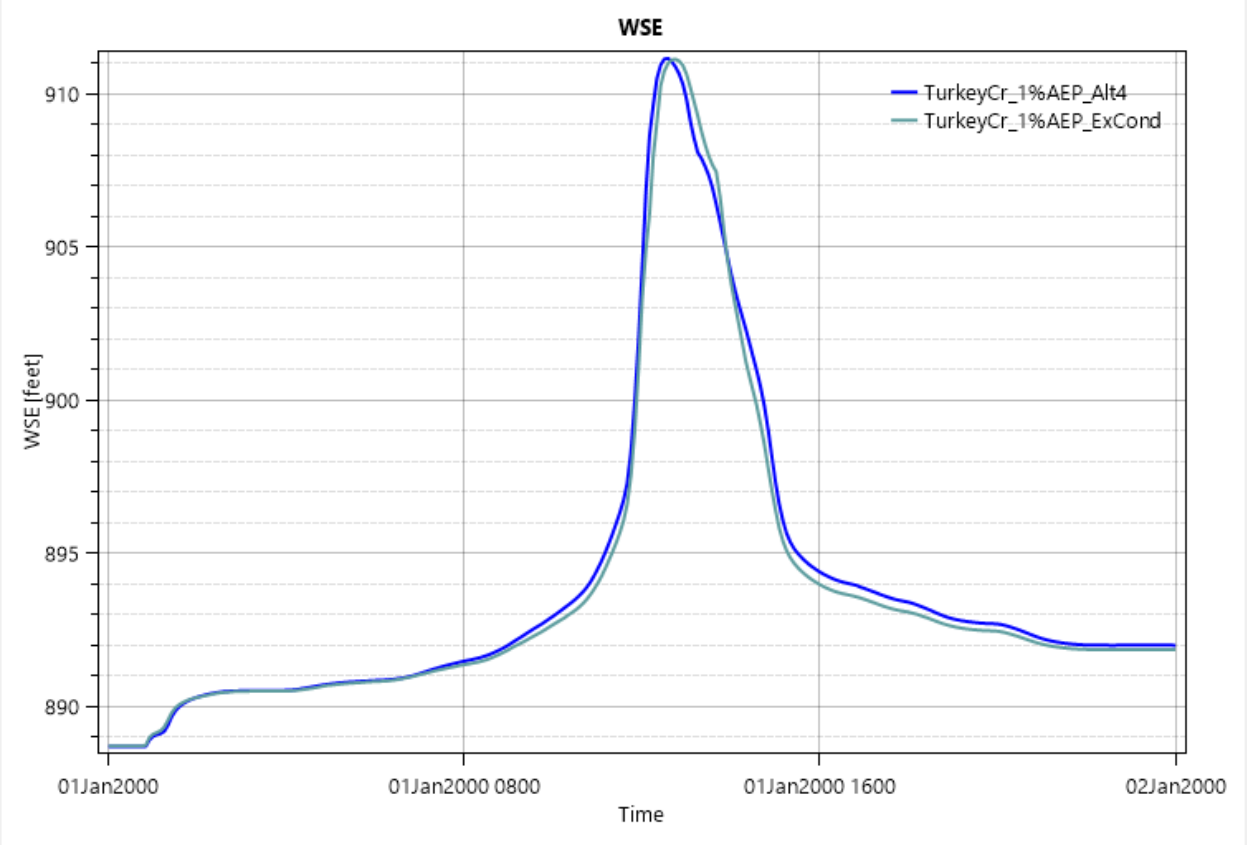


Figure 10. WSE hydrograph for Merriam Bridge (Channel STA 63+50) for FWOP and 50% Design

APPENDIX E.7 – STANTEC HYDRAULIC MODEL REPORT

Turkey Creek and Brush Creek 2D Watershed Hydrologic and Hydraulics Report

July 2024

Prepared for:
Johnson County, KS
Contract No. 2022-060

Prepared by:
Stantec
6800 College Blvd, Suite 750
Overland Park, KS 66211

Water Resources Solutions
5000 W 95th Street, Suite 290
Prairie Village, KS 66207

DRAFT

Table of Contents

1 INTRODUCTION 1

2 2D BLE HYDROLOGIC AND HYDRAULIC ANALYSIS 2

2.1 Hydrologic Inputs 2
2.1.1 Precipitation Data 2
2.1.2 Infiltration 5
2.1.3 Domain Development 6
2.1.4 Terrain Data 7
2.1.5 Survey 8
2.1.6 Manning's n Roughness 12
2.1.7 Mesh Refinement 14
2.1.8 Levee Analysis 16
2.1.9 Boundary Conditions 20
2.1.10 Computation Settings 20

3 CALIBRATION 21

3.1 StormWatch Gages 21
3.2 USGS Instantaneous Gage Data 23
3.3 Calibration Parameters 24
3.3.1 August 2017 Event 25
3.3.2 May 2020 Event 27
3.4 Observed Event Calibration Data 30
3.4.1 Parameter Sensitivity 30
3.4.2 Final Model Parameters 33
3.4.3 Model Limitations 37
3.5 July 2017 Validation 38

4 SYNTHETIC EVENT SIMULATION 40

4.1 USGS Gage Frequency Analysis & Rating Curve 40
4.2 Flow Comparison with Regression Estimates & Effective Data 42
4.3 Summary of Discharges 47
4.4 Stage Comparison 50

5 FLOODPLAIN MAPPING 50

6 CONCLUSION 50

7 REFERENCES 52

APPENDIX A – SENSITIVITY TESTING 53

APPENDIX B – BASE & CALIBRATED MODEL RESULTS 2017 EVENT 64

APPENDIX C – BASE & CALIBRATED MODEL RESULTS 2020 EVENT 71

APPENDIX D – JULY 2017 VALIDATION 77

APPENDIX E - HEC-SSP GAGE ANALYSIS 84

APPENDIX F – STAGE COMPARISON 88

LIST OF TABLES

Table 1: Study Watershed Drainage Area 1
Table 2: Effective Floodplain Mapping for Impacted Counties 2
Table 3: 12-hour Duration Rainfall Depths for Johnson County, Kansas 3
Table 4: Base Curve Numbers for Turkey Creek and Brush Creek Study Area 6
Table 5: Base Manning's n Roughness Coefficients 13
Table 6: Advanced Time Step Controls 20
Table 7: Computation Settings 20
Table 8: Turkey and Brush Creek Stage Collection StormWatch Gages and Zero Datums 22
Table 9: Uncalibrated Model Results Compared to StormWatch Data for August 2017 Event 27
Table 10: Uncalibrated Model Results Compared to StormWatch Data for May 2020 Event 30
Table 11: Parameter Sensitivity Run Summary 32
Table 12: Calibrated Manning's n Values based on Historic Events 34
Table 13: Calibrated Model Results Compared to StormWatch Data for August 2017 Event 35
Table 14: Calibrated Model Results Compared to StormWatch Data for May 2020 Event 35
Table 15: USGS Gage Analysis for 2D Watershed Calibration 41
Table 16: 1% Annual Channce Flow Comparison for Turkey Creek Watershed 43
Table 17: 1% Annual Channce Flow Comparison for Brush Creek Watershed 44
Table 18: Final Summary of Discharges for Turkey Creek Watershed 47
Table 19: Final Summary of Discharges for Brush Creek Watershed 48

LIST OF FIGURES

Figure 1: Turkey Creek and Brush Creek Study Area 1
Figure 2: 12-hour Storm, 3-hour Peak Nested Distribution 4
Figure 3: Rainfall Hyetograph for 12-hour Storm with 3-hr Peak 5
Figure 4: Turkey Creek and Brush Creek HEC-RAS Domains 7
Figure 5: DASC and MSDIS Terrain Supplemented with AIMS for the Turkey Creek Watershed 8
Figure 6: Turkey and Brush Creek Survey Locations 9
Figure 7: Turkey Creek Survey XS versus LiDAR near 635 10
Figure 8: Turkey Creek Survey XS versus LiDAR near Johnson Drive 10
Figure 9: Turkey Creek Survey XS versus LiDAR near 67th Street 11
Figure 10: Terrain Modification Example at Culvert 11
Figure 11: Terrain Modification Example of Upstream Hydraulic Connection 12
Figure 12: Manning's N Layer based on AIMS Data and Aerial Imagery 14
Figure 13: Example of Cell Size Refinement along Stream 15
Figure 14: Example of Cell Refinement along Buildings near Streams 16
Figure 15: Rock Creek Stream Restoration Floodwall Levee as shown in the NLD 17
Figure 16: Rock Creek Stream Restoration Floodwall Levee as shown in the National Flood Hazard Layer 18
Figure 17: Floodwall along Brush Creek at Roe Ave and W 75th St (Source: Google Earth) 19
Figure 18: StormWatch Gage and Zero Datum Relationship (Source: Overland Park) 22
Figure 19: Location of Calibration Gages within the Turkey and Brush Creek Study Area 23
Figure 20: USGS Gage 06893557 Instantaneous Stage and Discharge Data for 2017 Calibration Event 24
Figure 21: USGS Gage 06893557 Instantaneous Stage and Discharge Data for 2020 Calibration Event 24
Figure 22: StormWatch Precipitation Totals for August 2017 Event 25
Figure 23: August 2017 Rainfall across Study Area based on StormWatch Gages 26
Figure 24: StormWatch Precipitation Gage Rainfall Hyetographs for May 2020 Event 28
Figure 25: May 2020 Rainfall across Study Area based on StormWatch Gages 29
Figure 26: Uplands Area Refinement in Turkey and Brush Creek 2D Models 31
Figure 27: Abstraction Ratio Sensitivity in Brush Creek Model 33
Figure 28: Differences in Turkey Creek Modeled versus Observed Stages from Base and Calibrated Events 36

Figure 29: Differences in Brush Creek Modeled versus Observed Stages from Base and Calibrated Events (BR04 no observed data for 2020)..... 37
Figure 30: StormWatch Precipitation Totals for July 2017 Event (*gage did not collect data) 38
Figure 31: Differences in Turkey Creek Modeled versus Observed Stages with July 2017 Validation Event 39
Figure 32: Differences in Brush Creek Modeled versus Observed Stages with July 2017 Validation Event (BR02 no data for July 2017, BR04 no data for May 2020, BR09 no data for August 2017 or May 2020) 40
Figure 33: Location of USGS Gages within the Turkey and Brush Creek Watersheds 42
Figure 34: Flow Comparisons for the Turkey Creek Watershed..... 44
Figure 35: Flow Comparisons for the Brush Creek Watershed 46

1 Introduction

This report summarizes the hydrologic and hydraulic analysis for the 2-dimensional (2D) base level engineering (BLE) of Turkey Creek and Brush Creek in Johnson County, Kansas. This project includes new hydrologic and hydraulic analysis for both watersheds within the study area. The study area is shown below in Figure 1.

Table 1: Study Watershed Drainage Area

Watershed	Drainage Area (square miles)
Turkey Creek	17.1
Brush Creek	12.3

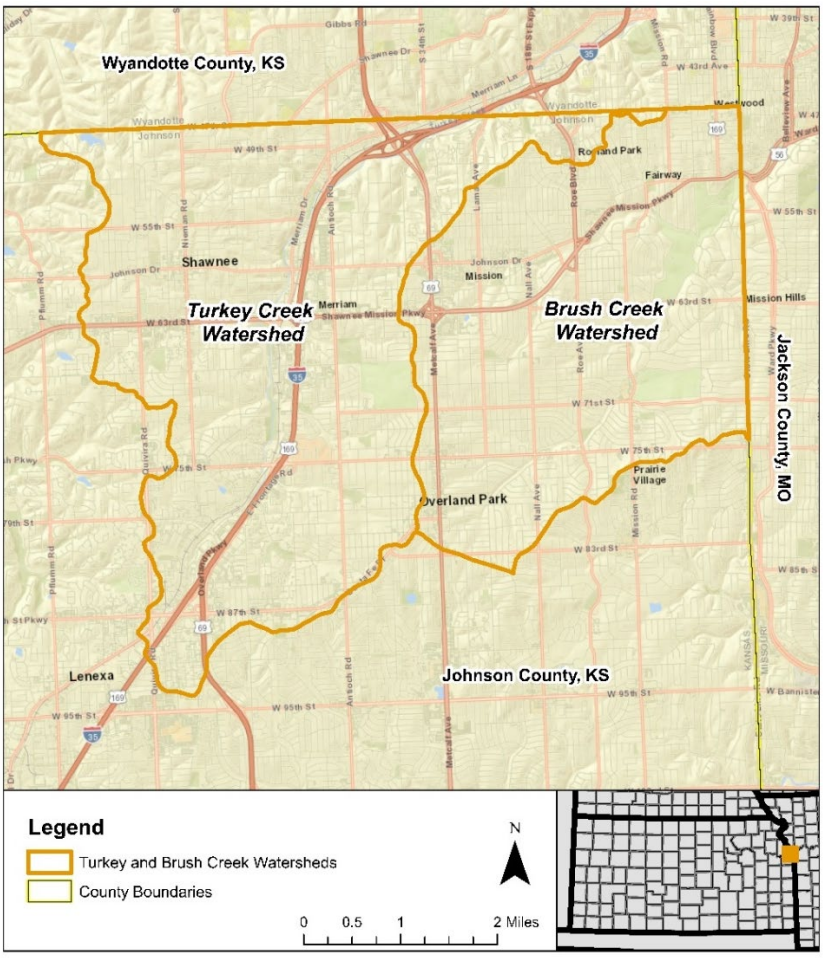


Figure 1: Turkey Creek and Brush Creek Study Area

This study includes analysis of the 50-, 10-, 4-, 2-, 1-, 1plus-, and 0.2-percent-annual-chance events. Flood hazards were identified for streams with drainage areas equal to or greater than 160 acres (0.25-square mile) or where effective floodplain mapping is present. These scoped streams match FEMA's enhanced study criteria and represent the extents where the floodplain mapping will be cutoff.

The effective hydrologic and hydraulic analysis for Turkey and Brush Creek watersheds were completed in May 2005. The hydrologic analysis was completed using HEC-1 and the hydraulic analysis was developed in HEC-RAS version 3.1.1. Floodplains were delineated based on interpolation using digital topographic maps with a contour interval of 1 foot (Reference 1). The effective floodplain mapping for Johnson County is summarized in Table 1:

Table 2: Effective Floodplain Mapping for Impacted Counties

County	Effective FEMA Floodplains
Johnson	08/03/2009

2 2D BLE Hydrologic and Hydraulic Analysis

2D models were developed for the Turkey Creek and Brush Creek watersheds to study the surface water bodies (within Johnson County) using Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 6.5 (Reference 2). Both hydrologic and hydraulic calculations were developed directly in the HEC-RAS models using a rain-on-grid, 2D approach.

2.1 Hydrologic Inputs

2.1.1 PRECIPITATION DATA

As part of the Indian Creek watershed study, Johnson County funded Dr. Bruce McEnroe, a technical expert on the hydrology within Johnson County, to evaluate the appropriate precipitation and temporal distribution for the watershed studies within the County. Dr. McEnroe's study also evaluated the appropriate rainfall distribution for the flood frequency watershed analysis. Typically, a 24-hour nested hyetograph with a 12-hour peak is applied for flood frequency watershed study; however, based on Dr. McEnroe's evaluation, a 12-hour nested hyetograph with a 3-hour peak was determined as an appropriate temporal distribution for Johnson County. A copy of the final report is included as an addendum.

NOAA Atlas 14 rainfall depths vary across the County. The 1-percent annual chance 12-hour rainfall ranges from approximately 7.4 inches to 7.8 inches from west to east across the County. Johnson County concluded that a single rainfall depth derived from NOAA Atlas 14 be applied to all watershed studies in the County. Those depths were derived from KDOT's rainfall tables which are averaged by county and are summarized in Table 3.

The 1%-plus storm event is not directly provided by Atlas 14 but was calculated using the Atlas 14 provided 90% confidence interval around the baseline 1% event depth. The plus event represents the 1 standard error depth associated with the 1-percent event, and assuming a normal distribution, can be calculated as follows:

$$\sigma = \frac{X_{90} - X_{10}}{Z_{90} - Z_{10}} \quad (1)$$

$$CI_{1pctplus} = (X - \sigma, X + \sigma) \quad (2)$$

Where σ , represents the standard deviation of an assumed normal distribution around the NOAA Atlas 14 frequency estimates; X_{90} and X_{10} represent the upper and lower estimates of the 90% confidence interval, respectively; and Z_{90} and Z_{10} represent the corresponding z-scores of the upper and lower estimates. The 1-percent-plus confidence interval is approximated by applying one standard deviation above the baseline 1-percent annual chance event. The average 90% and average 10% confidence interval depths for the County were used to estimate the 1% plus rainfall depth.

Table 3: 12-hour Duration Rainfall Depths for Johnson County, Kansas

	Annual Chance Exceedance Event						
	50%	10%	4%	2%	1%	1%+	0.2%
12-hr Rainfall Depth (in)	3.17	4.74	5.82	6.71	7.65	8.56	10.04

The nested distributions and corresponding hyetographs were developed based on the County-wide average rainfall depths and are shown in Figure 2 and Figure 3.

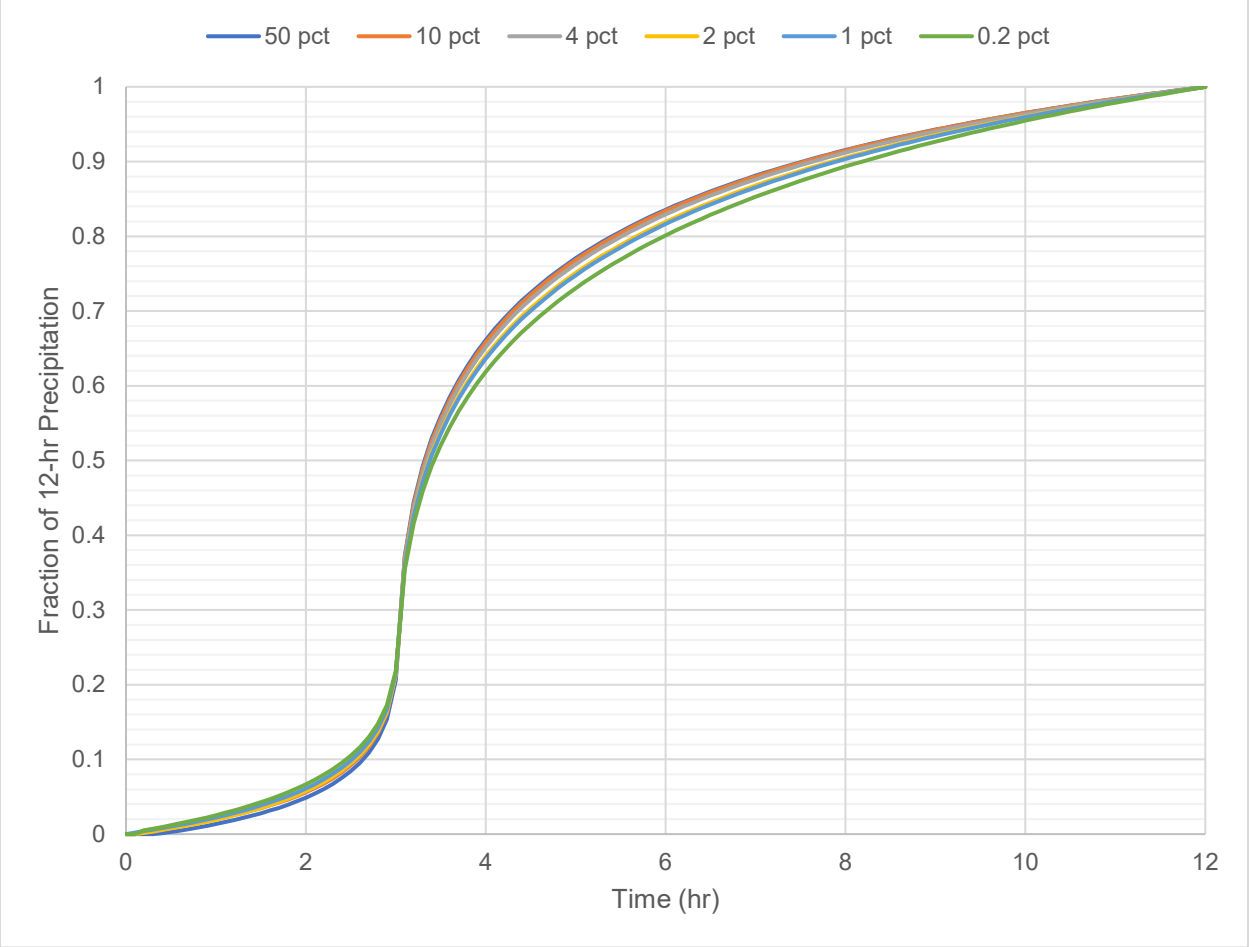


Figure 2: 12-hour Storm, 3-hour Peak Nested Distribution

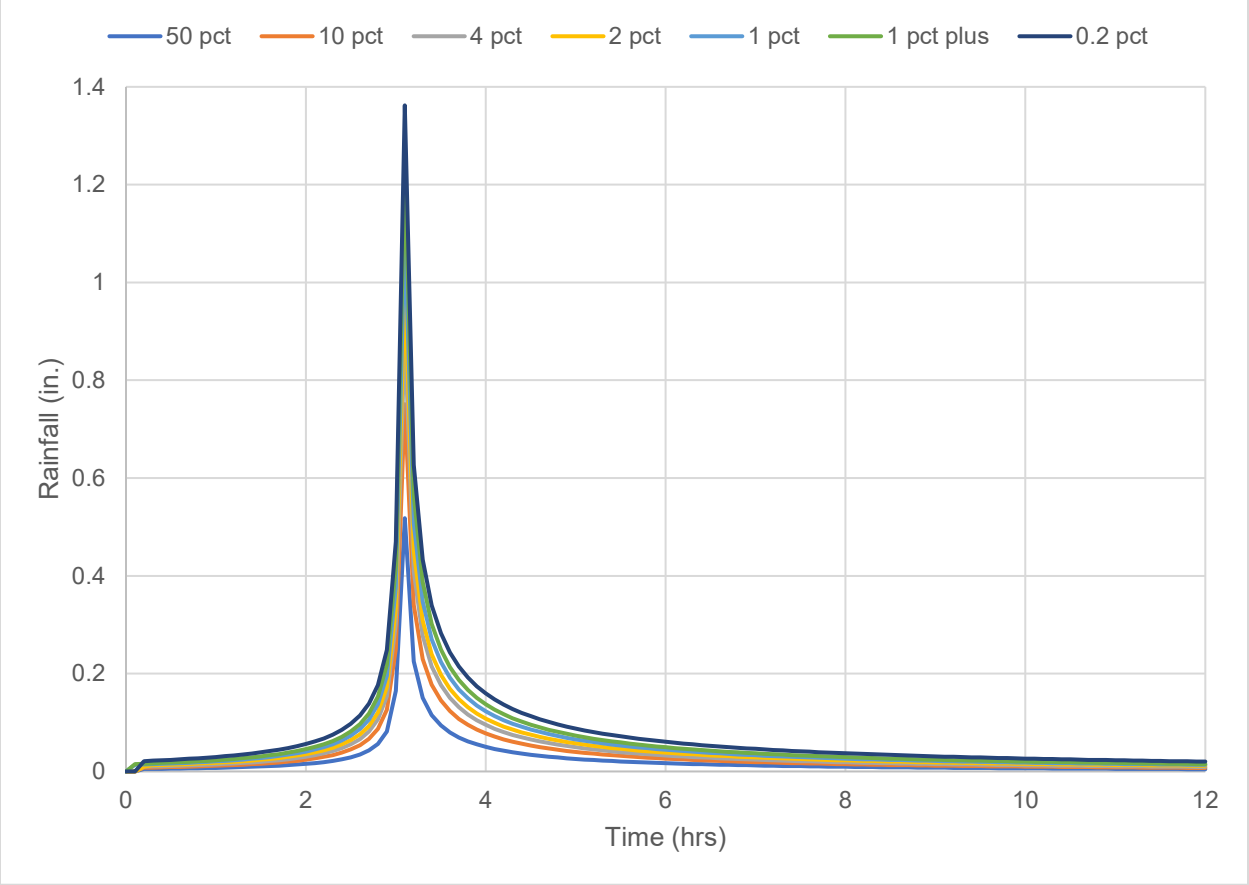


Figure 3: Rainfall Hyetograph for 12-hour Storm with 3-hr Peak

2.1.2 INFILTRATION

Hydrologic calculations were applied directly in HEC-RAS using the Soil Conservation Service (SCS) Curve Number (CN) method. Base CN data was developed using 2021 aerial imagery from Johnson County, Automated Information Mapping System (AIMS) datasets that include roadway, trees, channels (natural and concrete) and SSURGO Soil Data. One soil group within the study area did not have a hydrologic soil group defined (Urban land-Harvester complex, 2 to 9 percent slopes). In upland areas where adjacent soils were C, a soil type C was applied. For the areas with no defined soil group along Highway 69 and Interstate 635, a soil group type D was applied. One dual soil group was present in the Turkey Creek watershed (C/D); the lower of the two soil groups, D, was applied for the CN application. The dominant soil groups in the Turkey and Brush Creek watersheds are groups C and D. The base curve numbers were derived from the NRCS TR-55 document (Reference 6) and are consistent with the base curve numbers applied in the Indian Creek watershed study that was previously modeled in Johnson County (Reference 5).

Table 4: Base Curve Numbers for Turkey Creek and Brush Creek Study Area

Description	A	B	C	D
Open Water	100	100	100	100
Barren Land	76	85	90	93
Buildings	98	98	98	98
Grass	39	61	74	80
Pavement	98	98	98	98
Trees	36	60	73	79
Concrete Channel	98	98	98	98
Natural Channel	39	61	74	80

2.1.3 DOMAIN DEVELOPMENT

The northeast (NE) corner of Johnson County includes Turkey Creek and Brush Creek for the 2D modeling. The domains were based roughly on the National Hydrography Dataset (NHD) HUC12 watershed boundaries and were developed with 500-foot buffer to ensure all the contributing drainage area is captured in the flow routing based on the terrain data for the County. Each watershed domain was also extended approximate 1-mile downstream of the County boundary to ensure the downstream boundary condition does not directly impact the water surface elevations along the study streams.

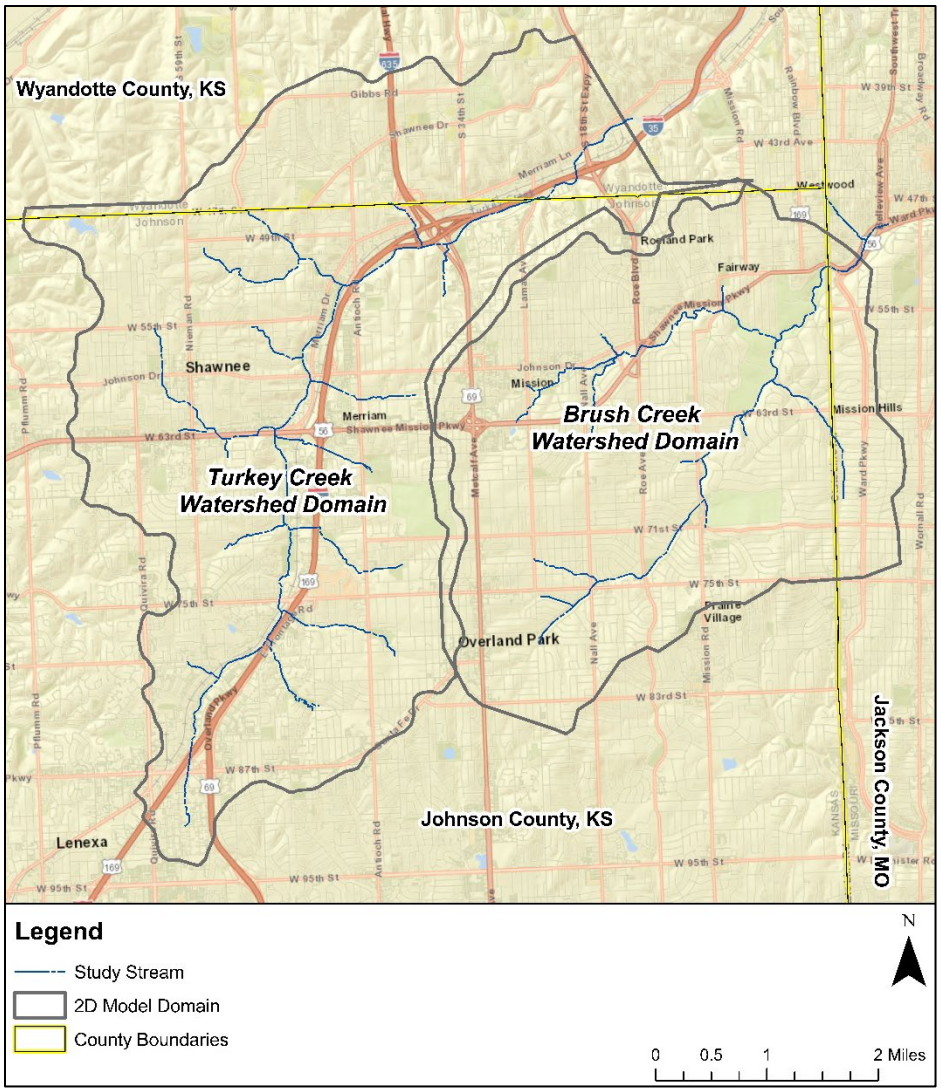


Figure 4: Turkey Creek and Brush Creek HEC-RAS Domains

2.1.4 TERRAIN DATA

Terrain data was provided by Johnson County through the Automated Information Mapping System (AIMS) for Turkey Creek and Brush Creek (2-ft resolution). The LiDAR was flown in December of 2020. For the portion of Turkey Creek that is outside of Johnson County, located in Wyandotte County, supplemental LiDAR was downloaded from Kansas' Data Access and Support Center (DASC) at 1-meter resolution. The LiDAR acquired from DASC was collected in 2018. The DASC LiDAR was reprocessed to match the resolution of the Johnson County LiDAR and were mosaicked together. Figure 5 below identifies the areal portion of the model that relies on DASC terrain data. A small portion of the Brush Creek watershed, within Jackson County, Missouri was obtained from the Missouri Spatial Data Information Service (MSDIS) to ensure the terrain covered the entire drainage area of Brush Creek. The MSDIS LiDAR has a 1-meter resolution and was acquired in 2006.

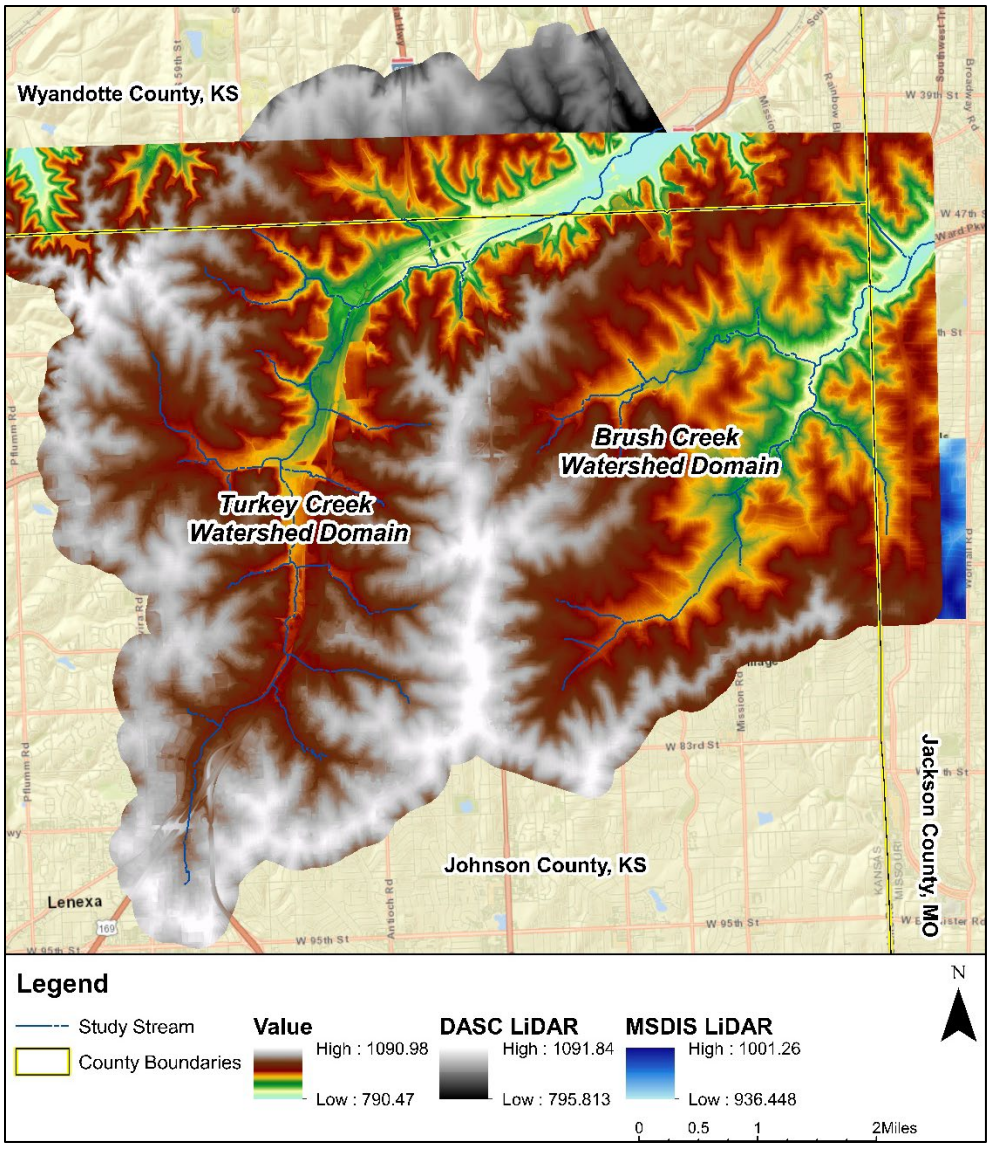


Figure 5: DASC and MSDIS Terrain Supplemented with AIMS for the Turkey Creek Watershed

2.1.5 SURVEY

Survey of bridges, culverts, and channel cross-sections were taken in the field by Renaissance Infrastructure Consulting (RIC) between January and March 2023. All structures along the study streams were surveyed and incorporated into the models as 2D connections. Survey locations are shown in Figure 6.

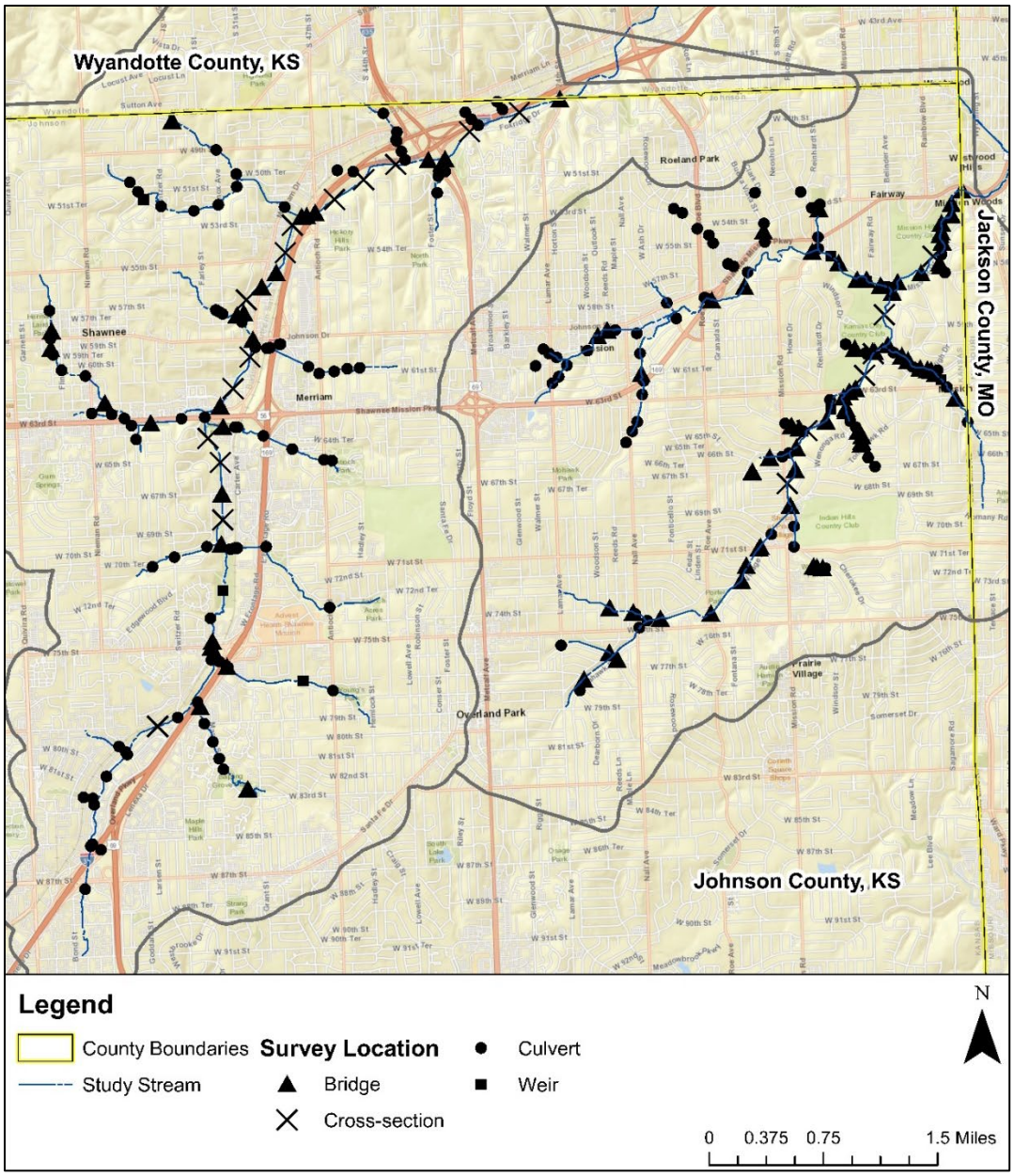


Figure 6: Turkey and Brush Creek Survey Locations

Cross-sectional channel survey was collected along Turkey and Brush Creeks. This channel bathymetry was burned into the terrain where differences in survey and LiDAR indicated that the LiDAR was not capturing the true channel bottom. The channel survey was interpolated and merged with the LiDAR to account for the additional conveyance not captured in the LiDAR. For Turkey Creek, surveyed cross-section locations showed little to no difference between the surveyed elevations and the LiDAR suggesting that the LiDAR was flow during a low-flow condition in the channel and adequately captures the channel conveyance. The survey was incorporated into the final 2D model terrain, interpolating

between survey locations and retaining only survey locations where the hydro-flatted LiDAR was higher than the survey.

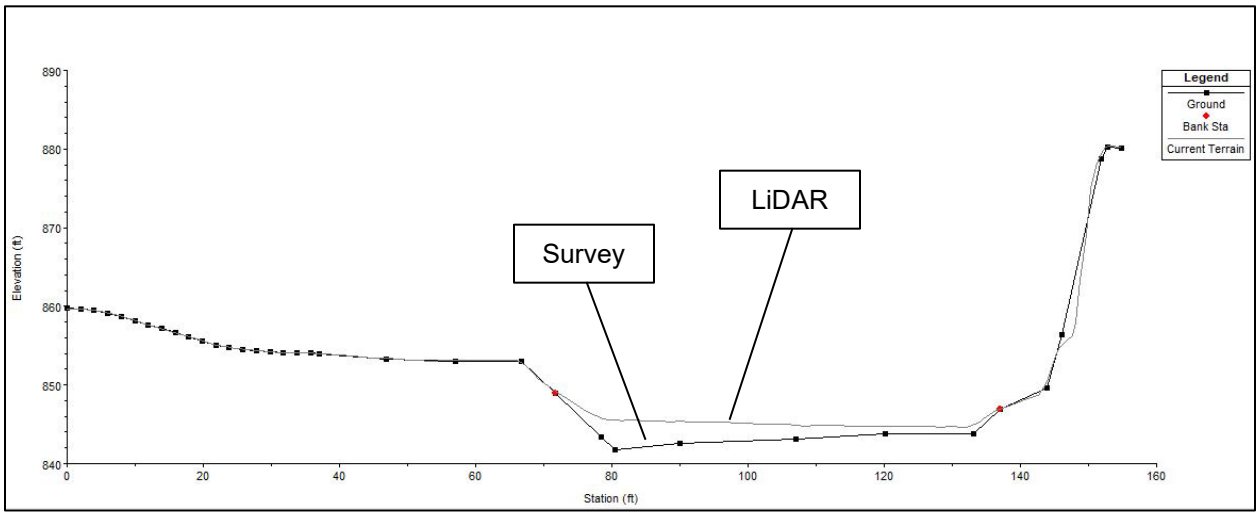


Figure 7: Turkey Creek Survey XS versus LiDAR near 635

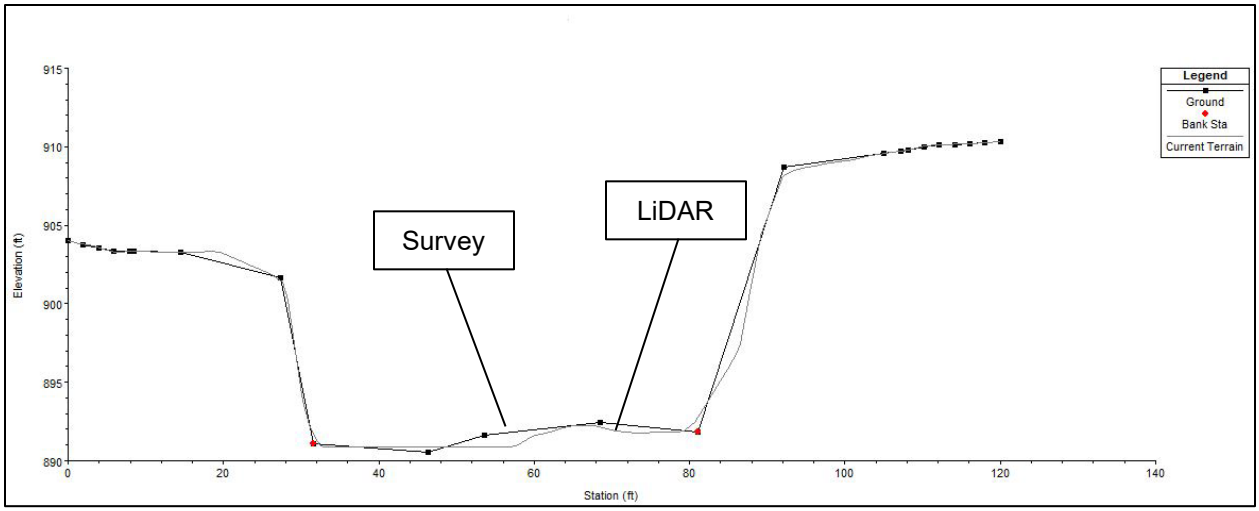


Figure 8: Turkey Creek Survey XS versus LiDAR near Johnson Drive

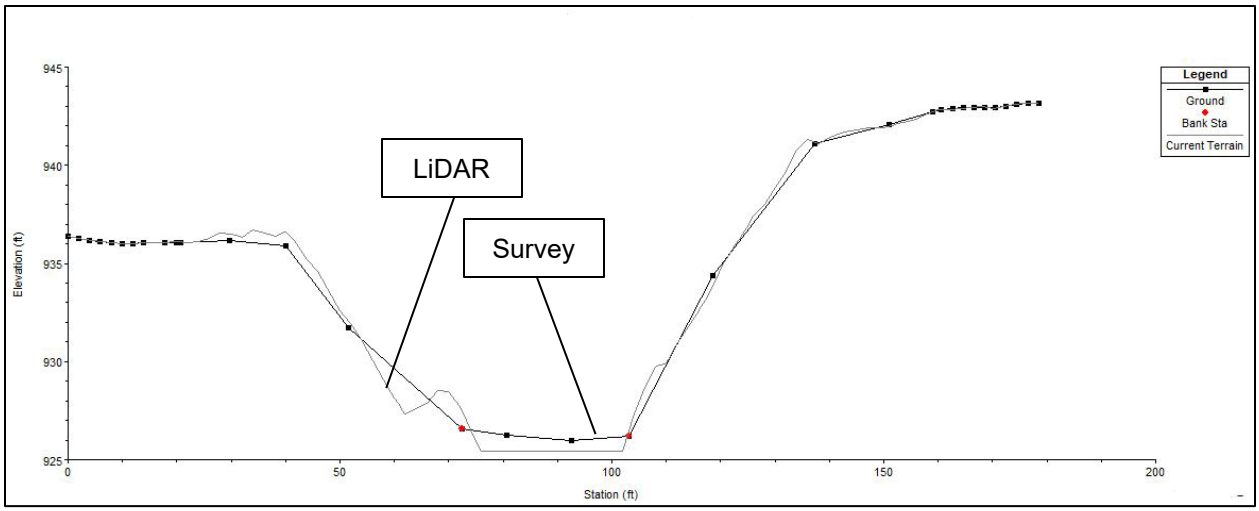


Figure 9: Turkey Creek Survey XS versus LiDAR near 67th Street

At some culvert locations, survey indicated that the culvert inverts were lower than the LiDAR elevations. In these cases, terrain modifications were applied at the upstream and/or downstream of the culverts to ensure the survey inverts were applied in the model as shown in Figure 10.

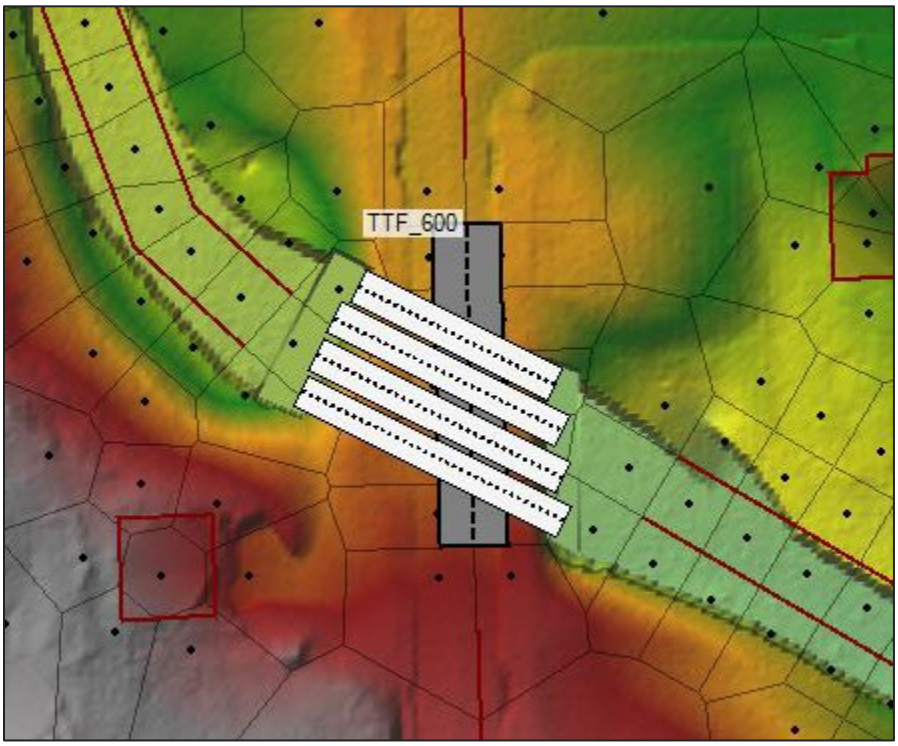


Figure 10: Terrain Modification Example at Culvert

A handful of structures in Brush Creek were irregular culvert shapes. These shapes were not readily available in HEC-RAS, and blocked obstructions were applied to better represent the culvert geometry

within the model. This impacted the following structures in the Brush Creek watershed: BRC_1300, BTU2_200, ROC_1400, RTB_200, RTB_400.

Railings were included on bridges and culverts based on engineering judgement, and they were included if the opening size would likely obstruct the flow should the water reach the railings.

During the model development, if non-surveyed, upstream impoundments or hydraulic connections were discovered based on aerial imagery and Johnson County's stormwater inventory, terrain modifications were applied to ensure flow could pass through the embankment and not cause artificial ponding as shown in Figure 11.

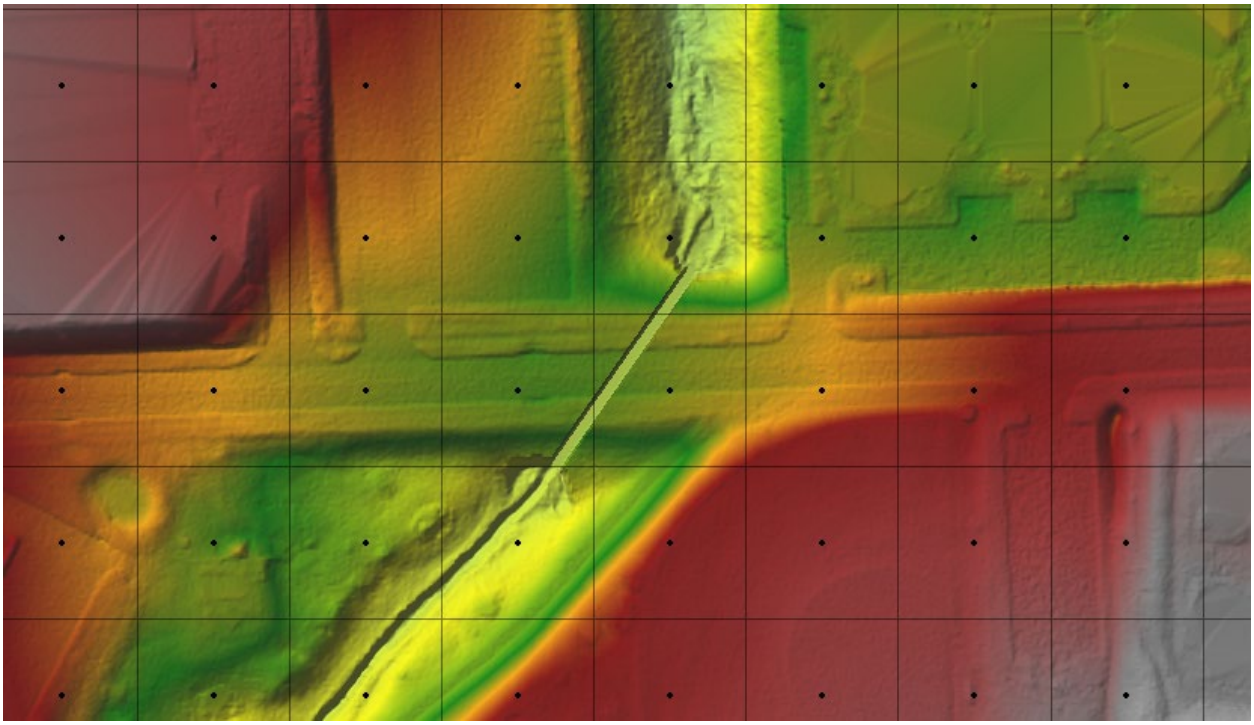


Figure 11: Terrain Modification Example of Upstream Hydraulic Connection

2.1.6 MANNING'S N ROUGHNESS

Manning's n roughness coefficient data was spatially defined for the study area and developed using the most recent aerial imagery provided by Johnson County (AIMS) with additional channel definition. Base roughness values were based on the HEC-RAS 2D User's Manual (Reference 7) and are consistent with the base values applied in the recent 2D analysis of the Indian Creek watershed, also located in Johnson County. These base values serve as the initial condition prior to model calibration. Additional discussion regarding the calibration modifications to the Manning's n roughness coefficients can be found in Section 3.4. Base Manning's n values are shown in Table 5.

The 2D watershed domains applied the spatially varied Manning's n on Faces option in HEC-RAS.

Table 5: Base Manning's n Roughness Coefficients

Description	Base n Value
Trees	0.085
Natural Channel	0.035
Concrete Channel	0.015
Pavement	0.02
Grass	0.03
Buildings	1
Barren Land	0.025
Open Water	0.025

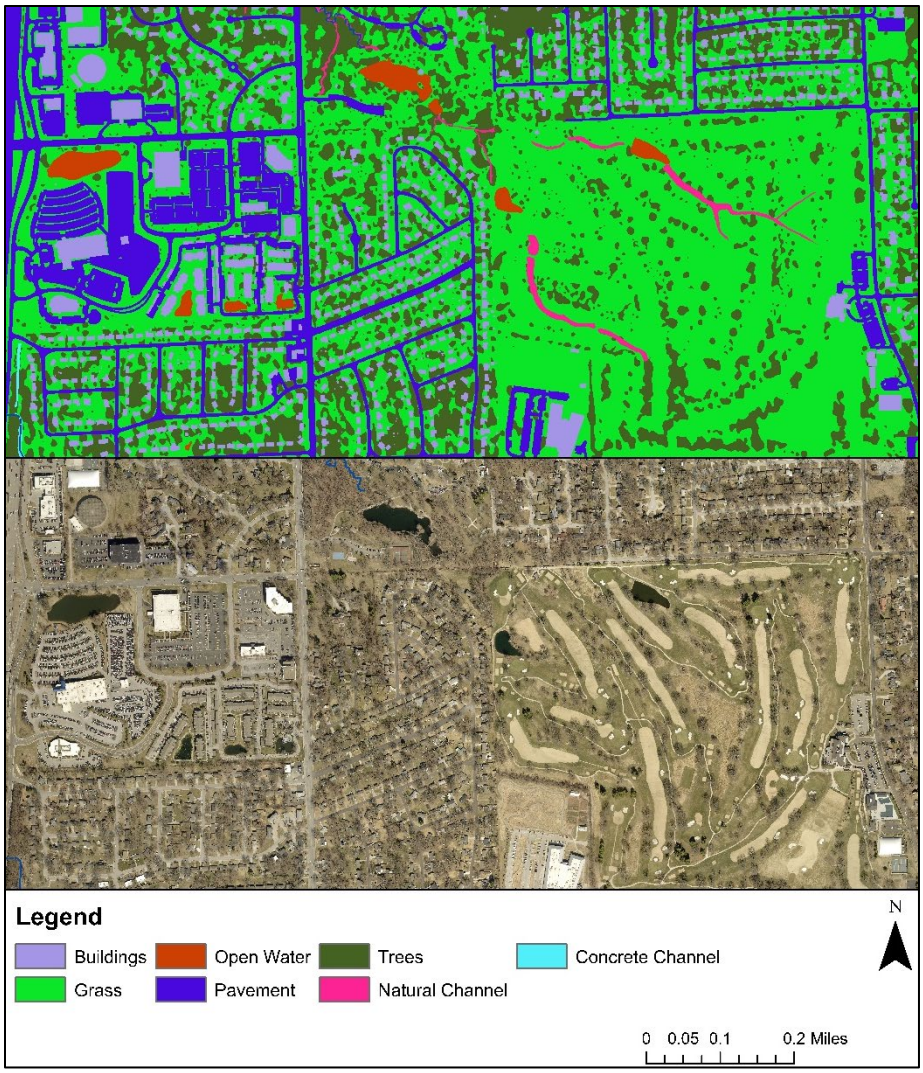


Figure 12: Manning's N Layer based on AIMS Data and Aerial Imagery

2.1.7 MESH REFINEMENT

Based on the cell sensitivity testing completed for the 2D study of the Indian Creek watershed in Johnson County, a base cell size of 100 feet by 100 feet was applied for both the Turkey and Brush Creek watershed models. Additional refinement was applied along stream centerlines and along significant topographic features such as roads and in more populated areas. The cell spacing applied for hydraulic structures (e.g., bridges and culverts), modeled as 2D Area Connections, was 25 x 25 foot and cell faces were aligned perpendicular to direction of flow. The cell spacing applied for channel refinement was 25 x 25 foot. Cell faces were aligned along the streams ensuring cell faces were perpendicular to the direction of flow.

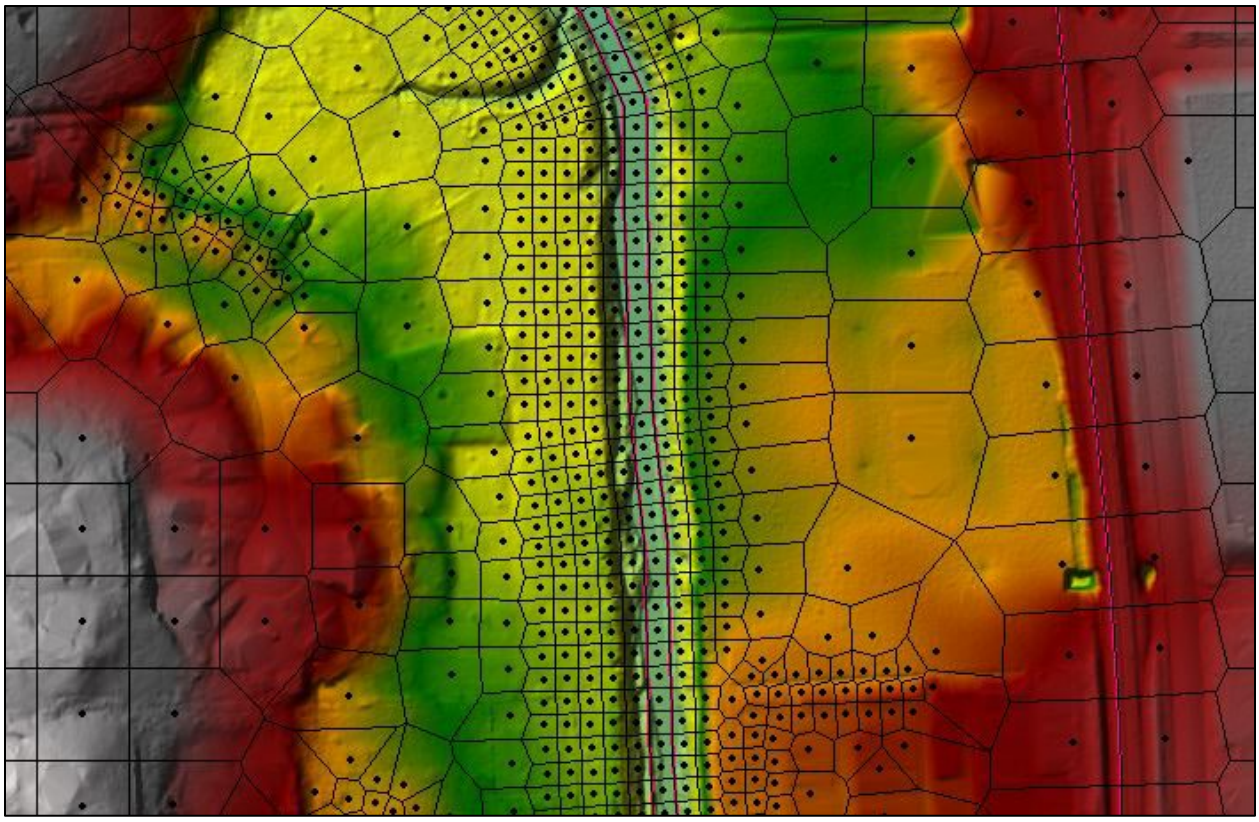


Figure 13: Example of Cell Size Refinement along Stream

Additional refinement was added to structures near the study streams. The AIMS building footprint dataset was leveraged to develop refinement regions within the 2D mesh. A 25 x 25 foot cell spacing was applied to the buildings near the study streams in order to orient cell faces along the building footprints. This mesh alignment applies the higher Manning's n roughness coefficient of the structure to the cell faces so the model recognizes conveyance is minimal at these locations. This application was not applied to all structures in the watershed but those along the floodplain with a larger impact on the conveyance of flood waters.

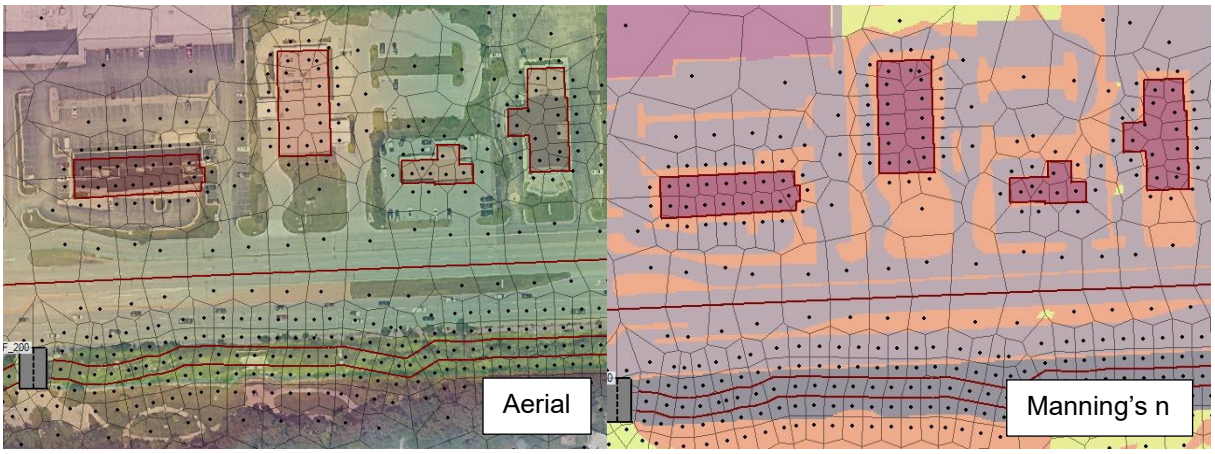


Figure 14: Example of Cell Refinement along Buildings near Streams

2.1.8 LEVEE ANALYSIS

2.1.8.1 Rock Creek Stream Restoration Floodwall

There is one levee located in the National Levee Database (NLD) within the Brush Creek Watershed. NLD System ID 280005000771 (Rock Creek Stream Restoration Floodwall) is a locally constructed, operated, and maintained levee system along Rock Creek in the City of Mission. The area behind the levee system is shown with reduced flood risk due to levee on the effective flood insurance rate map (20091C0024G) from a Letter of Map Revision (LOMR) dated February 8, 2012 (11-07-1190P).

Following FEMA levee guidance, the flood risk along the levee system was analyzed under both 'with levee' and Natural Valley conditions for the synthetic flood frequency events (Reference 10). The 'with levee' geometry includes the floodwall as a 2D connection with the floodwall elevations attributed based on LOMR model floodwall elevations. For the Natural Valley geometry, no 2D connection was included and the terrain elevations were used.

The 'with levee' scenario will dictate the base flood elevations within the channel while the Natural Valley scenario will identify the levee impacted area. As the Turkey and Brush Creek project moves to data development, further levee discussion will occur to determine the mapping path forward for the levee.

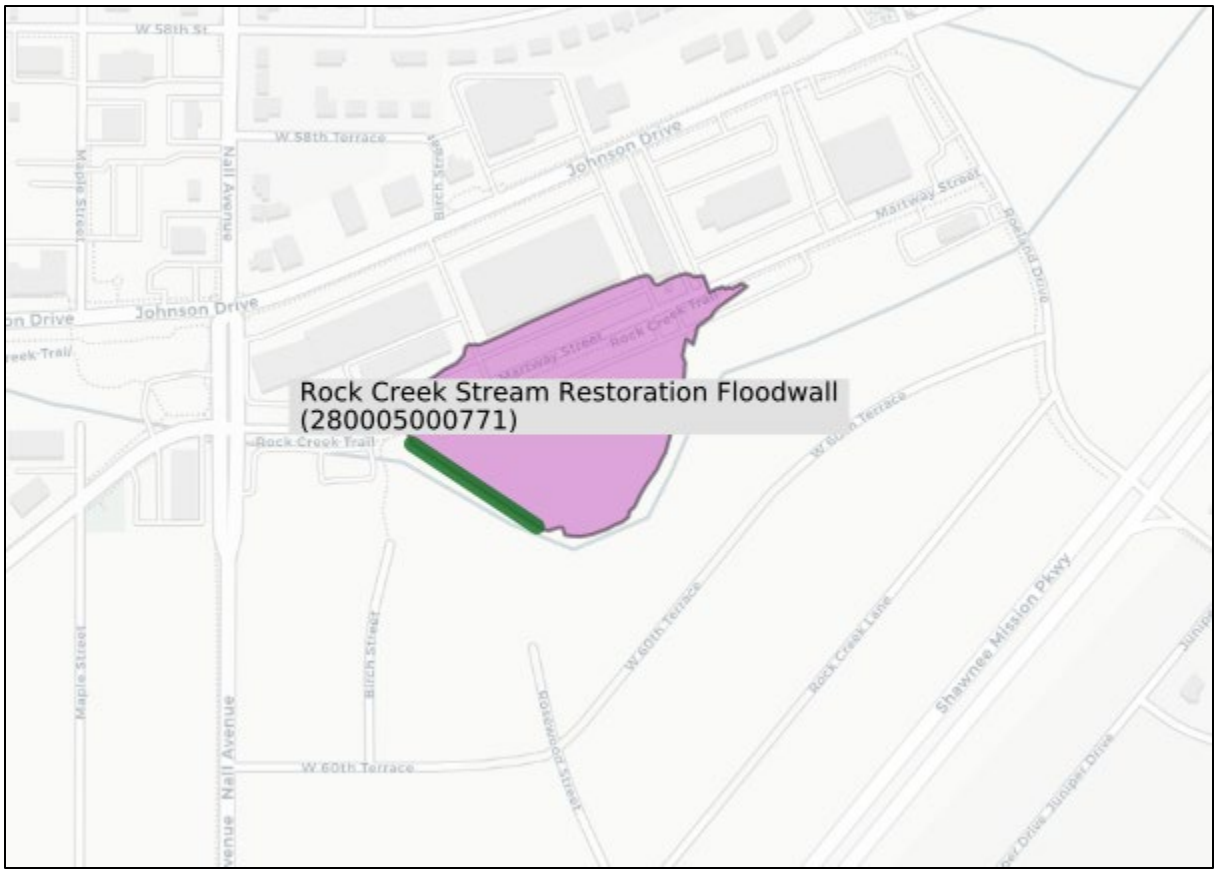


Figure 15: Rock Creek Stream Restoration Floodwall Levee as shown in the NLD

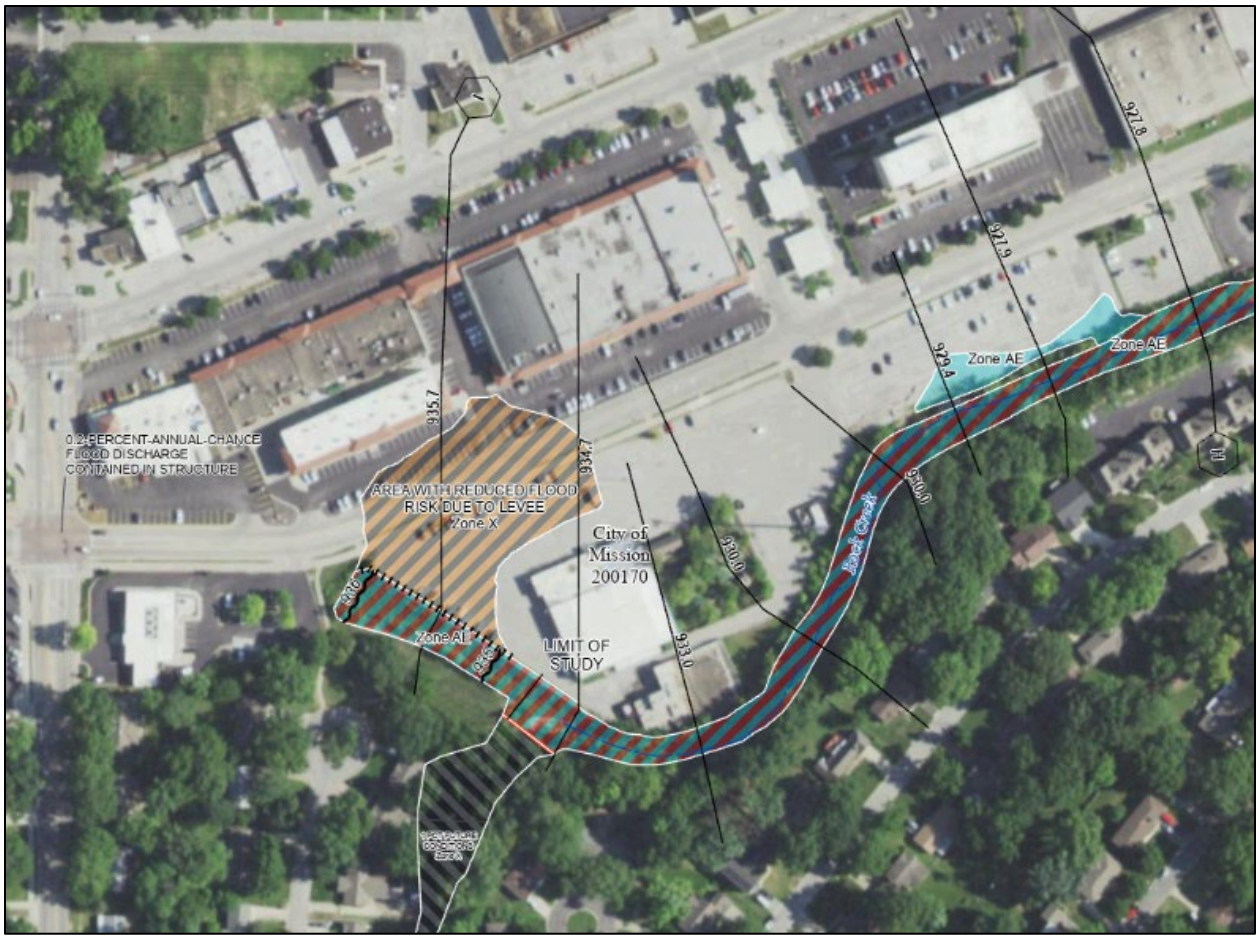


Figure 16: Rock Creek Stream Restoration Floodwall Levee as shown in the National Flood Hazard Layer

2.1.8.2 Brush Creek Floodwall at Roe Avenue

During internal review of the Brush Creek model, it was discovered that a floodwall exists along Brush Creek for approximately 700 feet until Roe Avenue. This floodwall is not shown on the effective FIRM and is not recorded in the NLD. However, it was decided that the floodwall would be included in the levee analysis as it was constructed to protect adjacent properties from flood waters. The same geometry methodology as was used on the Rock Creek flood wall was used for the Brush Creek Floodwall at Roe Avenue. The floodwall was constructed as part of S.M.A.C. Project No. BC-11-054 and design plans from the S.M.A.C. project were used to obtain floodwall elevations and alignments.

The 'with levee' scenario will dictate the base flood elevations within the channel while the Natural Valley scenario will identify the levee impacted area. As the Turkey and Brush Creek project moves to data development, further levee discussion will occur to determine the mapping path forward for the floodwall.



Figure 17: Floodwall along Brush Creek at Roe Ave and W 75th St (Source: Google Earth)

2.1.9 BOUNDARY CONDITIONS

Normal depth boundary conditions were applied to the downstream extent of all modeled terrain. The slope was estimated based on the approximate channel slope at the downstream. For the model domains, each downstream boundary was extended approximately 1-mile downstream of the County boundary to ensure the boundary condition did not have an immediate impact on the water surface elevations at the study extent.

To ensure artificial ponding along the domain boundary was not impacting the hydraulic routing due to the 500-foot HUC12 buffer, an additional normal depth boundary condition was applied to the upstream perimeter of the model to allow non-contributing drainage to leave the domain.

2.1.10 COMPUTATION SETTINGS

The 2D models were run using the Shallow Water Equations, Eulerian-Lagrangian Method. The computational time setting was adjusted based on Courant numbers with a base timestep of 30 seconds. The simulated time step was controlled using the Courant number with specifications outlined in Table 6. The unsteady flow computation settings are outlined in Table 7.

Table 6: Advanced Time Step Controls

Control	Values
Maximum Courant:	1
Minimum Courant:	0.4
No. of steps below Minimum before doubling:	3
Maximum number of doubling base time step:	1 (60 sec)
Maximum number of halving base time step:	5 (0.94 sec)

Table 7: Computation Settings

Setting	Values
Computation Interval:	30 sec
Mapping Output Interval:	6 min
Hydrograph Output Interval:	1 min
Detailed Output Interval:	30 min

3 Calibration

The Turkey Creek and Brush Creek 2D models were calibrated using 2 historic storm events in the watershed. Storm events were chosen based on the magnitude of the event, ideally larger than a 10-percent annual chance event, as well as having occurred recently so that StormWatch data was available to calibrate to in both watersheds. The events selected for calibration were August 22, 2017 and May 28, 2020.

3.1 StormWatch Gages

The City of Overland Park employs a weather and flood warning system called StormWatch which is made up of monitoring stations throughout the metro region. These gages collect rainfall, stream level (stage), and more to provide information about current stream conditions. These gages were used to develop the precipitation input for the calibration events, utilizing the StormWatch precipitation gages and incremental rainfall for the periods of interest. These gages were also used to calibrate the model based on the recorded stages during the historic flood events throughout the watershed.

A total of 27 precipitation gages were used to develop the rainfall for the August 2017 event and a total of 26 precipitation gages were used to develop the May 2020 event. Gage TU08 did not collect precipitation data during the 2020 event. The rainfall data was entered into the HEC-RAS model as Meteorological Data using the gages as meteorological stations to develop the rainfall across the entire domain. The Thiessen Polygon Interpolation method was applied for the historic point gage data to apply the rainfall across the domains. The August 2017 event was simulated from 12:00 AM August 21, 2017 to 12:00 AM August 23, 2017 to capture the whole storm event. The May 2020 event was simulated from 12:00 PM May 27, 2020 to 12:00 PM May 29, 2020.

Stage data is collected at select StormWatch gages. There are 6 StormWatch gages within the Turkey Creek watershed and 4 within the Brush Creek watershed that collected stage measurements during the two selected events that were used calibrate the 2D model parameters. BR04 did not collect stage data for either event, and BR09 did not collect stage data for the 2020 event. BR07 collected data for the 2017 event but was not considered for calibration because the City of Overland Park was not confident in the gage datum as it has been relocated. The City of Overland Park provided gage datum information to ensure the appropriate zero datum was being applied when comparing the observed water surface elevations and the modeled water surface elevations, summarized in Table 8. For concrete channels, the channel bottoms and the zero datum are the same because scour is not taken into consideration. For natural stream channels, the relationship between the checkbar and zero datum is shown in Figure 15.

Table 8: Turkey and Brush Creek Stage Collection StormWatch Gages and Zero Datums

Gage site	Elevation location	Surveyed Elevation (NAVD88)	Checkbar Set (NAVD88)	Elevation of Zero Datum (NAVD88)
69th Street @ Turkey Creek **(TU02)**	"0" on staff	949.125	N/A	949.125
67th Street @ Turkey Creek **(TU03)**	Top of Check Bar	948.17	22.22	925.95
Johnson Dr. @ Turkey Creek **(TU04)**	Check Bar	919.62	26.61	893.01
Antioch Rd. @ Turkey Creek **(TU06)**	Check Bar	898.33	31.38	866.95
Metcalf Ave. @ Turkey Creek **(TU07)**	Check Bar	888.52	44.61	843.91
Lamar Ave. @ Turkey Creek **(TU08)**	Check Bar	861.13	35.19	825.946
Porter Park – Brush Creek **(BR02)**	"0" at Floor	928.55	N/A	928.55
74th Street @ Brush Creek **(BR03)**	"0" on Staff	920.87	N/A	920.87
66th Street & Mission Rd Master **(BR04)**	Floor at Conduit	889.29	N/A	889.29
Martway @ Rock Creek **(BR06)**	"0" at Floor	916.29	N/A	916.29
Belinder Ave. @ Brush Creek Master **(BR09)**	Floor at Conduit	854.57	N/A	854.57

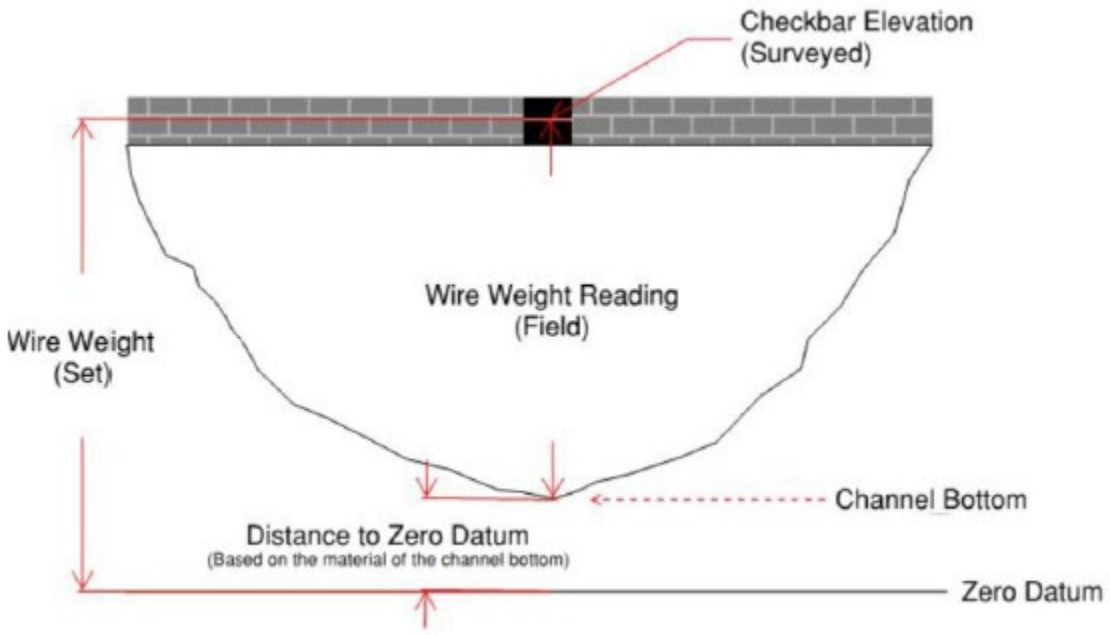


Figure 18: StormWatch Gage and Zero Datum Relationship (Source: Overland Park)

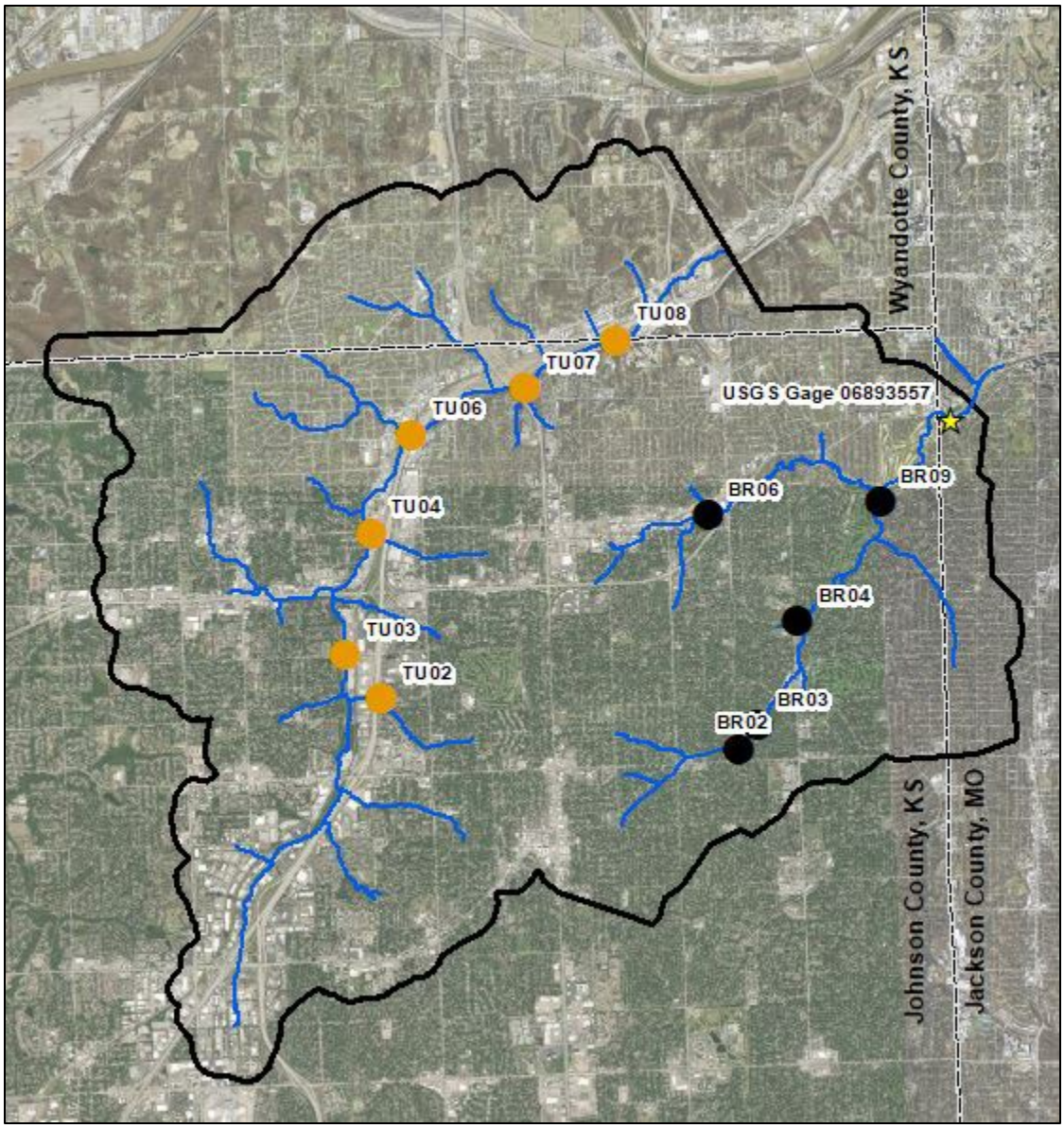


Figure 19: Location of Calibration Gages within the Turkey and Brush Creek Study Area

3.2 USGS Instantaneous Gage Data

There is one USGS gage along Brush Creek, just downstream of the Johnson County, KS and Jackson County, Missouri border, USGS 06893557 Brush Creek at Ward Parkway in Kansas City, Missouri (see Figure 17). The gage collects discharge and height for the periods of record for both historical calibration events. The data is collected on an approximately 15-minute interval. This gage was also used for the historical event calibration with this site having highest priority for calibration as the USGS monitors and maintains the stage-discharge relationship at the gage location.

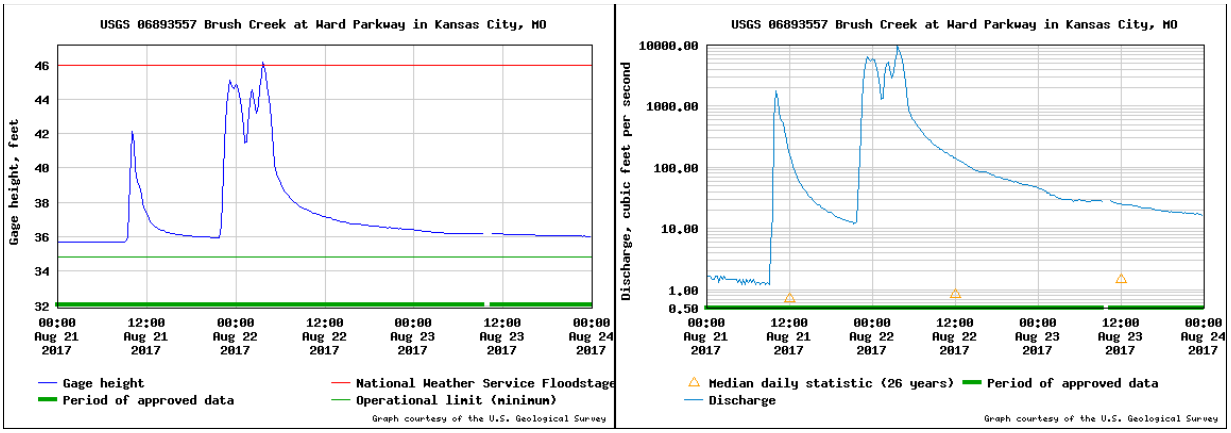


Figure 20: USGS Gage 06893557 Instantaneous Stage and Discharge Data for 2017 Calibration Event

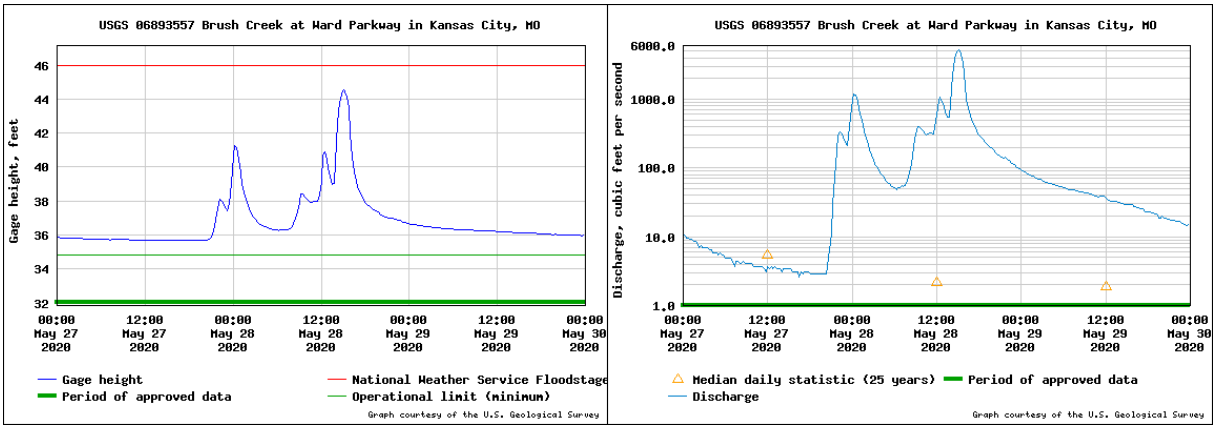


Figure 21: USGS Gage 06893557 Instantaneous Stage and Discharge Data for 2020 Calibration Event

3.3 Calibration Parameters

The desired goal of the calibration exercise was to arrive at a modeled stage height within six inches of the actual gage height recorded during historic storm events and to also closely mirror the timing of the peak gage height.

The parameters that were considered for adjustment during calibration were the abstraction ratio, infiltration Curve Numbers, Manning's n roughness coefficients, a hotstart initial condition, and upland terrain modifications to avoid artificial ponding where storm drain networks were present but not modeled.

Calibration of the parameters between the two watersheds were modified in conjunction with one another so that changes were agreeable between adjacent basins. Further discussion of the calibration parameters is reported in Section 3.4.

3.3.1 AUGUST 2017 EVENT

The StormWatch precipitation gages for the August 2017 event recorded rainfall depths ranging from approximately 4 to 9 inches across the study area (4 to 6 inches within Turkey and Brush Creek).

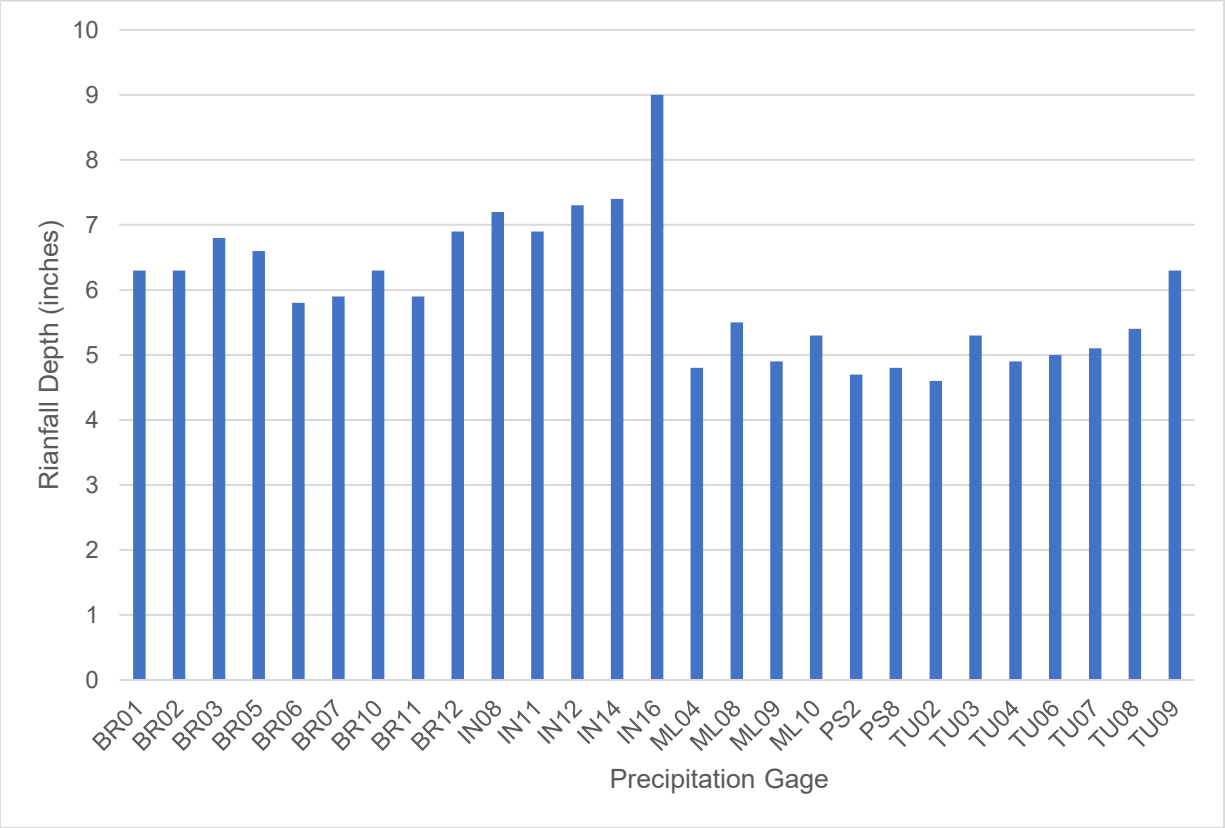


Figure 22: StormWatch Precipitation Totals for August 2017 Event

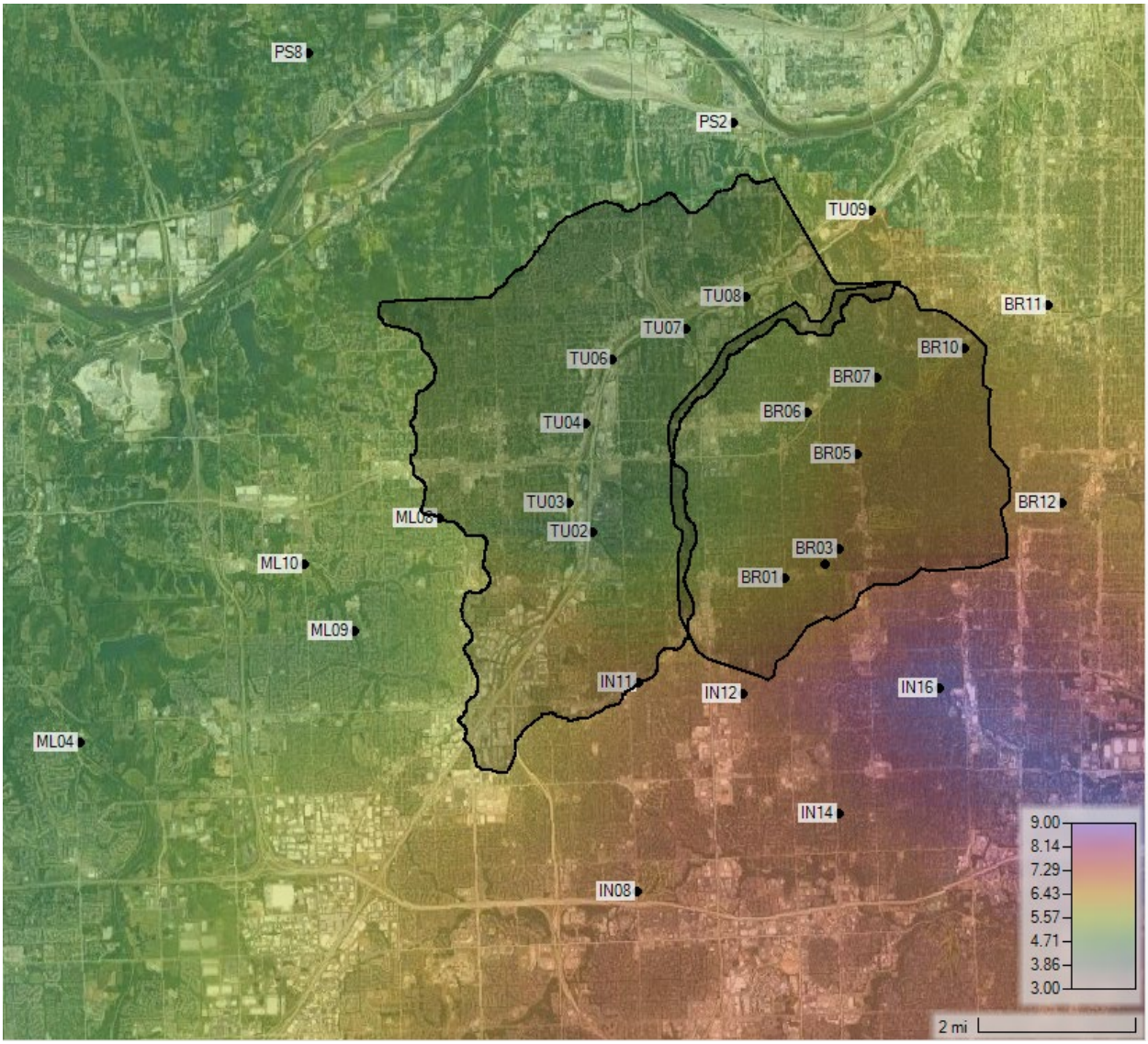


Figure 23: August 2017 Rainfall across Study Area based on StormWatch Gages

Base parameter results of the August 2017 event indicated the overall response was similar to the observed StormWatch stages; however, the magnitude of the peak stage within the Turkey Creek model was mostly larger than the observed StormWatch measurements with the largest difference at gage TU04 at almost 5 feet. Gage TU07 was the only gage within the 0.5-foot tolerance goal at -0.1 feet.

For the Brush Creek model, base parameter results of the August 2017 event also indicated the overall response was similar to the observed StormWatch stages; however, the magnitude of the peak stage within the Brush Creek model was mostly larger than the observed StormWatch measurements with the exception of gage BR06 with a modeled peak 3.1 feet less than the recorded peak. The largest positive difference in modeled stage versus recorded stage occurred at gage BR03 with a modeled stage peak 3 feet greater than the recorded peak. Gage BR02 was the only gage within the 0.5-foot tolerance goal

at 0.2 feet. The modeled stage peak at the USGS gage on brush creek at state line where 0.7 feet greater than the recorded peak.

Base model results compared to observed StormWatch data at each gage can be found in Appendix A.

Table 9: Uncalibrated Model Results Compared to StormWatch Data for August 2017 Event

Gage	Base Model Peak (ft)	Recorded Peak (ft)	Base Difference (ft)
TU02	955.7	**953.2**	2.5
TU03	938.6	**937.5**	1.1
TU04	909.4	**904.5**	4.9
TU06	886.8	**883.0**	3.7
TU07	860.2	**860.3**	-0.1
TU08	842.2	**841.0**	1.1
Absolute Average Turkey Creek			2.3
BR02	934.4	**934.3**	0.1
BR03	928.5	**925.6**	2.9
BR04	900.4	**899.5**	0.9
BR06	921.1	**924.3**	-3.2
BR09*	-	-	-
Absolute Average Brush Creek			1.7
USGS Gage 06893557	848.1	**846.7**	1.4

* Gage did not collect data for event

3.3.2 MAY 2020 EVENT

The StormWatch precipitation gages for the May 2020 event recorded rainfall depths ranging from approximately 1 to 5 inches across the study area (2 to 5 inches within Turkey and Brush Creek). Gage TU08 did not record any precipitation for the event.

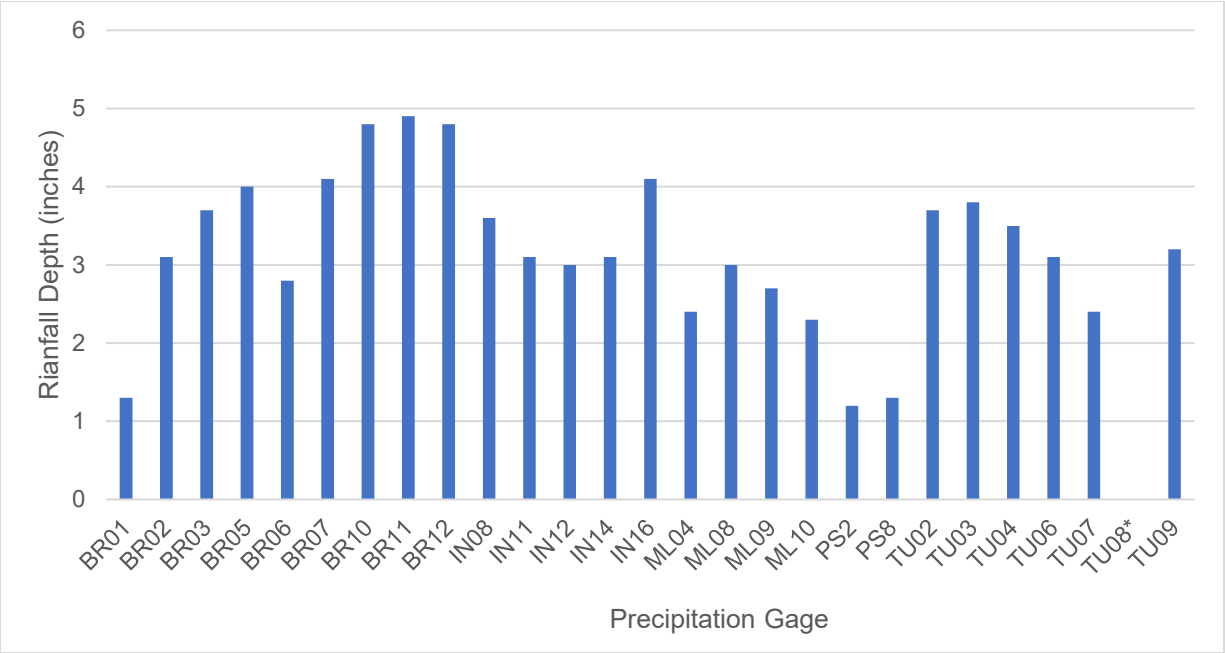


Figure 24: StormWatch Precipitation Gage Rainfall Hyetographs for May 2020 Event

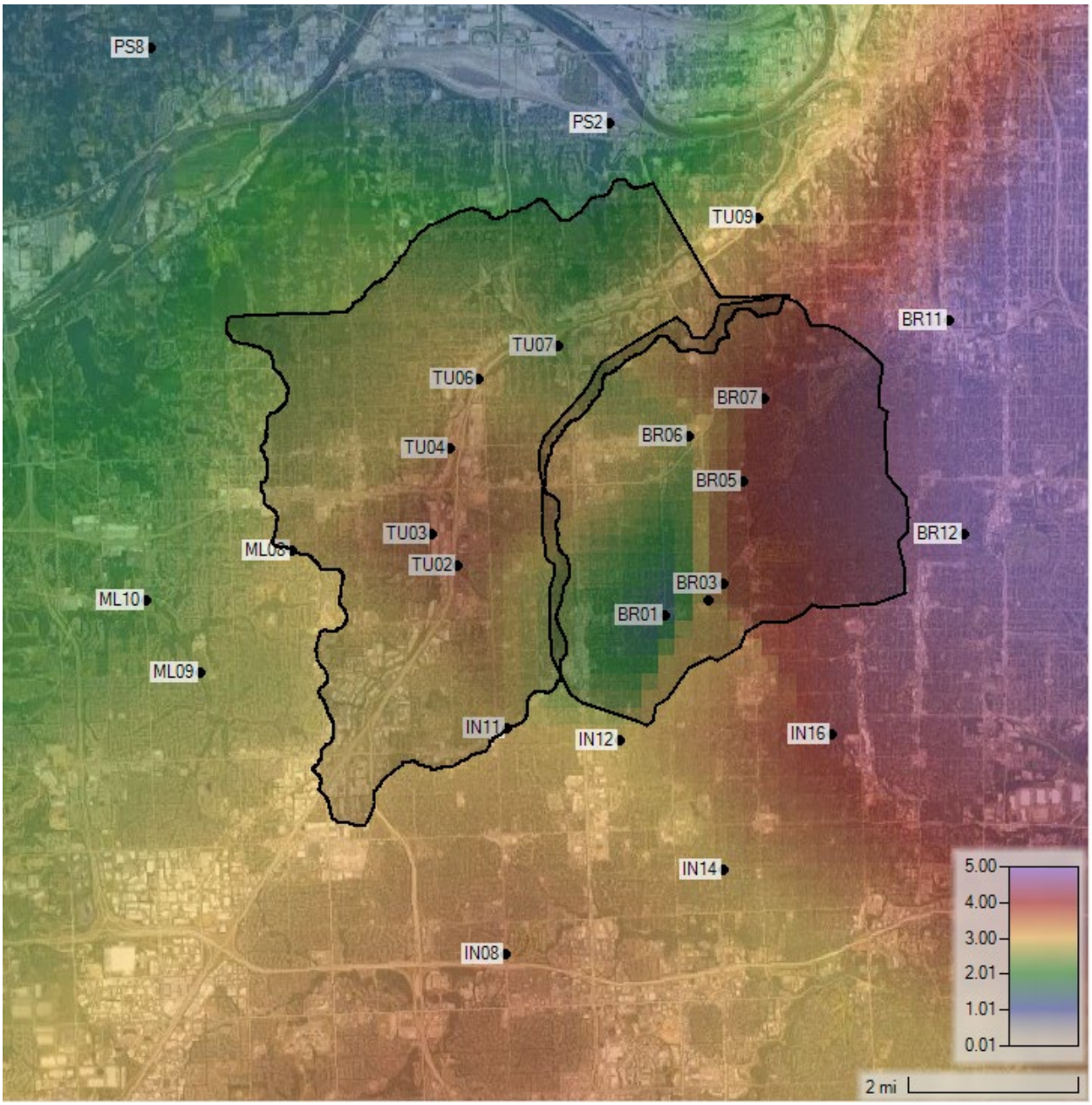


Figure 25: May 2020 Rainfall across Study Area based on StormWatch Gages

Similar to the 2017 event, initial model results showed the stage hydrograph shape closely followed the observed StormWatch stages; however, there were still discrepancies in Turkey Creek with the peak stages as half of the gages simulated higher than observed stages and half the gages simulated lower than observed stages. Again, gage TU04 showed the largest stage discrepancy of all the StormWatch gages in Turkey Creek, reporting a stage 2.7 feet higher than the observed stage.

The initial model results for Brush Creek also showed that the stage hydrograph shape closely followed the observed StormWatch stages; however, two of the three stormwatch gages on Brush Creek with recorded stages for the 2020 event recorded peak stages more than 0.5 feet higher than the base model results. Similarly, the USGS gage recorded a peak stage 1 foot higher than the base model results.

Base model results compared to observed StormWatch data at each gage can be found in Appendix B.

Table 10: Uncalibrated Model Results Compared to StormWatch Data for May 2020 Event

Gage	Base Model Peak (ft)	Recorded Peak (ft)	Base Difference (ft)
TU02	954.9	**953.7**	1.8
TU03	936.4	**937.3**	-1.1
TU04	907.2	**904.2**	2.7
TU06	884.7	**882.7**	2.0
TU07	858.6	**860.0**	-1.4
TU08	840.3	**840.9**	-0.6
Absolute Average Turkey Creek			1.5
BR02	930.3	**930.8**	-0.5
BR03	923.0	**924.4**	-1.4
BR04*	-	-	-
BF06	918.7	**919.8**	-1.1
BR09*	-	-	-
Absolute Average Brush Creek			0.9
USGS Gage 06893557	844.5	**845.0**	-0.5

* Gage did not collect data for event

3.4 Observed Event Calibration Data

3.4.1 PARAMETER SENSITIVITY

The modeling parameters considered for calibration were the curve number, abstraction ratio, and Manning's n roughness coefficients. These parameters have the least amount of certainty and impact the volume, peak, and timing of the flood response. These parameters are estimated from physical data about streams and watersheds, but the values are not exact. The watersheds were evaluated using a range for CN, abstraction ratio, and Manning's n values for the study area to evaluate the sensitivity to the model results. A high and low run were simulated for CN and Manning's n values. The high and low values for the Manning's n roughness coefficients were determined based on the ranges reported in the HEC-RAS 2D User's Manual. Additionally, the manual notes that n values for shallow, overland flow are generally much higher due to the relative roughness compared to the flow depth. While HEC-RAS has not published any Manning's n values for shallow, overland flow, TR-55 does have published values for sheetflow which were considered for an uplands calibration region. The uplands area was determined

based on a buffer from the channel locations identified in the land classification layer. Smaller, upland channels applied an approximate buffer of 100 feet and increases as the stream and corresponding floodplain areas are larger. The ‘Uplands Area’ for Turkey Creek and Brush Creek can be seen in Figure 26. Again, a high and low scenario were run.

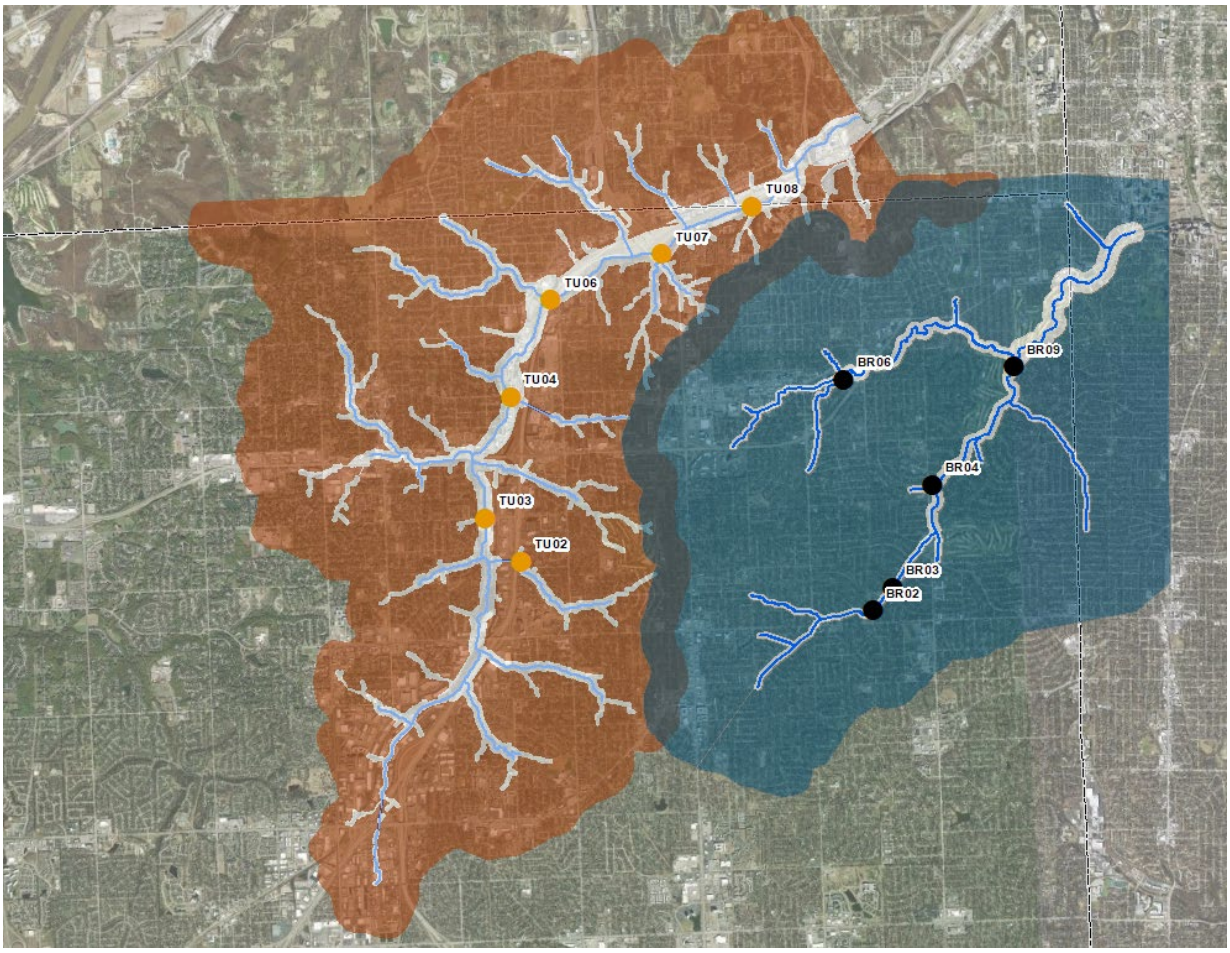


Figure 26: Uplands Area Refinement in Turkey and Brush Creek 2D Models

High and low CN simulations were run by increasing and reducing the base CN for pervious land classifications by a value of 3 to assess the sensitivity of the model response. No changes were made to impervious surface CN; these values were left at 98 and 100. The abstraction ratio was run under a base condition of 0.2; the sensitivity tests ran scenarios where the value was changed to 0.1 and 0.05.

Finally, a hotstart initial condition was run to assess the sensitivity of the results from the dry assumption. For this scenario, the end of simulation of the 10-percent annual chance synthetic event was used as the initial conditions for the simulation. The sensitivity runs are summarized in Table 11. More details on the specific modeling parameters used in the sensitivity testing and results at each comparison location can be found in Appendix A.

Table 11: Parameter Sensitivity Run Summary

Scenario Name	Parameter Updates
High Mann	Update base Manning's n values to high end of recommended values in HEC-RAS Manual (No modification to base CN)
Low Mann	Update base Manning's n values to low end of recommended values in HEC-RAS Manual (No modification to base CN)
High Mann - Upland Refine	Upland calibration region added with high end of TR-55 sheetflow values (Base Manning's n in floodplain, no modification to base CN)
Low Mann - Upland Refine	Upland calibration region added with low end of TR-55 sheetflow values (Base Manning's n in floodplain, no modification to base CN)
Low CN	Base CN reduced values by 3 (No changes to Impervious CN 98 or 100, no modification to base Manning's n)
High CN	Base CN increased values by 3 (No changes to Impervious CN 98 or 100, no modification to base Manning's n)
Ia = 0.1	Abstraction Ratio set to 0.1 (Base CN, Base Manning's n)
Ia = 0.05	Abstraction Ratio set to 0.05 (Base CN, Base Manning's n)
Hotstart	Initial conditions set to the end of simulation for the 10-percent rainfall event

Turkey and Brush Creek showed similar sensitivity to the modeling parameters investigated. Brush Creek was more sensitive to the hotstart, showing increases in stage at all gage check locations in the magnitude of 1-2 feet from the base condition while the Turkey Creek model only showed impacts of 0.5 feet or less at the comparison locations. This is likely due to more runoff storage existing in the Brush Creek Watershed.

The most impactful parameter to lowering the stages were the addition of the upland Manning's refinement regions, with the higher Manning's upland refinement resulting in the lowest stages in both basins.

Increasing and decreasing the CN showed minor impacts to the stage in both basins, resulting in an increase or decrease of less than 0.5 ft, on average. The impact of reducing the abstraction ratio similar effect as increasing the CN by 3, with an increase in stage of approximately 0.5 ft. The Brush Creek and Turkey Creek models showed different responses to changing the abstraction ratio from 0.1 to 0.05. In the Turkey Creek model, the further reduction to 0.05 resulted in slightly higher stages in the order of 0.25 ft.

HEC-RAS peak stage and flow rates were expected to be inversely related to the initial abstraction ratio as it impacts the total volume of runoff in the watershed. However, results in Brush Creek showed that as the initial abstraction ratio was reduced from 0.2 to 0.1 and finally 0.05 the peak stage and flow rate at stream gages on the downstream end of the watershed were highest with the 0.1 value followed closely by 0.05 and then the base 0.2. This can be explained by the effect of how HEC-RAS handles the initial abstraction ratio on hydrograph timing. As the initial abstraction ratio is increased the time to the start of hydrograph runoff is also increased. As shown in Figure 27. This change in the time of runoff initiation changes the hydrograph's shape as it attenuates downstream and causes artificial

alignment of flow peaks. Based on the sensitivity testing, the watersheds are not sensitive to abstraction ratios less than 0.1.

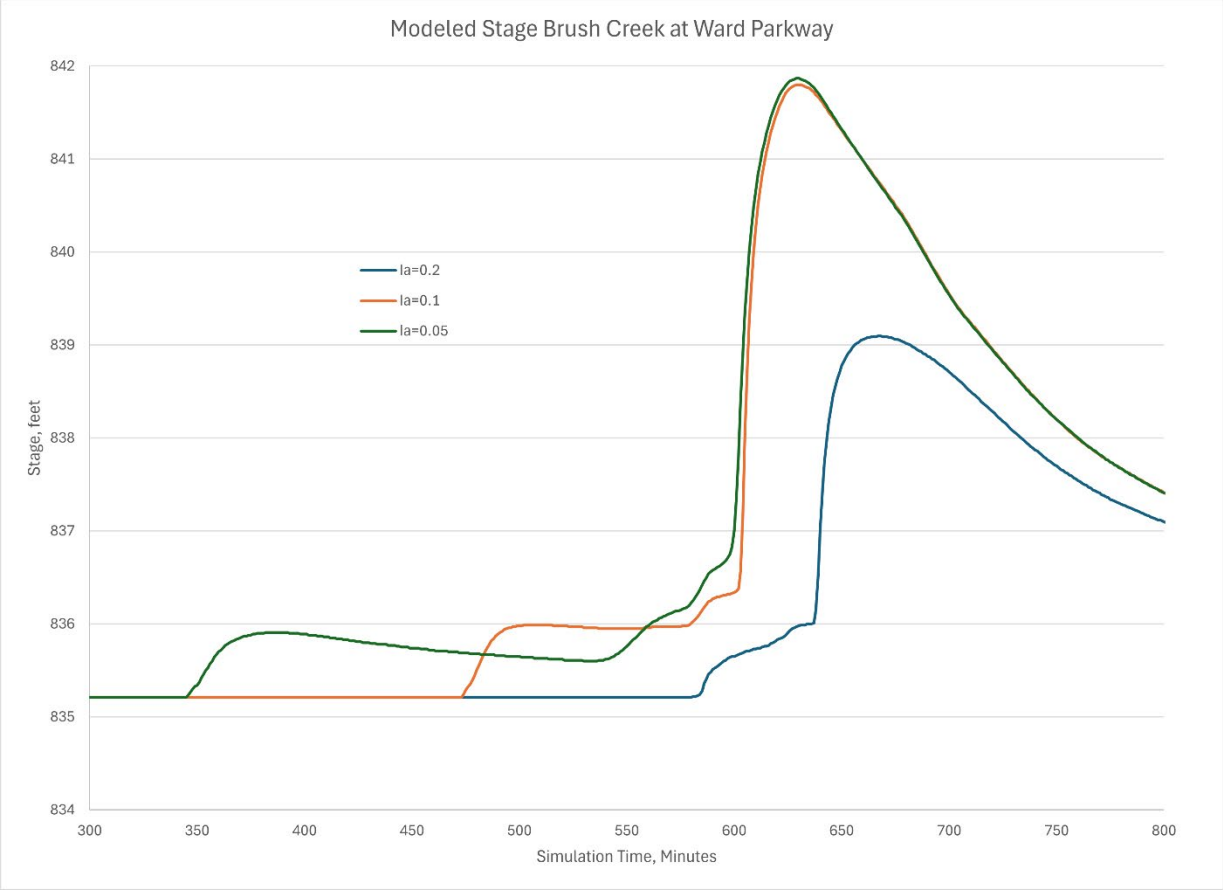


Figure 27: Abstraction Ratio Sensitivity in Brush Creek Model

3.4.2 FINAL MODEL PARAMETERS

Initial discussions with the City of Overland Park regarding the previous datums of the StormWatch gages indicated there could be some uncertainty associated with historical readings; however, there is no indication specific gages within the Turkey or Brush Creek watersheds should be considered unreliable for the historical events. Therefore, equal weight was given to all StormWatch gage locations when considering the basin calibration. Base results and test runs revealed some gage locations with higher stages than observed and others with lower stages than observed indicating that uniform modifications throughout the watershed would not result in reaching a 0.5 foot calibration tolerance at each gage. Accordingly, the model parameter calibration focused on minimizing the absolute difference in stage at all of the StormWatch gage locations.

Based on the initial parameter sensitivity testing and the majority of the gage locations for both events producing higher stages than the observed data, an upland Manning's n refinement region was added. Additionally, the abstraction ratio was reduced to represent the runoff more accurately in the fully developed basins.

Multiple iterations of the August 2017 and May 2020 event were simulated with varying degrees of change to the Manning's n roughness coefficients to evaluate the impact to the resulting stage at each of the StormWatch calibration locations.

In the Brush Creek Watershed the calibrated parameter values from the Turkey Creek Watershed model where used. A Manning's n refinement region was added within the stream channel at the downstream reaches of Brush Creek and Rock Creek. Channel roughness along Brush Creek and Rock Creek ranged from 0.029 to 0.035.

For the Turkey Creek watershed, for both calibration events, the model results at gages TU04 and TU06 were consistently much higher than the observed stages. Just downstream of TU06, gages TU07 and TU08 exhibited modeled stages much closer to the StormWatch stages. In an effort to avoid decreasing stages everywhere in the watershed to get gages TU04 and TU06 more in-line with the observed data, Manning's n calibration regions were added along Turkey Creek to adjust the channel roughness values at different locations. Channel roughness along Turkey Creek ranged from 0.025 to 0.035 with the lower n values along the stretch of Turkey Creek along TU04 and TU06.

The final calibrated Manning's n roughness coefficients for the Turkey and Brush Creek watersheds are summarized in Table 12. For the uplands area, it was assumed that the roughness for trees was controlled by grass understory cover because the watershed is highly developed and no true forest areas are present.

Table 12: Calibrated Manning's n Values based on Historic Events

Description	Base n Value	Calibrated Values	
		Floodplain n Value	Upland n Value
Trees	0.085	0.060	0.140
Natural Channel	0.035	0.035	0.035
Concrete Channel	0.015	0.015	0.015
Pavement	0.020	0.015	0.020
Grass	0.030	0.030	0.140
Buildings	1	1	1
Barren Land	0.025	0.025	0.045
Open Water	0.025	0.025	0.030

There were no modifications made to the base curve numbers (Table 4). The final abstraction ratio for both basins was set to 0.1.

Based on the sensitivity testing, there was minimal impact to peak stages in the Turkey Creek watershed when applying the hotstart. While the Brush Creek model showed more impact to a hotstart, the observed data suggested that the model was overestimating stage under this initial condition. This could be due to runoff storage no longer being available. A hotstart was not applied to either watershed model for the final calibration.

The final calibrated model results for the August 2017 and May 2020 historic events are summarized in Table 13. Stage hydrograph plots for each gage location can be found in Appendix B and C.

Table 13: Calibrated Model Results Compared to StormWatch Data for August 2017 Event

Gage	Base Model Peak (ft)	Calibrated Model Peak (ft)	**Recorded Peak (ft)**	Base Difference (ft)	Calibrated Difference (ft)
TU2	955.7	**954.5**	**953.2**	2.5	**1.3**
TU3	938.6	**937.9**	**937.5**	1.2	**0.4**
TU4	909.4	**907.0**	**904.5**	4.9	**2.5**
TU6	886.8	**884.4**	**883.0**	3.8	**1.4**
TU7	860.2	**859.8**	**860.3**	-0.1	**-0.5**
TU8	842.1	**841.6**	**841.0**	1.1	**0.6**
Absolute Average Turkey Creek				2.3	**1.1**
BR02	934.4	**933.9**	**934.3**	0.1	**-0.4**
BR03	928.5	**927.7**	**925.6**	2.9	**2.1**
BR04	900.4	**899.7**	**899.5**	0.9	**0.2**
BF06	921.1	**920.5**	**924.3**	-3.2	**-3.8**
BR09*	-	**-**	-	-	-
Absolute Average Brush Creek				1.7	**1.5**
USGS Gage 06893557	848.1	**847.7**	**846.7**	1.4	**1**

* Gage did not collect data for event

Table 14: Calibrated Model Results Compared to StormWatch Data for May 2020 Event

Gage	Base Model Peak (ft)	Calibrated Model Peak (ft)	**Recorded Peak (ft)**	Base Difference (ft)	Calibrated Difference (ft)
TU2	954.9	**953.9**	**953.7**	1.3	**0.2**
TU3	936.4	**935.6**	**937.3**	-0.9	**-1.7**
TU4	907.2	**904.8**	**904.2**	3.1	**0.6**
TU6	884.7	**882.6**	**882.7**	2.0	**-0.5**
TU7	858.6	**857.9**	**860.0**	-1.4	**-2.1**
TU8	840.3	**839.5**	**840.9**	-0.6	**-1.3**
Absolute Average Turkey Creek				1.5	**1.1**

BR02	930.3	930.1	**930.8**	-0.5	**-0.7**
BR03	923.0	922.8	**924.4**	-1.04	**-1.6**
BR04*	-	-	**-**	-	**-**
BF06	918.7	918.7	**919.8**	-1.1	**-1.1**
BR09*	-	-	**-**	-	**-**
Absolute Average Brush Creek				0.9	**0.8**
USGS Gage 06893557	844.5	844.2	**845.0**	-0.5	**-0.8**

* Gage did not collect data for event

For the Turkey Creek model, it was not possible to achieve a 0.5 foot calibration at each gage location due to the differences observed at the StormWatch gages within the basins. Compared to the observed, TU02, TU04, and TU06 reported model stages higher than observed while TU03, TU07, and TU08 reported model stages at or lower than observed. Therefore, universal changes to the model with inversely impact these gages. The applied model adjustments reduced the overall absolute difference at the gage locations while not being overly conservative to lower the stages at TU04 and TU06 that would likely result in artificially low stages at TU03, TU07, and TU08. Figure 27 below shows the initial (base) differences between the 2D model and observed StormWatch stages and the final calibrated differences.

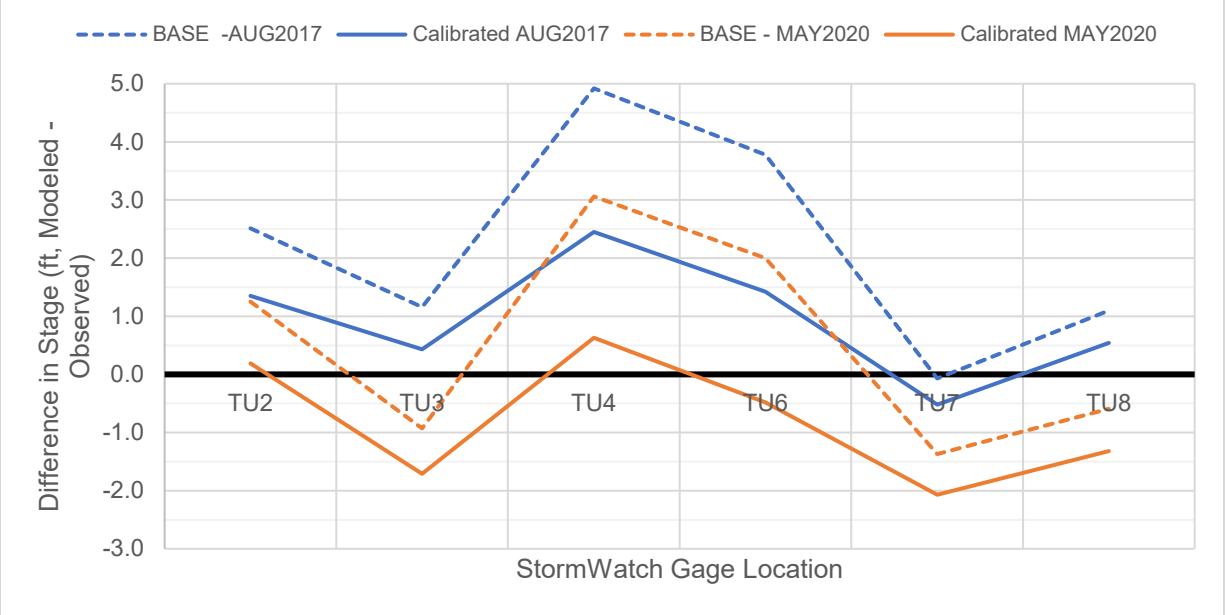


Figure 28: Differences in Turkey Creek Modeled versus Observed Stages from Base and Calibrated Events

Simularly for the Brush Creek model, it was not possible to achieve a 0.5 foot calibration at each gage location due to the differences observed at the StormWatch gages within the basins. Compared to the observed, BR03, and BR04 reported model stages higher than observed while BR06 reported model stages at or lower than observed. Therefore, universal changes to the model will inversely impact these

gages. The applied model adjustments reduced the overall absolute difference at the gage locations while not being overly conservative to lower the stages at BR06. Figure 27 below shows the initial (base) differences between the 2D model and observed StormWatch stages and the final calibrated differences.

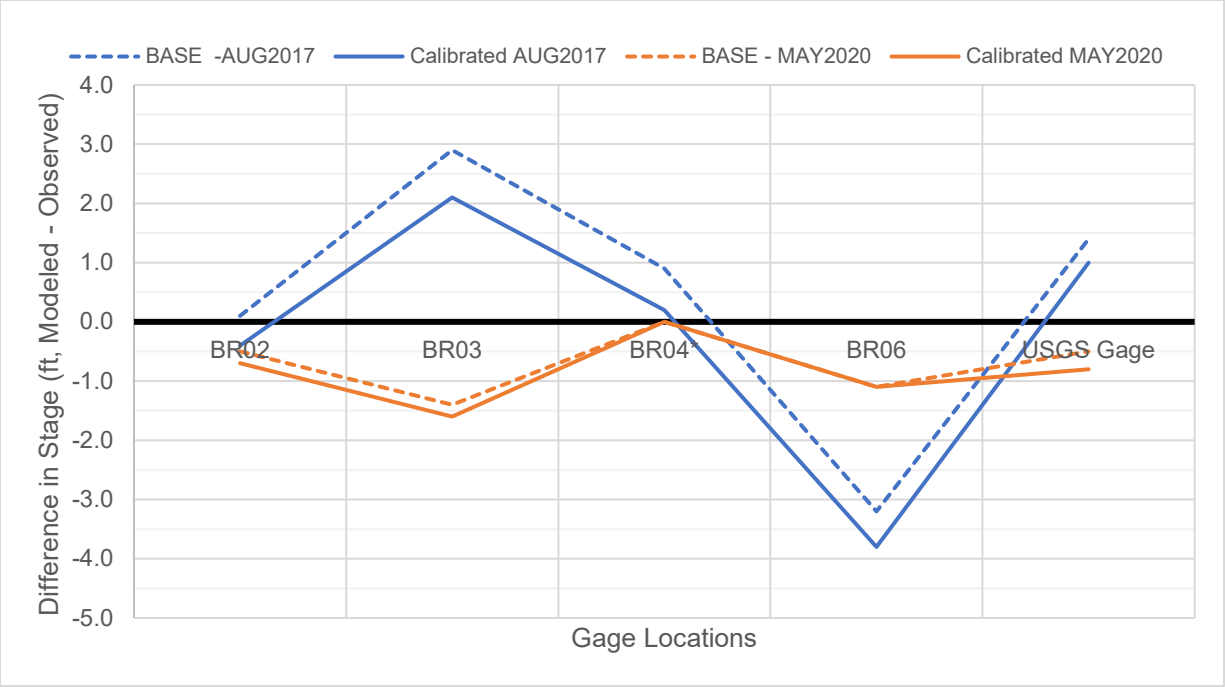


Figure 29: Differences in Brush Creek Modeled versus Observed Stages from Base and Calibrated Events (BR04 no observed data for 2020)

3.4.3 MODEL LIMITATIONS

Calibration of the watershed models to the StormWatch gages could not meet the 0.5 foot criteria initially proposed. There are a variety of factors that potentially impact the watershed response. While the StormWatch point precipitation data is collected at a high-resolution interval, it is only at point specific locations. The interpolation across the watershed likely results in some over and under estimation of rainfall for the historic events.

Additionally, the StormWatch calibration locations record the water stage which is highly dependent on the local channel hydraulics. The 2D model relies on the LiDAR and channel survey to route the flood water and determine water surface elevations. Turkey, Brush, and Rock Creeks can be turbulent channels with pools and riffles. Discrepancies between the LiDAR cross-sectional area and the true channel cross-sectional area at the StormWatch gage locations may also contribute to differences between the modeled stages and the observed stages.

Finally, HEC-RAS currently does not have the capability to incorporate enclosed storm drain networks. Both the Turkey and Brush Creek watersheds are highly urbanized with extensive storm drain

networks. These systems were not modeled in the 2D domains and therefore, the watershed response timing is impacted as seen in the slight delay in 2D hydrograph response at the StormWatch gages compared to the observed data. The impact of these systems would be most impactful on smaller events where these systems can handle the runoff volume. The 2D models have incorporated terrain modifications to simulate some conveyance through these systems but is not intended to model the enclosed systems. While the storm drain systems are not included, the model can still provide valuable information in the upland areas where these systems exist. Overland flows are routed based on the terrain and highlight potential consequence of failure and inundation extents.

3.5 July 2017 Validation

An additional high flow event was evaluated post calibration to validate the model response to large historic events. The July 2017 StormWatch rainfall was simulated in the model and results were compared to the observed StormWatch stages at the gage locations. Rainfall for this event was ranged from approximately 4 to 6 inches across the watershed, as shown in Figure 29.

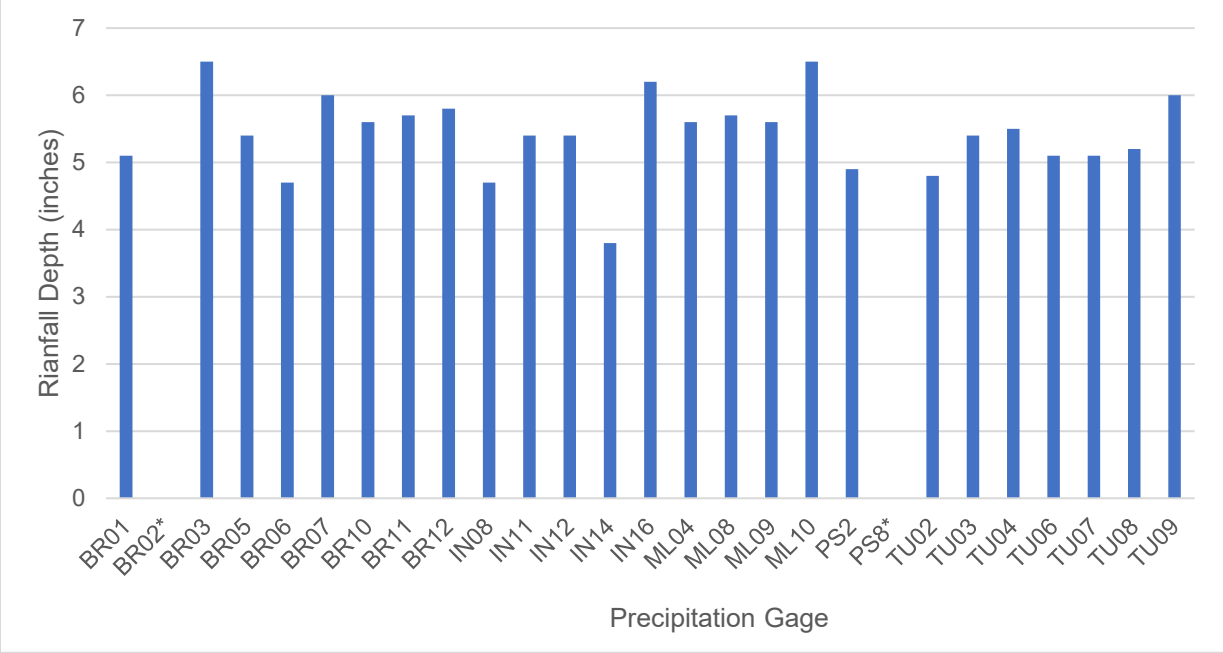


Figure 30: StormWatch Precipitation Totals for July 2017 Event (*gage did not collect data)

Similar to the calibration events, in the Turkey Creek watershed, TU04 and TU06 reported stages higher than the observed, while TU02, TU03, and TU07 reported stages lower and TU08 was in agreement with observed. The model produced similar results for this high flow event as observed during calibration events. While the validation event was still not within the desired 0.5-foot tolerance, the average absolute difference for all gages was 1.2 feet. Figure 30 below shows the differences in stage at each gage location for the calibration and validation events in Turkey Creek.

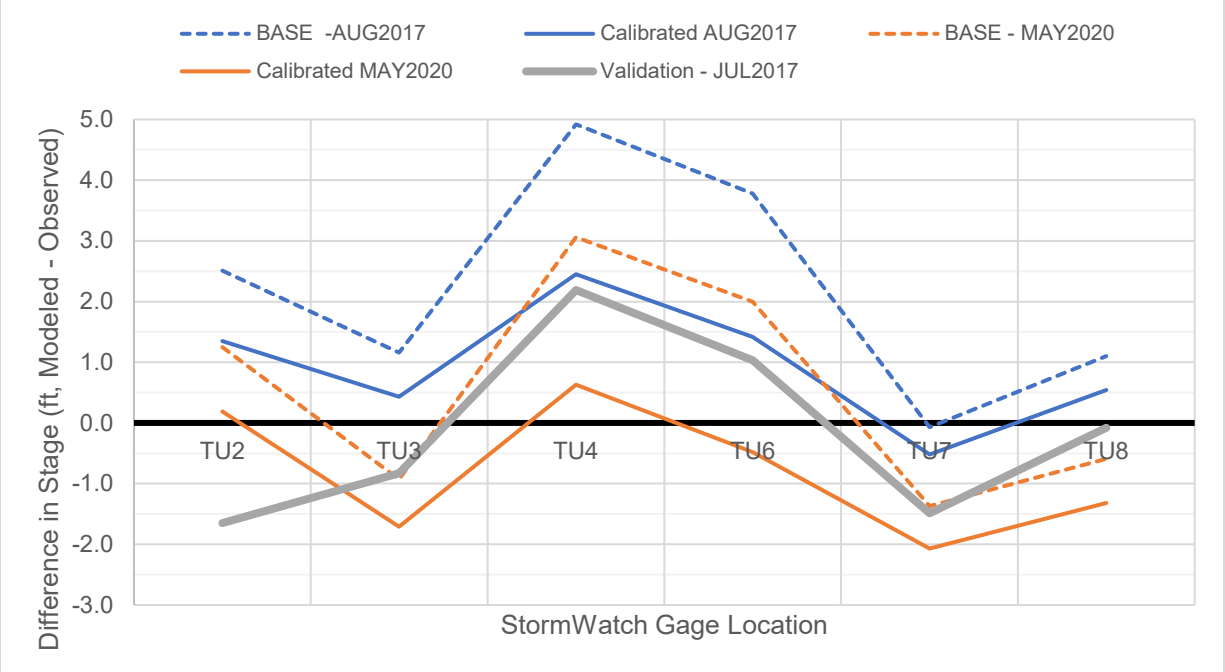


Figure 31: Differences in Turkey Creek Modeled versus Observed Stages with July 2017 Validation Event

While the results of the May 2020 event in Brush Creek showed model estimates lower than the observed stages, the larger flood events did not demonstrate this underestimation. The July 2017 event matched the observed data almost exactly for the BR02 and BR06 gages and were higher than observed for the 3 additional gages.

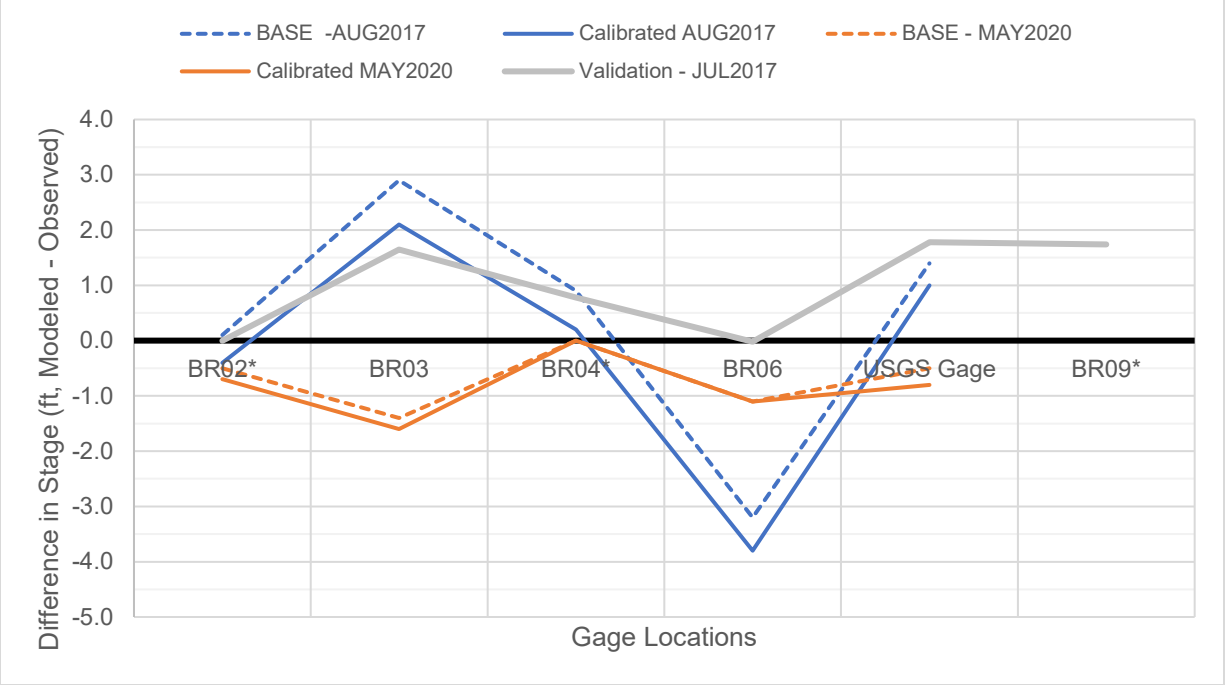


Figure 32: Differences in Brush Creek Modeled versus Observed Stages with July 2017 Validation Event (BR02 no data for July 2017, BR04 no data for May 2020, BR09 no data for August 2017 or May 2020)

Stage hydrographs for the July 2017 event at the StormWatch gage and USGS gage locations can be found in Appendix D.

4 Synthetic Event Simulation

The 50-, 10-, 4-, 2-, 1-, 1plus-, and 0.2-percent annual chance exceedance events were also applied to the calibrated Turkey and Brush Creek models. Single precipitation depths based on NOAA Atlas 14, using a 12-hr, 3-hr peak, distribution were simulations as discussed in Section 2.1.1.

4.1 USGS Gage Frequency Analysis & Rating Curve

There is one USGS stream gage located within the Turkey Creek watershed in the City of Merriam along Turkey Creek in Johnson County, Kansas. There is one USGS stream gage located just downstream of the Johnson County boundary along Brush Creek in Jackson County, Missouri. Only the USGS gage along Brush Creek collects instantaneous streamflow and stage data.

Both of these gages have 20+ years of peakflow data and these gages were used for the validation of flows in the Turkey and Brush Creek models. The USACE software HEC-SSP was used to develop estimates for the 1-percent-annual-chance event using 17C analysis. The results of that analysis for the

USGS gages in the Turkey and Brush Creek watersheds are summarized in Table 15. HEC-SSP output can be found in Appendix E.

Table 15: USGS Gage Analysis for 2D Watershed Calibration

USGS Station ID	Drainage Area (sq.mi)	Flooding Source	Years of Record	1% Annual Chance Discharge (cfs, 17C)
06892800	6.76	Turkey Creek	22 (1974-1986*, 2006-2016)	9,678
06893557	12.2	Brush Creek	24 (1999-2022*)	12,890

*gage records missing year 1985 for Turkey Creek and 2016 for Brush Creek

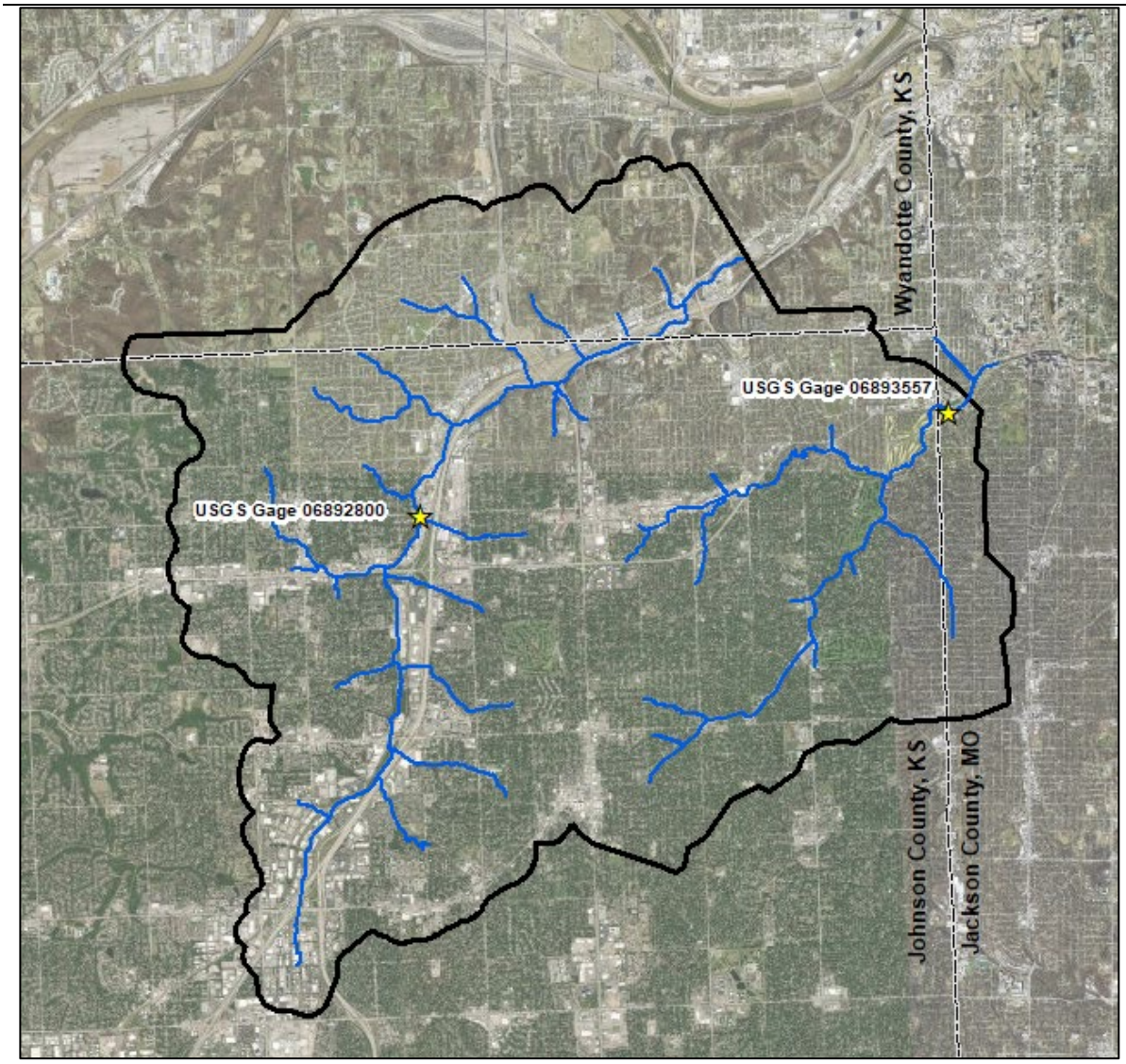


Figure 33: Location of USGS Gages within the Turkey and Brush Creek Watersheds

4.2 Flow Comparison with Regression Estimates & Effective Data

FEMA guidance states that base flood discharges are considered reasonable if they are generally within the prediction error of the regression and gages station estimates. The 1-percent annual chance discharges from the 2D, calibrated models were compared to both the USGS bulletin 17C analysis and regression estimates based on the current document for the state of Kansas (SIR 2017-5063). The prediction of error associated with statistical gage analysis is defined as the upper 84% confidence limit. The standard error of prediction for the Kanas regression equations for Region 1 (where Johnson County falls) is 51.9%.

Flow check locations were developed based on the locations reported in the effective FIS for Johnson County (August 2009) for streams within the Turkey and Brush Creek watersheds. Comparisons between the Effective FIS, Regression, and 2D Models can be found in Table 16 and Table 17, as well as graphically in Figure 29 and Figure 35.

Table 16: 1% Annual Channce Flow Comparison for Turkey Creek Watershed

Stream	Location	DA (sqmi)	1% Annual Chance Discharge (cfs)				
			Effective	Regression	Regres. low[1]	Regres. high[1]	2D Model
Turkey Creek	At Metcalf Ave	17.0	17,404	11,600	5,580	17,620	14,934
	Approximately 1,600 ft downstream of Antioch Rd	14.6	15,971	10,700	5,147	16,253	13,267
	At convergence of Turkey Creek Tributary F	10.8	15,007	9,290	4,468	14,112	12,077
	At West 67th Street*	6.7	8,875	7,350	3,535	11,165	7,358
	At Railroad	4.4	7,434	5,960	2,867	9,053	5,056
	At West 81st Street	1.6	5,085	3,710	1,785	5,635	2,289
	At Marshall Drive	0.8	2,377	2,600	1,251	3,949	1,174
Turkey Creek Tributary C	At Merriam Drive	1.5	3,268	3,470	1,669	5,271	2,013
Turkey Creek Tributary F	Approximately 450 ft downstream of Goddard St	2.7	6,354	4,710	2,266	7,154	3,605
	Approximately 500 ft upstream of Goddard St	2.4	5,242	4,400	2,116	6,684	3,034
	At Flint Street	1.2	3,319	3,090	1,486	4,694	1,867
Turkey Creek Tributary J	At West 81st Street	1.1	2,946	3,000	1,443	4,557	1,546
	Approximately 1,950 feet upstream of Mastin St	0.8	2,573	2,680	1,289	4,071	1,383
Turkey Creek Tributary JA	Approximately 1,100 ft upstream of East Frontage Rd	0.9	2,443	2,730	1,313	4,147	1,343
	At Antioch Road	0.6	1,082	2,190	1,053	3,327	1,017
	At West 79th Street	0.3	642	1,520	731	2,309	796

*Location of USGS Gage 06892800 with 17C discharge of 9,678 cfs
[1]Regression low/high based on 51.9% standard error of prediction

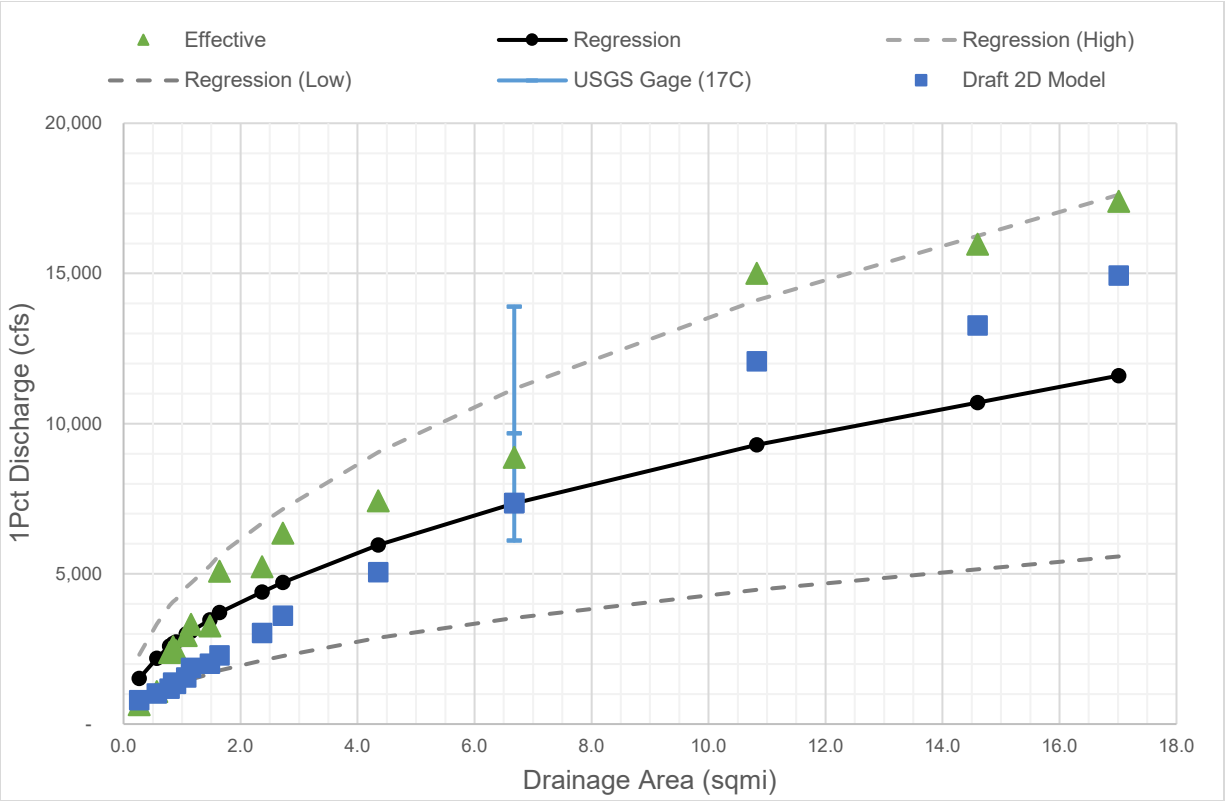


Figure 34: Flow Comparisons for the Turkey Creek Watershed

Table 17: 1% Annual Channce Flow Comparison for Brush Creek Watershed

Stream	Location	DA (sqmi)	1% Annual Chance Discharge (cfs)				
			Effective	Regression	Regres. low[1]	Regres. high[1]	2D Model
Brush Creek	At County Boundary	12.7	15,741	10,046	4,832	15,259	15,869
	At Indian Lane	7.2	10,052	7,642	3,676	11,608	9,585
	At West 63rd Street	5.4	7,974	6,649	3,198	10,100	7,298
	Approximately 450 feet downstream of Tomahawk Rd	4.0	7,141	5,750	2,766	8,734	5,063
	at West 71st Street	3.2	5,460	5,180	2,492	7,869	4,631
	at West 75th Street	1.4	2,720	3,516	1,691	5,341	2,459
	at West 79th Street	0.5	1,537	2,174	1,046	3,302	951
Rock Creek	Approximately 1,100 feet upstream of the confluence with Brush Creek	4.5	8,103	5,982	2,877	9,086	6,071

Stream	Location	DA (sqmi)	1% Annual Chance Discharge (cfs)				
			Effective	Regression	Regres. low[1]	Regres. high[1]	2D Model
	Just upstream of the confluence of Rock Creek Tributary B	3.6	7,386	5,259	2,530	7,989	4,983
	Approximately 1,075 feet upstream of Sheridan Drive	2.8	5,894	4,800	2,309	7,291	3,936
	At the confluence of Rock Creek Tributary G	0.4	1,357	2,114	1,017	3,211	1,190
	at Lamar Avenue	0.3	681	1,532	737	2,327	701
Rock Creek Tributary A	At Shawnee Mission Parkway	0.5	1,506	2,073	997	3,149	732
	At West 53rd Street	0.3	787	1,462	703	2,221	514
	Approximately 1,300 feet upstream of west 52nd Place	0.2	488	1,117	537	1,696	274
Rock Creek Tributary B	At Shawnee Mission Parkway	0.1	336	1,001	482	1,521	202
Rock Creek Tributary D	at West 55th Street	0.3	861	1,551	746	2,355	490
Rock Creek Tributary E	at West 57th Street	0.4	1,108	1,815	873	2,757	601

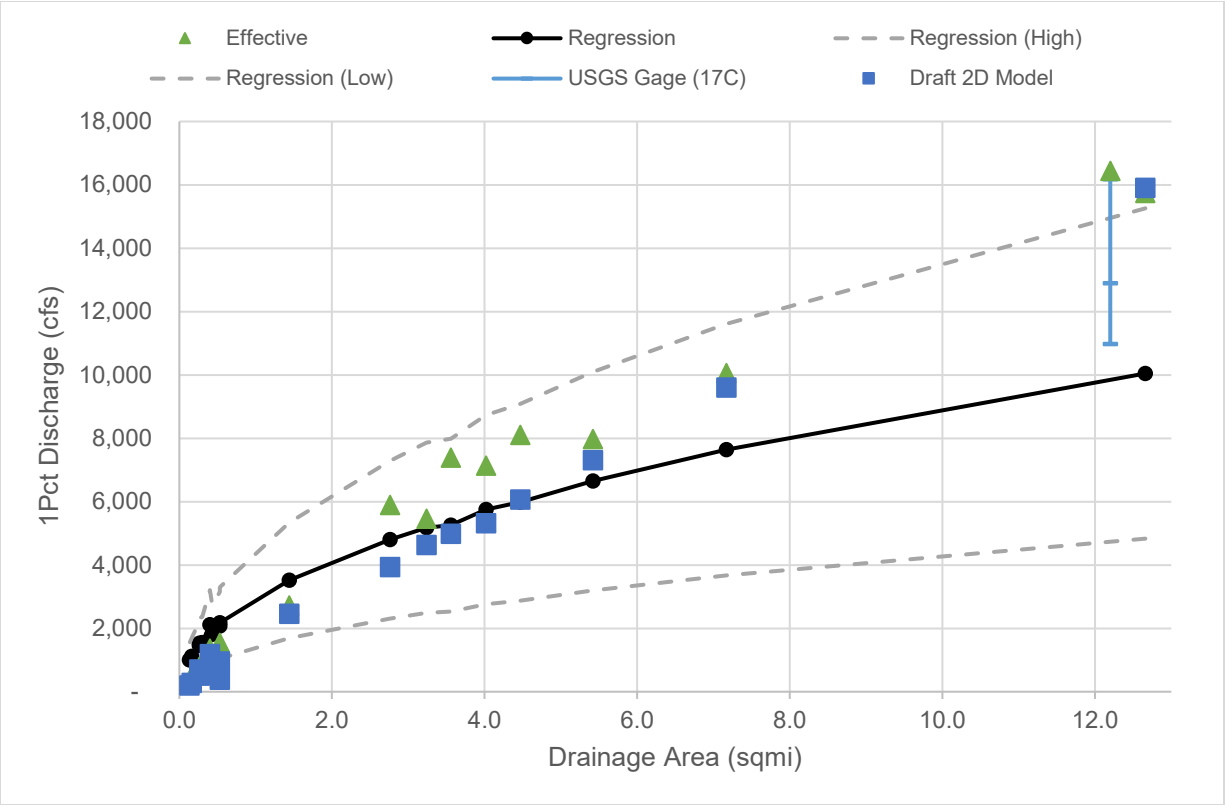


Figure 35: Flow Comparisons for the Brush Creek Watershed

In Turkey Creek, the 2D watershed model with the 12-hour, 3-hr peak nested storm generally produced 1-percent annual chance discharges lower than the effective flood insurance study discharges. The discharges fall within 1 standard error of the Kansas regression equation estimates and within the 84-percent confidence interval of the available USGS gage data along the stream. For the smallest drainage areas reported along Tributary JA, the 2D model produced equal and slightly higher discharges than the effective flows; however, both the 2D model and effective flows are on the lower end of the regression estimates for the smaller drainage areas.

Similarly, the majority of the 1-percent annual chance event flows in the Brush Creek watershed 2D model are lower than the effective flood insurance study discharges but are generally within the standard error of the Kansas regression estimates. At the most downstream portion of the watershed, downstream of the confluence with Rock Creek, the flows along Brush Creek are similar to the effective and slightly higher than the USGS 17C gage analysis. Further investigation was conducted at this location and it was determined that the higher discharge was reasonable for the watershed. The USGS peak flow records are from 1999 to 2023. It does not include the floods of record along Brush Creek from 1977 and 1998.

4.3 Summary of Discharges

The final flows for the Turkey and Brush Creek models are summarized below in Table 18 and Table 19 below.

Table 18: Final Summary of Discharges for Turkey Creek Watershed

Stream	Location	DA (sqmi)	Annual Chance Peak Discharge (cfs)					
			10%	4%	2%	1%	1% +	0.2%
Turkey Creek	At Metcalf Ave	17.0	8,293	10,922	12,492	14,934	17,095	20,482
	Approximately 1,600 ft downstream of Antioch Rd	14.6	7,360	9,655	10,890	13,267	15,230	17,839
	At convergence of Turkey Creek Tributary F	10.8	6,135	8,094	9,674	12,077	13,859	16,699
	At West 67th Street	6.7	3,899	5,184	6,185	7,358	8,382	10,112
	At Railroad	4.4	2,655	3,601	4,302	5,056	5,645	5,840
	At West 81st Street	1.6	1,173	1,601	1,911	2,289	2,606	2,936
	At Marshall Drive	0.8	578	802	960	1,174	1,392	1,656
Turkey Creek Tributary A	At Foster Street	0.5	263	401	511	639	769	901
Turkey Creek Tributary B	Just upstream of railroad	0.7	850	1,205	1,375	1,516	1,659	1,854
Turkey Creek Tributary C	At Merriam Drive	1.5	820	1,211	1,566	2,013	2,435	2,968
Turkey Creek Tributary CA	At W 51st Street	0.7	422	637	798	987	1,180	1,472
Turkey Creek Tributary D	At Knox Avenue	0.9	180	270	346	455	549	687
Turkey Creek Tributary E	Just upstream of Slater Street	0.5	253	381	485	598	743	923
Turkey Creek Tributary F	Approximately 450 ft downstream of Goddard St	2.7	1,689	2,423	2,958	3,605	4,304	5,354
	Approximately 500 ft upstream of Goddard St	2.4	1,378	1,969	2,429	3,034	3,564	3,913
	At Flint Street	1.2	789	1,166	1,450	1,867	2,305	2,834
Turkey Creek Tributary FA	At confluence with Turkey Creek Tributary F	1.1	195	286	358	461	567	721
Turkey Creek Tributary FB	At confluence with Turkey Creek Tributary F	1.3	555	758	918	1,121	1,349	1,683

Stream	Location	DA (sqmi)	Annual Chance Peak Discharge (cfs)					
			10%	4%	2%	1%	1% +	0.2%
Turkey Creek Tributary G	At confluence with Turkey Creek	0.3	352	496	621	891	1,127	1,358
Turkey Creek Tributary H	At W 70th Terrace	0.3	319	485	629	826	1,000	1,221
Turkey Creek Tributary I	At railroad	1.0	648	901	1,074	1,258	1,449	1,757
Turkey Creek Tributary J	At West 81st Street	1.1	714	1,000	1,263	1,546	1,902	2,402
	Approximately 1,950 feet upstream of Mastin St	0.8	603	876	1,077	1,383	1,659	2,102
Turkey Creek Tributary JA	Approximately 1,100 ft upstream of East Frontage Rd	0.9	607	880	1,092	1,343	1,593	1,883
	At Antioch Road	0.6	440	643	802	1,017	1,233	1,511
	At West 79th Street	0.3	341	495	626	796	960	1,207
Turkey Creek Tributary K	At confluence with Turkey Creek	0.3	281	415	509	610	700	786

Table 19: Final Summary of Discharges for Brush Creek Watershed

Stream	Location	DA (sqmi)	Annual Chance Discharge (cfs)					
			10%	4%	2%	1%	1%+	0.2%
Brush Creek	At County Boundary	12.7	7,276	10,594	13,006	15,869	16,176	22,973
	At Indian Lane	7.2	4,484	6,553	7,922	9,585	9,768	13,520
	At West 63rd Street	5.4	3,396	5,007	6,132	7,298	7,405	10,275
	Approximately 450 feet downstream of Tomahawk Rd	4.0	2,549	3,815	4,523	5,063	5,117	6,270
	At West 71st Street	3.2	2,141	3,201	3,900	4,631	4,702	6,893
	At West 75th Street	1.4	1,098	1,648	2,007	2,459	2,493	3,728
	At West 79th Street	0.5	419	624	767	951	973	1,479
Brush Creek Tributary A	At confluence with Brush Creek	1.5	1,016	1,486	1,861	2,307	2,359	3,616
Brush Creek Tributary B	At confluence with Brush Creek	0.4	287	419	502	614	627	1,001
Brush Creek Tributary C	At confluence with Brush Creek	0.3	190	275	326	394	403	595
Brush Creek Tributary D	At confluence with Brush Creek	0.5	256	389	510	657	671	996
Brush Creek Tributary E1	At confluence with Brush Creek	0.7	318	467	579	725	740	1,135

Stream	Location	DA (sqmi)	Annual Chance Discharge (cfs)					
			10%	4%	2%	1%	1%+	0.2%
Brush Creek Tributary E2	At confluence with Brush Creek	0.7	360	557	718	917	939	1,433
Brush Creek Tributary F	At confluence with Brush Creek	0.5	339	504	629	776	791	1,161
Rock Creek	Approximately 1,100 feet upstream of the confluence with Brush Creek	4.5	2,736	3,982	4,942	6,071	6,167	8,780
	Just upstream of the confluence of Rock Creek Tributary B	3.6	2,234	3,282	4,050	4,983	5,071	7,140
	Approximately 1,075 feet upstream of Sheridan Drive	2.8	1,857	2,672	3,224	3,936	4,006	5,719
	At the confluence of Rock Creek Tributary G	0.4	608	836	980	1,190	1,191	1,793
	at Lamar Avenue	0.3	350	489	583	701	711	1,028
Rock Creek Tributary A	At Shawnee Mission Parkway	0.5	299	447	579	732	750	1,192
	At West 53rd Street	0.3	204	313	399	514	522	826
	Approximately 1,300 feet upstream of west 52nd Place	0.2	123	176	223	274	277	420
Rock Creek Tributary B	At Shawnee Mission Parkway	0.1	107	150	178	202	202	250
Rock Creek Tributary C	At confluence with Rock Creek	0.1	25	66	95	156	159	345
Rock Creek Tributary CA	At confluence with Rock Creek	<0.1	81	85	87	92	92	101
Rock Creek Tributary D	At West 55th Street	0.3	211	304	472	490	522	694
Rock Creek Tributary E	At West 57th Street	0.4	263	399	490	601	613	897
Rock Creek Tributary F	At confluence with Rock Creek	0.6	388	578	716	872	891	1,347
Rock Creek Tributary G	At confluence with Rock Creek	0.4	238	327	452	541	552	775

4.4 Stage Comparison

Base flood elevations (BFEs) from the effective digital flood insurance rate map (DFIRM) and the 2D model were compared for streams with effective Zone AE mapping. Water surface elevations from the 2D model were pulled along the stream profile line and plotted with the effective BFEs to compare the two data sources.

Overall, the 2D model results either match the effective BFEs or are slightly lower. Some of the smaller tributaries to Turkey and Rock Creek show lower stages in the 2D model primarily due to a decrease in flows at these locations. Plots for all streams with effective BFEs on the current FIRM for the Turkey and Brush Creek watersheds are in Appendix F and include the following streams:

- Brush Creek
- Turkey Creek
- Turkey Creek Tributary C
- Turkey Creek Tributary F
- Turkey Creek Tributary J
- Rock Creek
- Rock Creek Tributary A
- Rock Creek Tributary B
- Rock Creek Tributary D
- Rock Creek Tributary E

5 Floodplain Mapping

Draft 1- and 0.2-percent annual chance event floodplains will be developed for the study streams. Due to HEC-RAS rain-on-grid methodology, post processing of the floodplain is necessary to remove the pluvial influence that gets reported based on the maximum water surface elevations of the model simulation.

ArcGIS will be used with the maximum water surface elevation outputs to rerender the fluvial flooding elevations associated with the study streams. The post-processing will be completed for the 1- and 0.2-percent annual chance events and will be submitted with the Floodplain Mapping task.

6 Conclusion

The results of the 2D BLE watershed models will be used to develop draft 1-percent and 0.2-percent annual chance floodplains based on the 2-foot resolution LiDAR. These draft floodplains will be included in the floodplain mapping deliverable for Johnson County and will be shared with communities (Lenexa, Overland Park, Prairie Village, Mission Hills, Merriam, Fairway, Mission, Roeland Park,

Westwood, and Westwood Hills) for their review. This data can be used as “best available”. Additional data development and model refinement may be performed based on input from community stakeholders.

7 References

1. Federal Emergency Management Agency, *Flood Insurance Study Johnson County, Kansas and Incorporated Areas*, August 3, 2009. Flood Insurance Study Number 20091CV001B.
2. U.S. Army Corps of Engineers, Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 6.3.1, USACE, Davis, CA. 30 September 2022.
3. National Oceanic and Atmospheric Administration, NOAA Atlas 14 Volume 8, Precipitation-Frequency Atlas of the United States: Midwestern States (Colorado, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Oklahoma, South Dakota, Wisconsin), Silver Spring, Maryland, 2013.
4. Natural Resources Conservation Service, National Engineering Handbook, Part 630 Hydrology, Chapter 4: Storm Rainfall Depth and Distribution, issued August 2019.
5. Olsson, *Indian Creek 2D Modeling Project (Indian Creek, Tomahawk Creek, Dyke Branch) Hydrology and Hydraulics*. March 6, 2023.
6. US. Department of Agriculture, NRCS, *Urban Hydrology for Small Watersheds TR-55*. June 1986
7. U.S. Army Corps of Engineers, Hydrologic Engineering Center's River Analysis System (HEC-RAS) 2D User's Manual, USACE, Davis, CA.
8. Natural Resources Conservation Service, National Engineering Handbook, Part 630 Hydrology, Chapter 9: Hydrologic Soil-Cover Complexes, issued July 2004.
9. Natural Resources Conservation Service, National Engineering Handbook, Part 630 Hydrology, Chapter 10: Estimation of Direct Runoff from Storm Rainfall, issued July 2004.
10. Federal Emergency Management Agency, *Guidance for Flood Risk Analysis and Mapping: Levees*, Levees, Guidance Document No. 95, November 2023.

Appendix A – Sensitivity Testing

A total of 9 sensitivity scenarios were analyzed to assess the Turkey Creek and Brush Creek watershed sensitivity to the basin parameters. The scenarios are summarized below.

Table A 1: Sensitivity Test Scenarios

	Scenario Name	Parameter Updates
A	High Mann	Update base Manning's n values to high end of recommended values in HEC-RAS Manual (No modification to base CN)
B	Low Mann	Update base Manning's n values to low end of recommended values in HEC-RAS Manual (No modification to base CN)
C	High Mann - Upland Refine	Upland calibration region added with high end of TR-55 sheetflow values (Base Manning's n in floodplain, no modification to base CN)
D	Low Mann - Upland Refine	Upland calibration region added with low end of TR-55 sheetflow values (Base Manning's n in floodplain, no modification to base CN)
E	Low CN	Base CN reduced values by 3 (No changes to Impervious CN 98 or 100, no modification to base Manning's n)
F	High CN	Base CN increased values by 3 (No changes to Impervious CN 98 or 100, no modification to base Manning's n)
G	Ia = 0.1	Abstraction Ratio set to 0.1, base modeling parameters
H	Ia = 0.05	Abstraction Ratio set to 0.05, base modeling parameters
I	Hotstart	Initial conditions added based on the end of simulation results from the 10-pct 12-hr run.

Specific modeling parameter and the adjustments applied for each scenario are summarized in the tables below. The impacts of each scenario were compared at the StormWatch gages where observed data was recorded. These gages only record an observed stage. Additionally, the StormWatch gages are primarily on the mainstem of Turkey Creek. An additional comparison location along Tributary F was included to represent a smaller drainage area. Comparison can be seen in the Figures below.

Table A 2: Manning's n Sensitivity Scenario Adjustments

	Base Manning's n	High Manning's n (Scenario A)	Low Manning's n (Scenario B)	High Manning's n Upland Refine (Scenario C)	Low Manning's n Upland Refine (Scenario D)
Trees	0.085	0.16	0.08	0.8	0.4
Natural Channel	0.035	0.05	0.025	NA	NA
Concrete Channel	0.015	0.02	0.011	NA	NA
Pavement	0.02	0.022	0.011	0.011	0.011
Grass	0.03	0.05	0.025	0.4	0.15

Buildings	1	1	0.5	1	1
Barren Land	0.025	0.03	0.023	0.05	0.05
Open Water	0.025	0.05	0.025	0.025	0.025

Table A 3: CN Sensitivity Scenario Adjustments

	A	B	C	D
Open Water	100	100	100	100
Barren Land	-	85	90	93
		82	**87**	**90**
		88	**93**	**96**
Buildings	98	98	98	98
Grass	-	61	74	80
		58	**71**	**77**
		64	**77**	**83**
Pavement	98	98	98	98
Trees	-	60	73	79
		57	**70**	**76**
		63	**76**	**82**
Concrete Channel	98	98	98	98
Natural Channel	-	61	74	80
		58	**71**	**77**
		64	**77**	**83**

Low CN (Scenario E), **High CN (Scenario F)**

Table A 4: Abstraction Ratio Sensitivity Scenario Adjustments

Base Abstraction Ratio	Scenario G	Scenario H
0.2	0.1	0.05

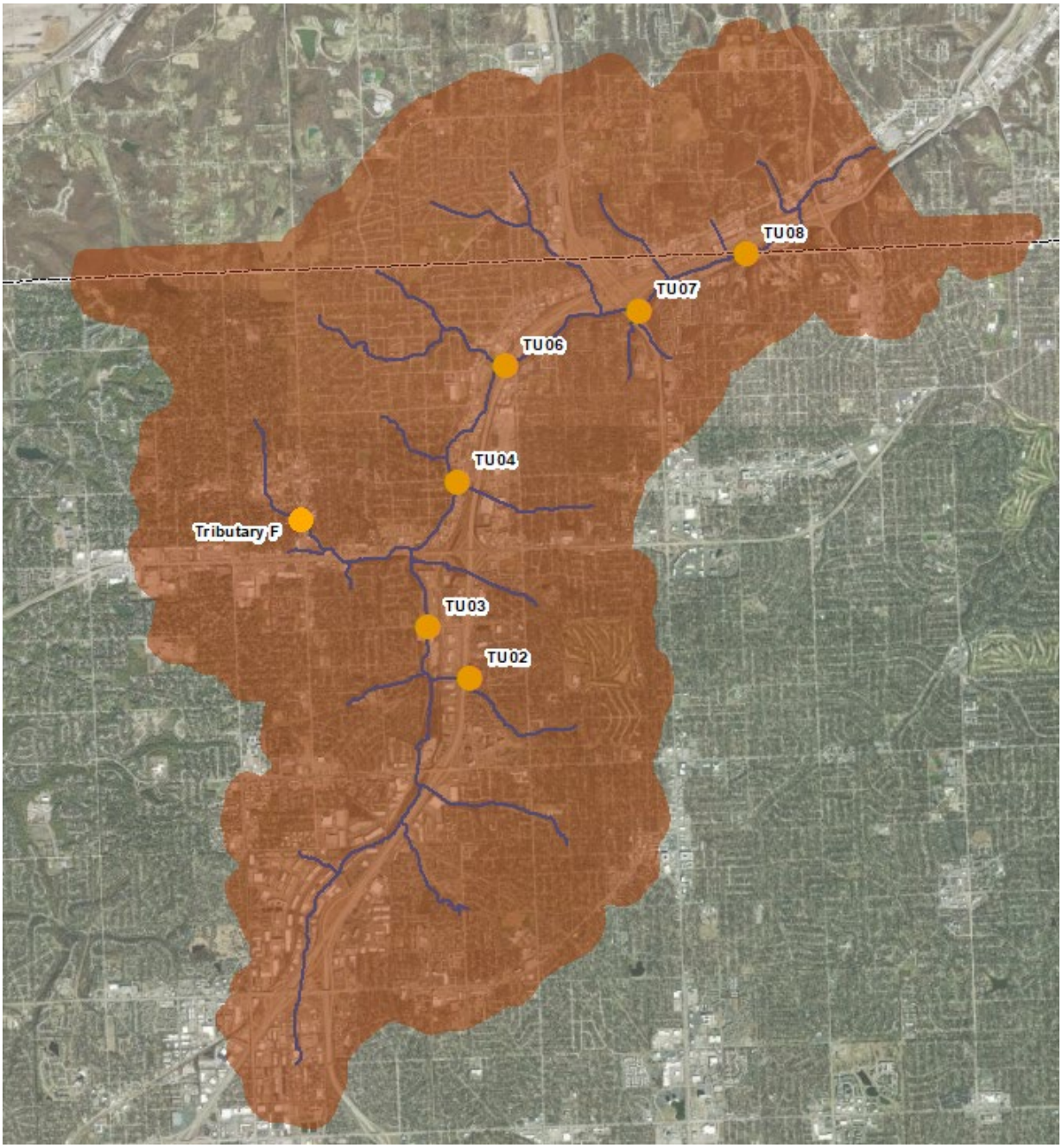


Figure A 1: Turkey Creek Sensitivity Comparison Locations

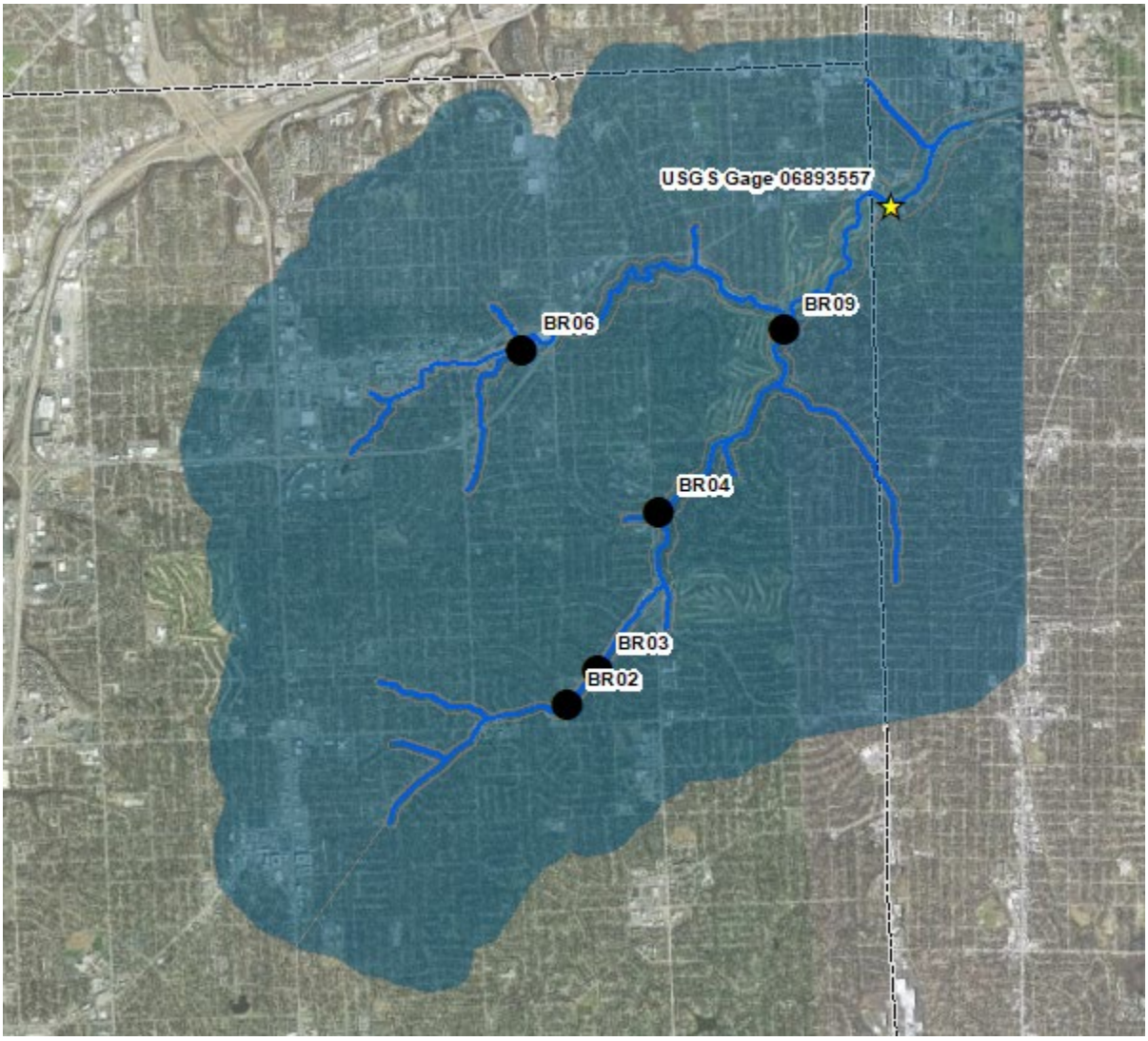


Figure A 2: Brush Creek Sensitivity Comparison Locations

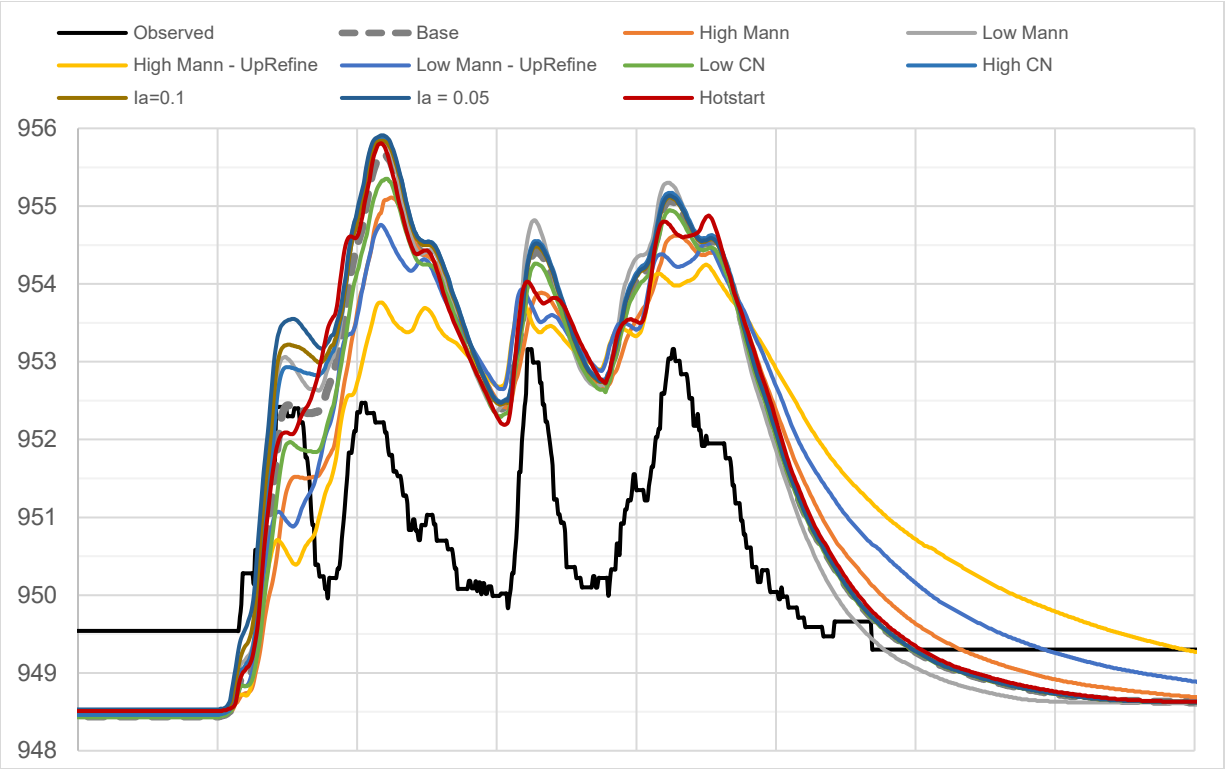


Figure A 3: Stage (ft) Sensitivity Results at TU02 for August 2017 Rainfall Event

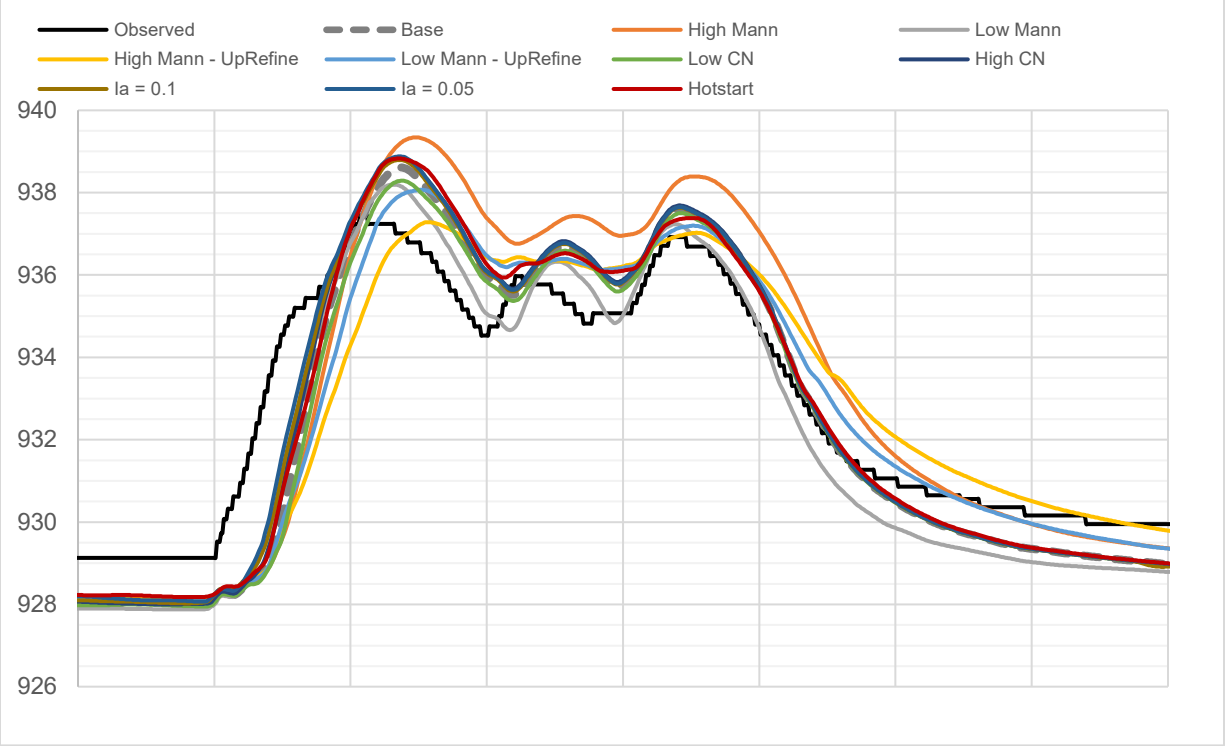


Figure A 4: Stage (ft) Sensitivity Results at TU03 for August 2017 Rainfall Event

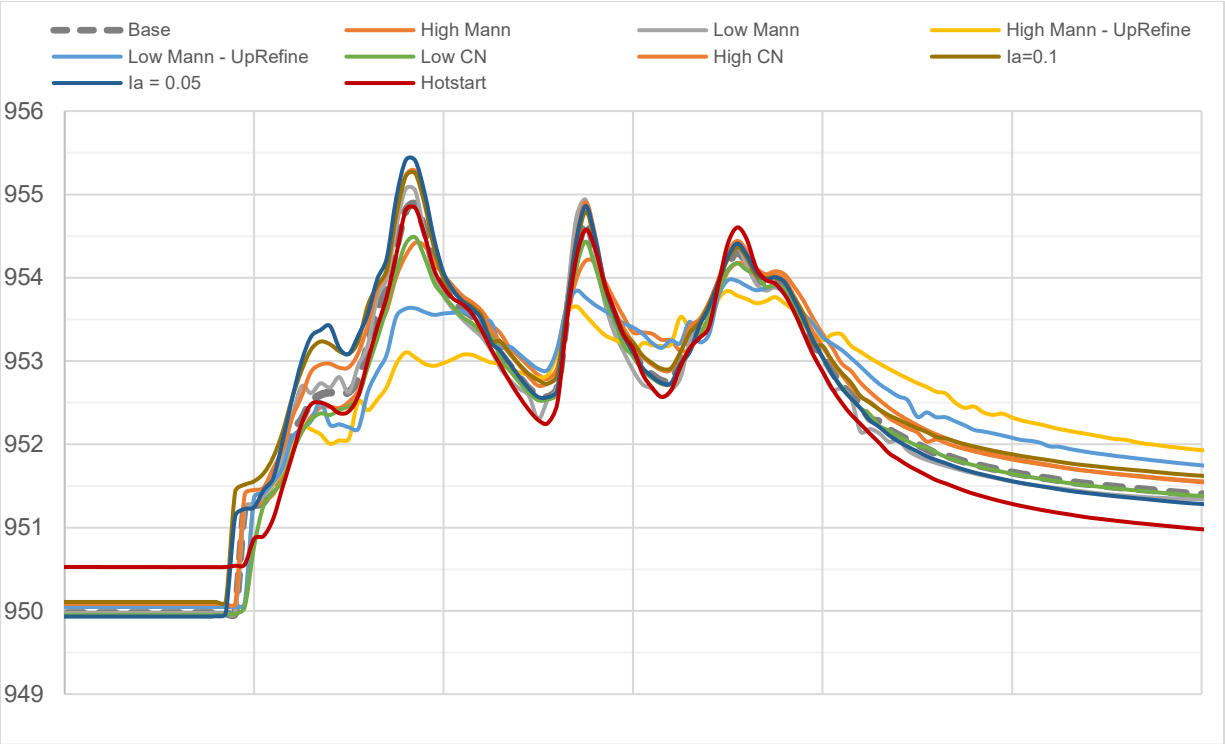


Figure A 5: Stage (ft) Sensitivity Results at Tributary F for August 2017 Rainfall Event

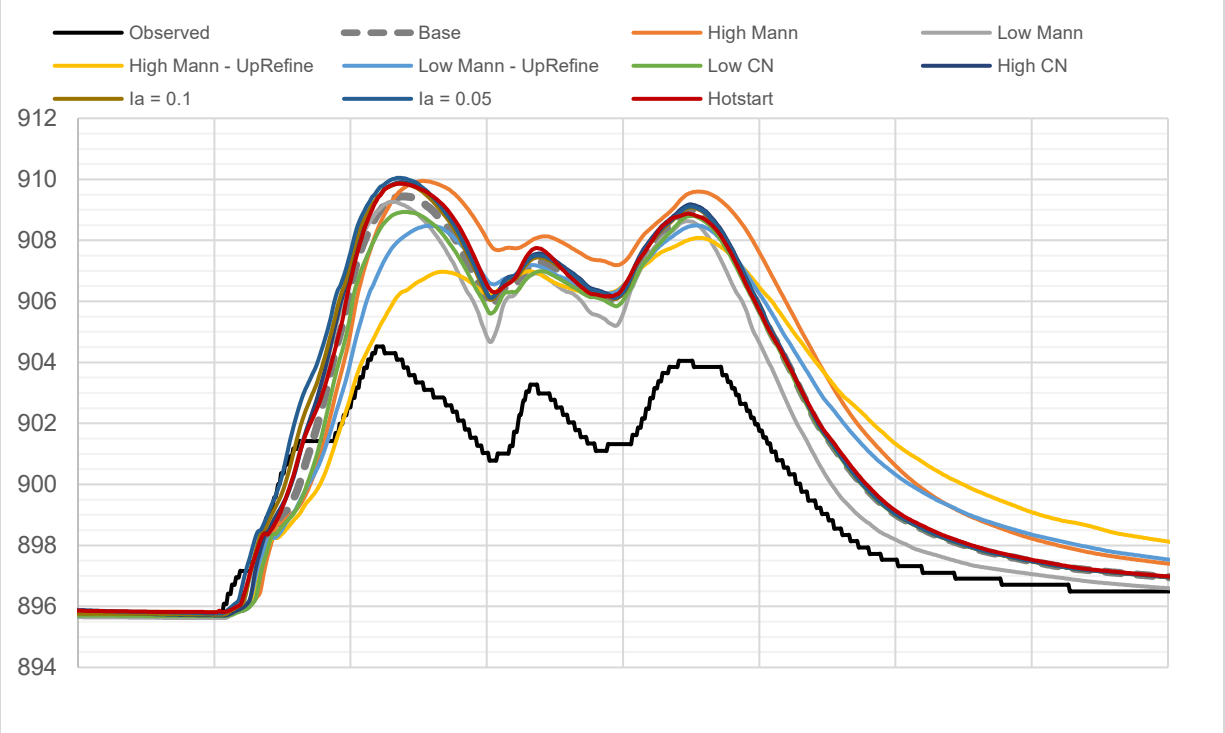


Figure A 6: Stage (ft) Sensitivity Results at TU04 for August 2017 Rainfall Event

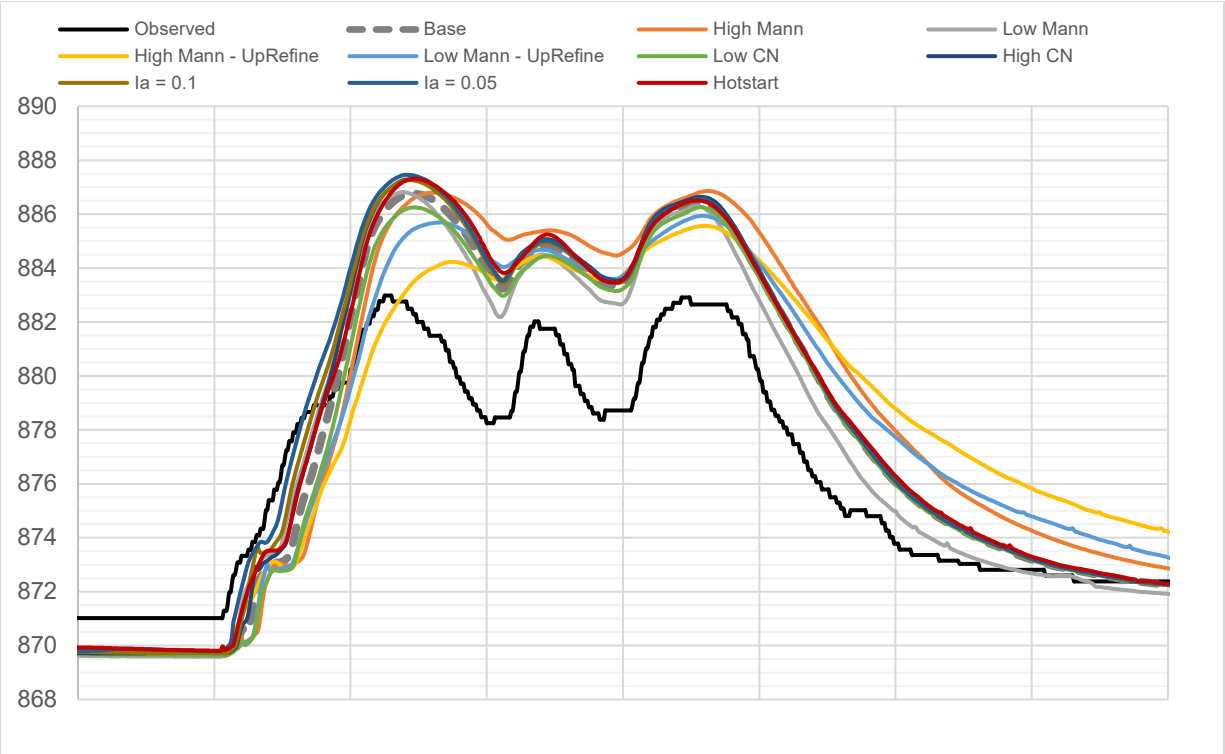


Figure A 7: Stage (ft) Sensitivity Results at TU06 for August 2017 Rainfall Event

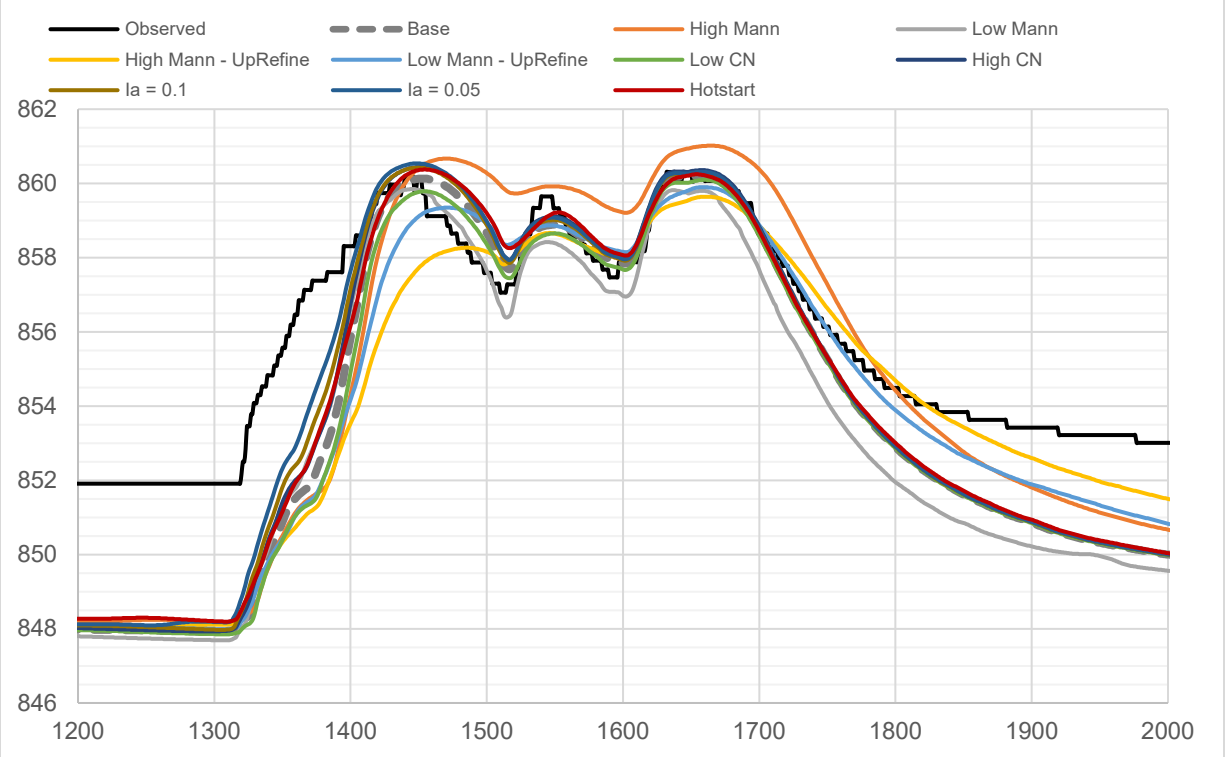


Figure A 8: Stage (ft) Sensitivity Results at TU07 for August 2017 Rainfall Event

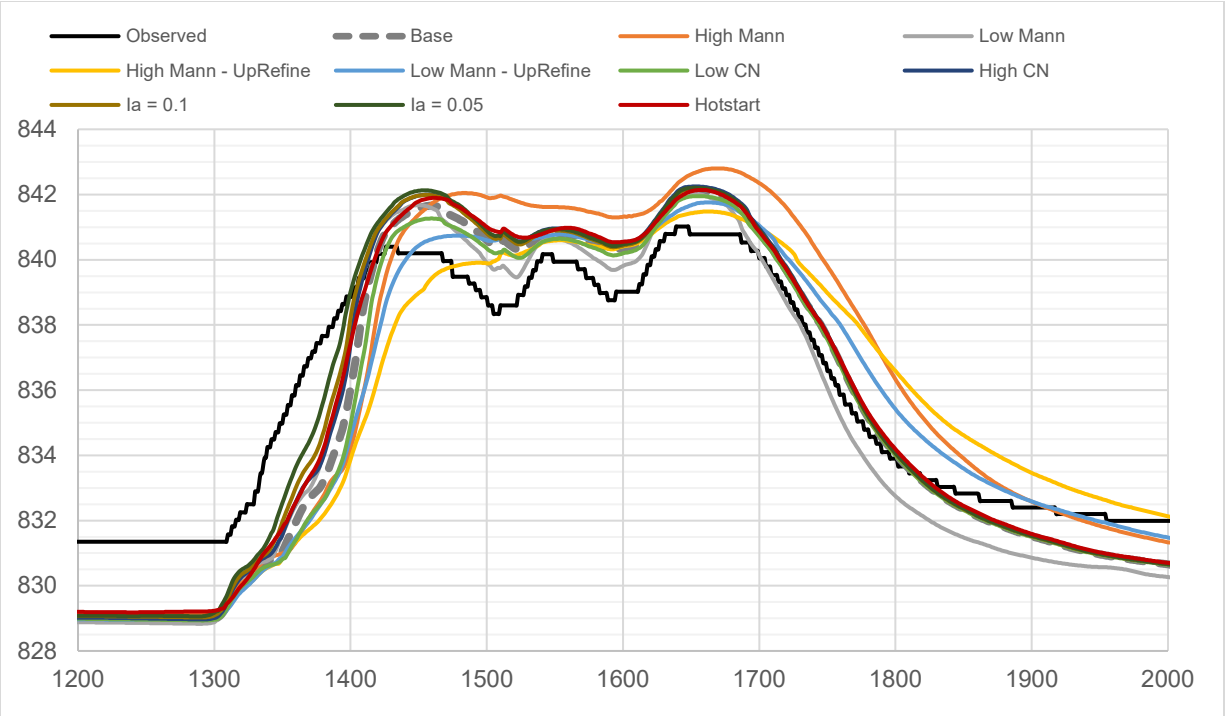


Figure A 9: Stage (ft) Sensitivity Results at TU08 for August 2017 Rainfall Event

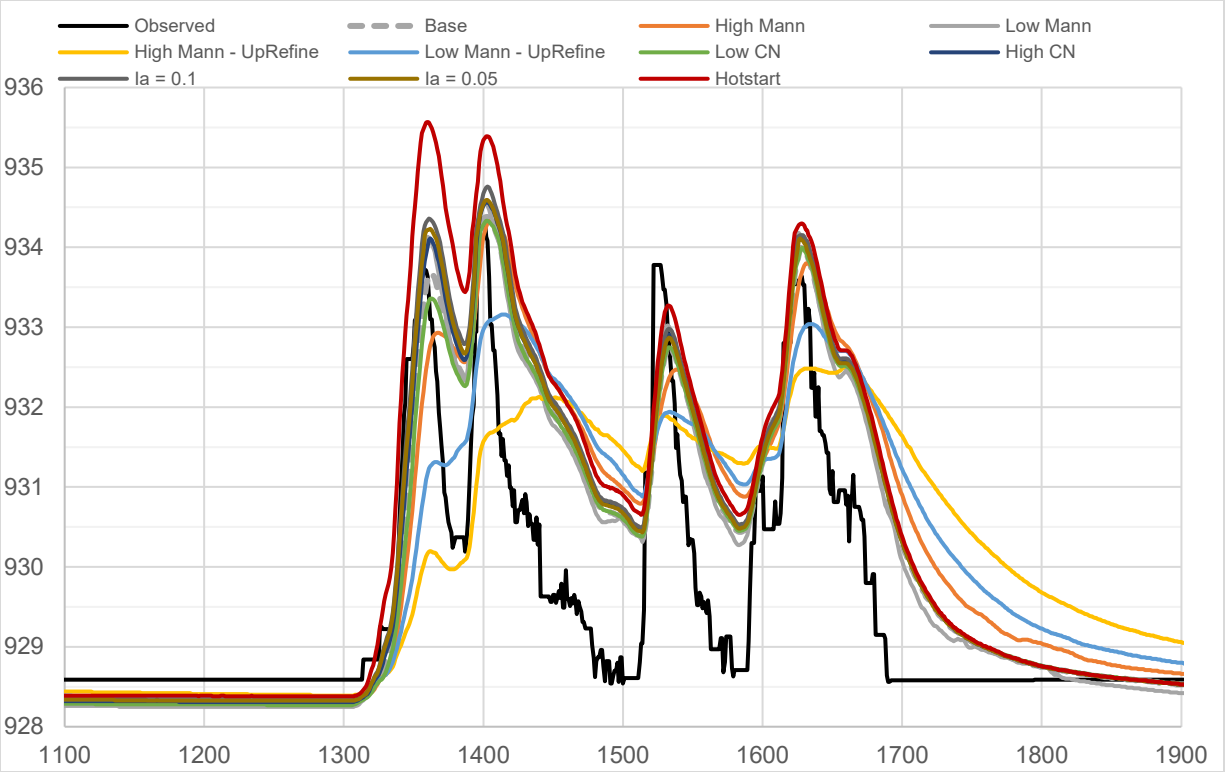


Figure A 10: Stage (ft) Sensitivity Results at BR02 for August 2017 Rainfall Event

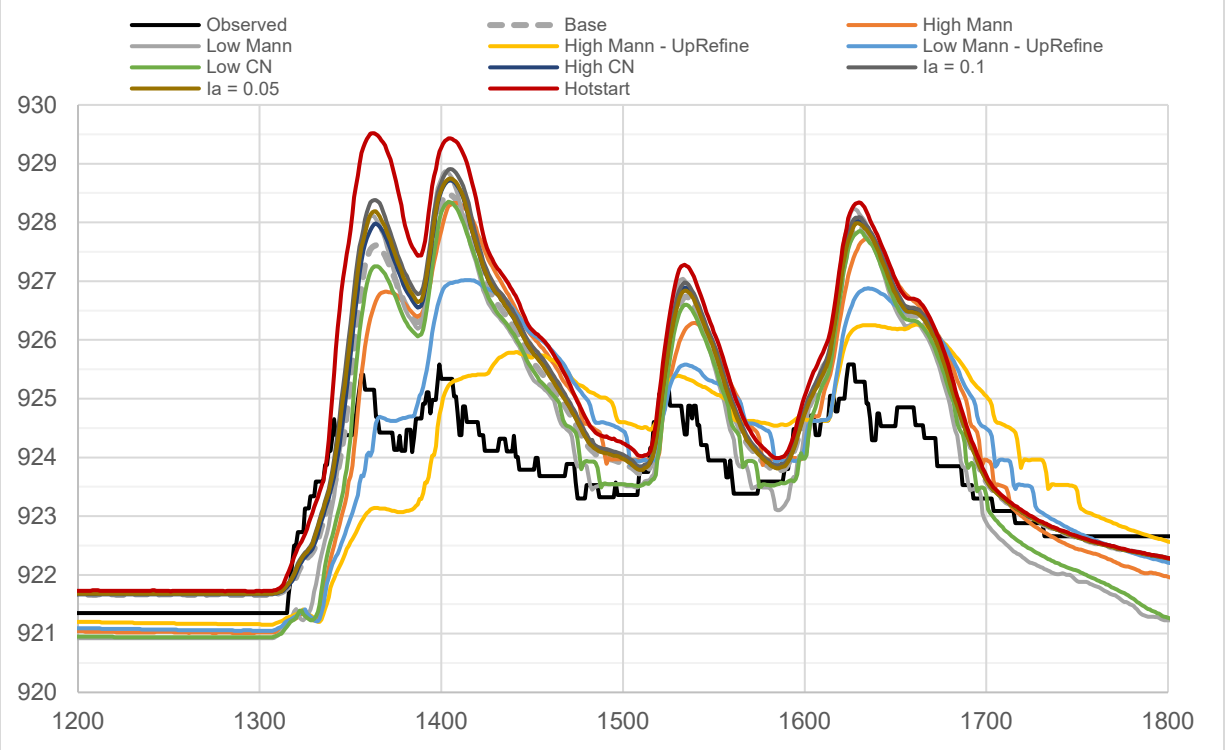


Figure A 11: Stage (ft) Sensitivity Results at BR03 for August 2017 Rainfall Event

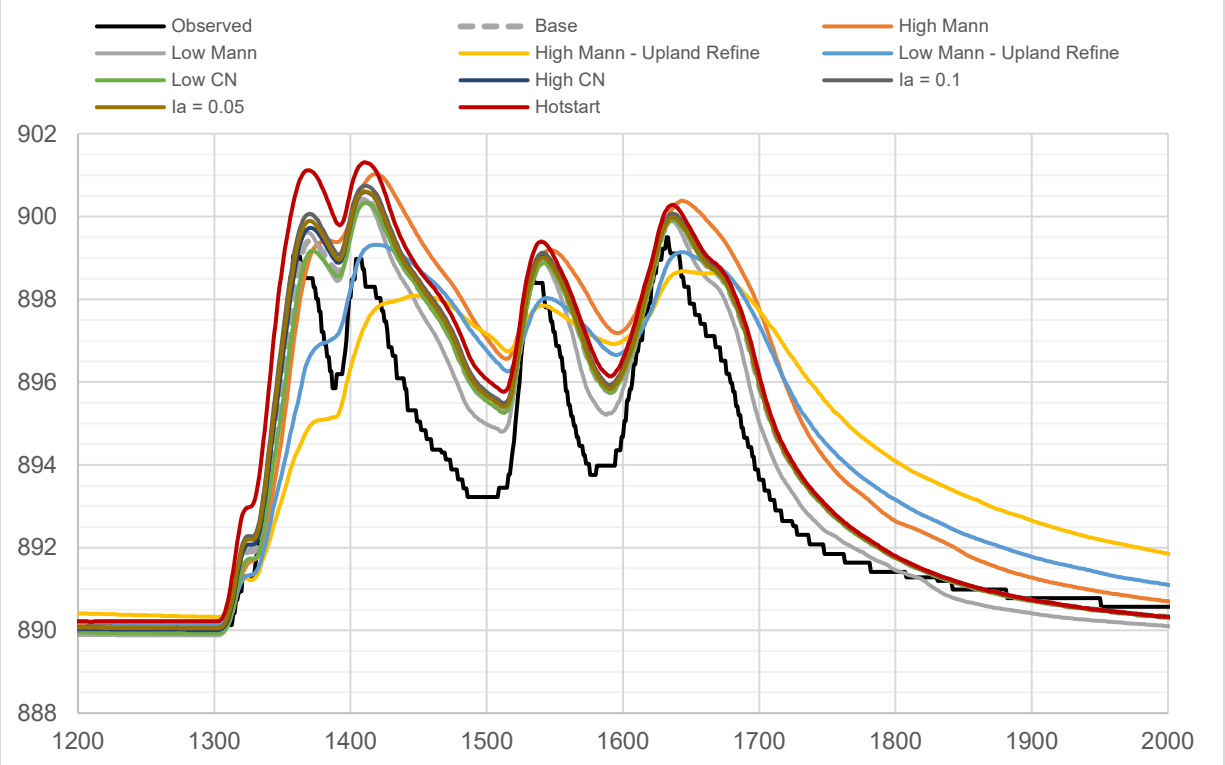


Figure A 12: Stage (ft) Sensitivity Results at BR04 for August 2017 Rainfall Event

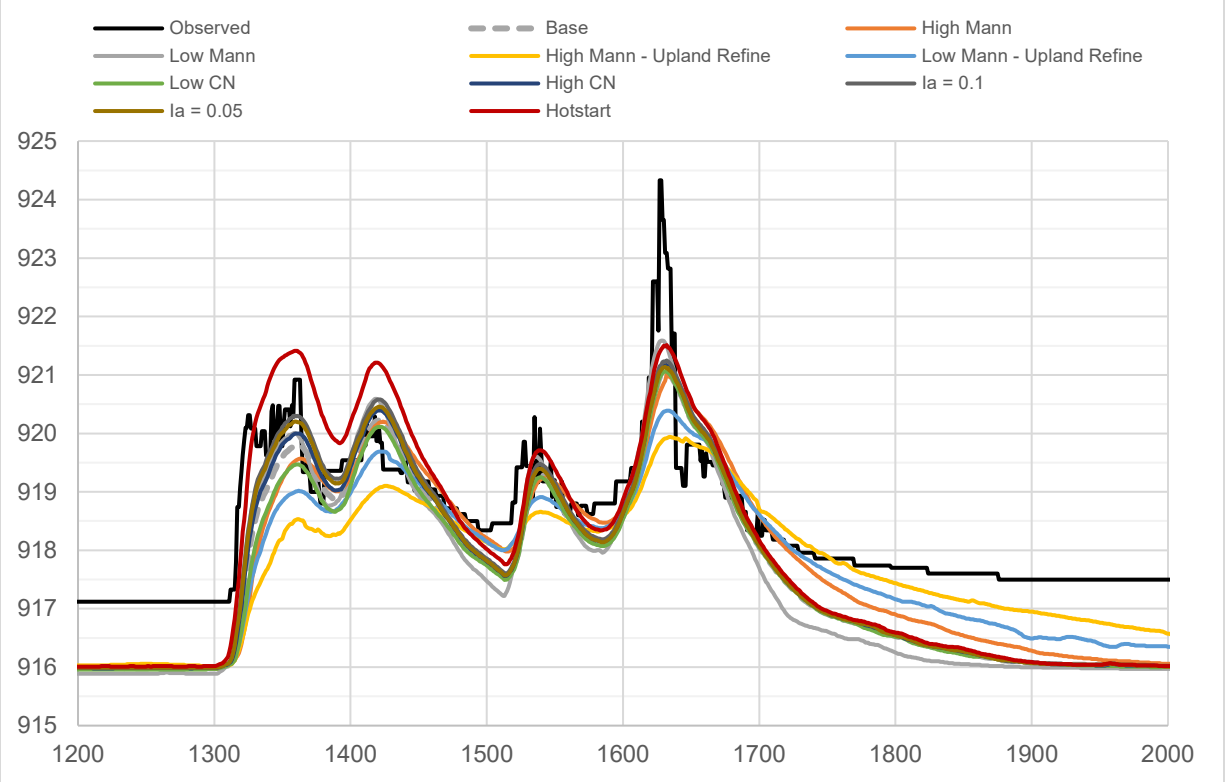


Figure A 13: Stage (ft) Sensitivity Results at BR06 for August 2017 Rainfall Event

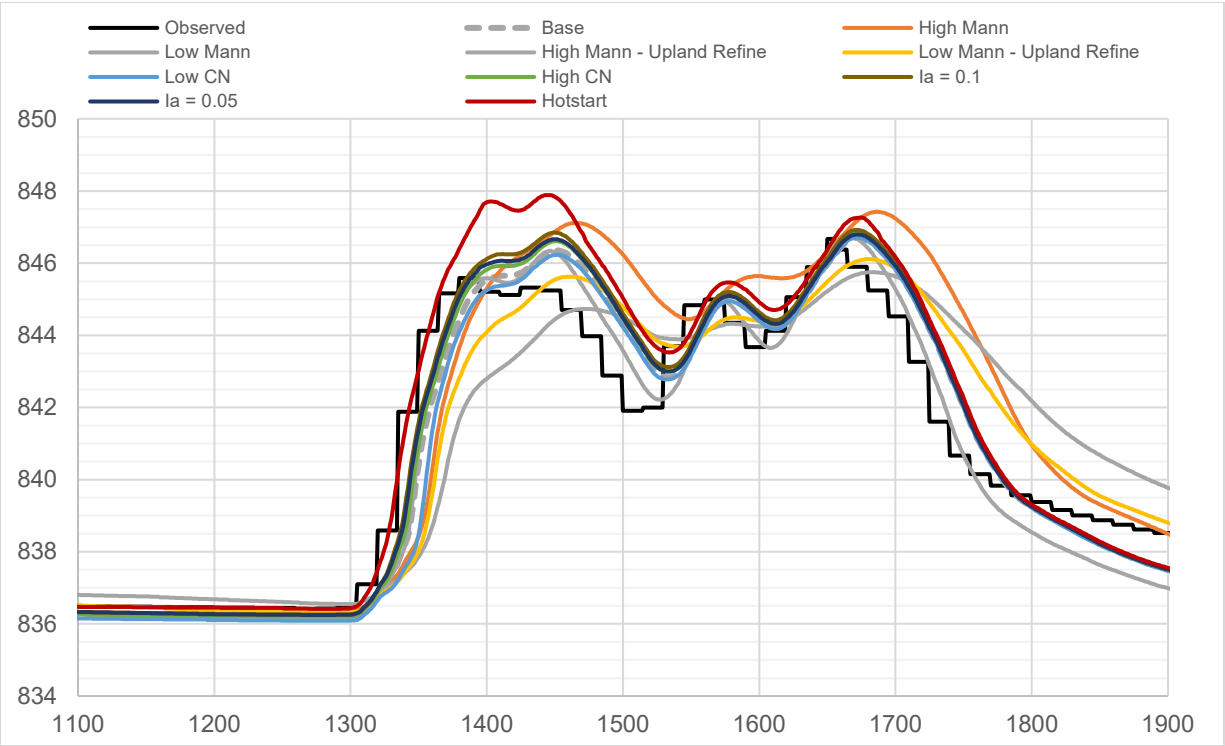


Figure A 14: Stage (ft) Sensitivity Results at USGS Gage for August 2017 Rainfall Event

Appendix B – Base & Calibrated Model Results 2017 Event

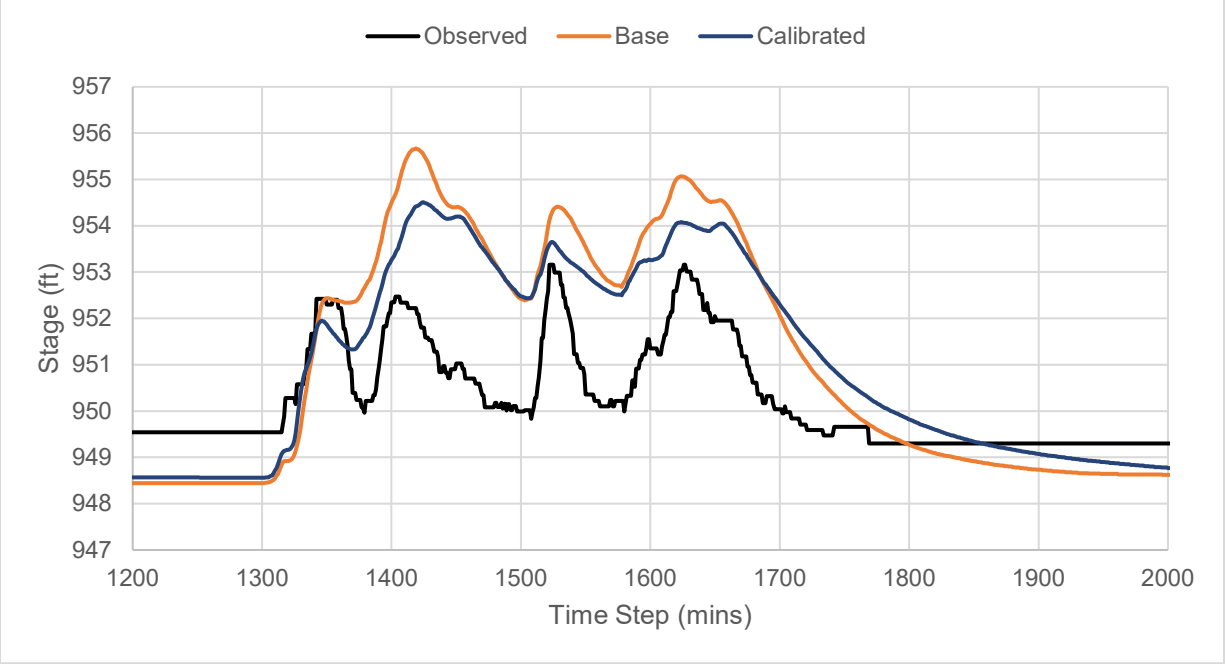


Figure B 1: Base & Calibrated Model Results at TU02-69th St @ Turkey Creek

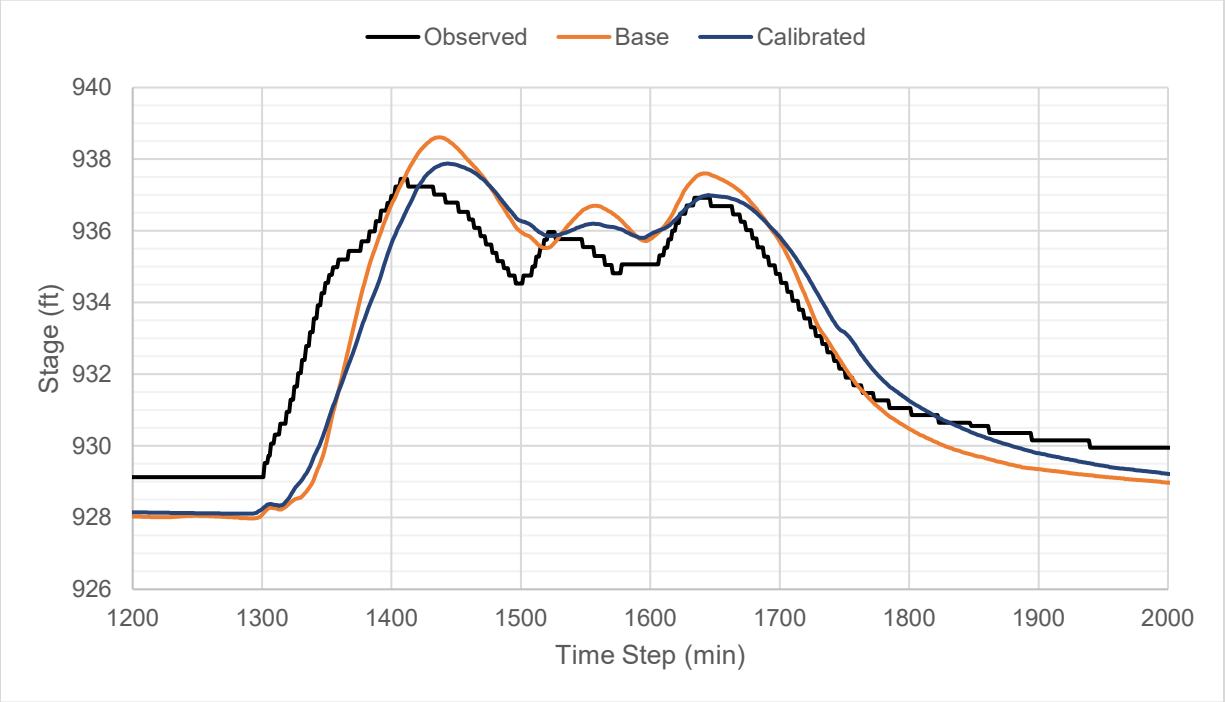


Figure B 2: Base & Calibrated Model Results at TU03-67th St @ Turkey Creek

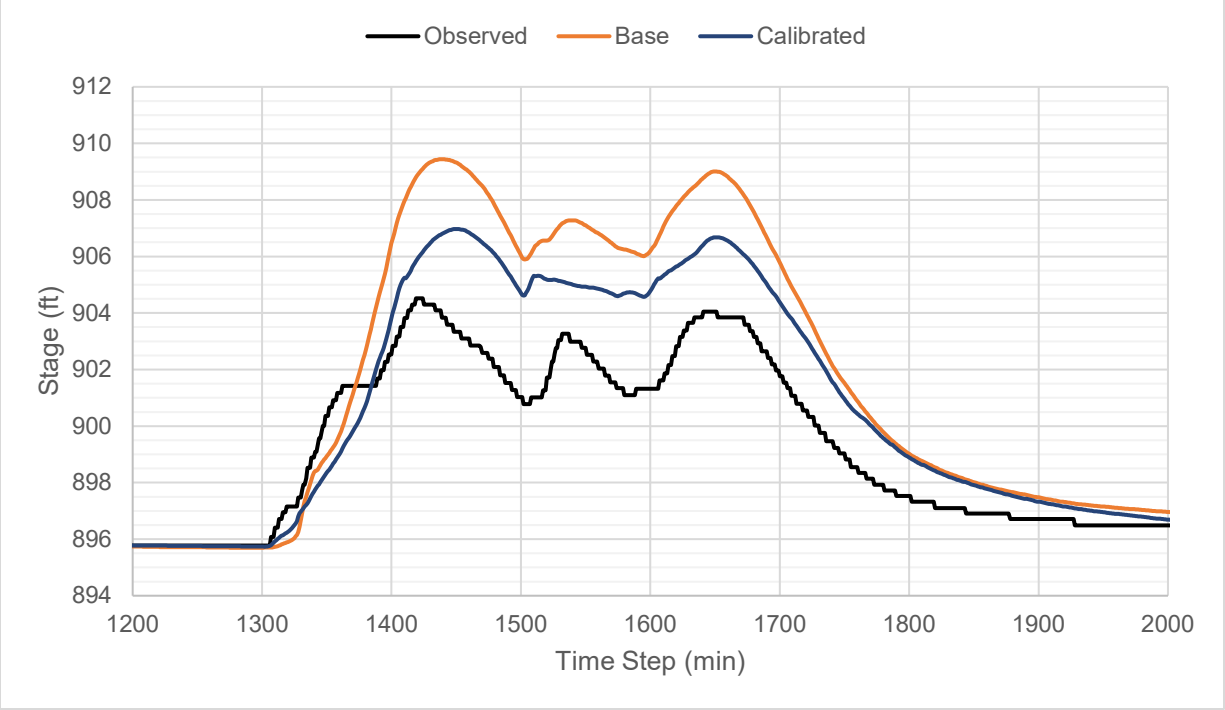


Figure B 3: Base & Calibrated Model Results at TU04-Johnson Dr @ Turkey Creek

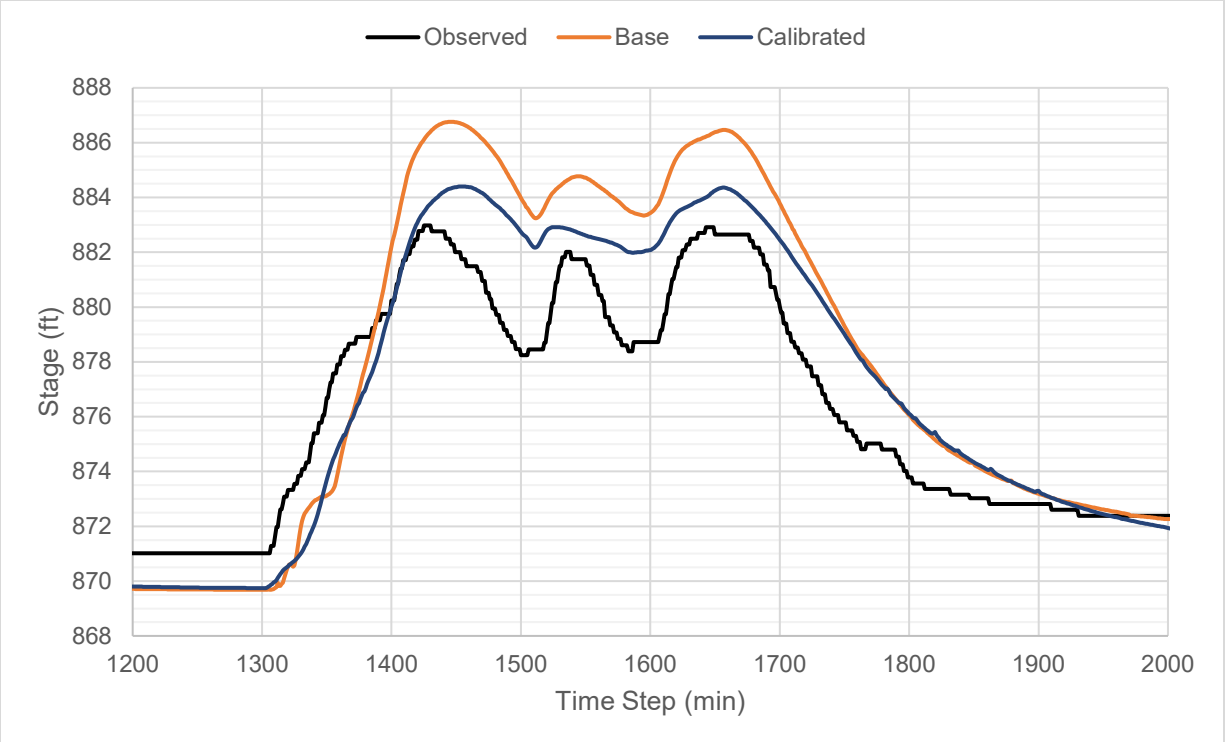


Figure B 4: Base & Calibrated Model Results at TU-06Antioch Rd On-Ramp @ Turkey Creek

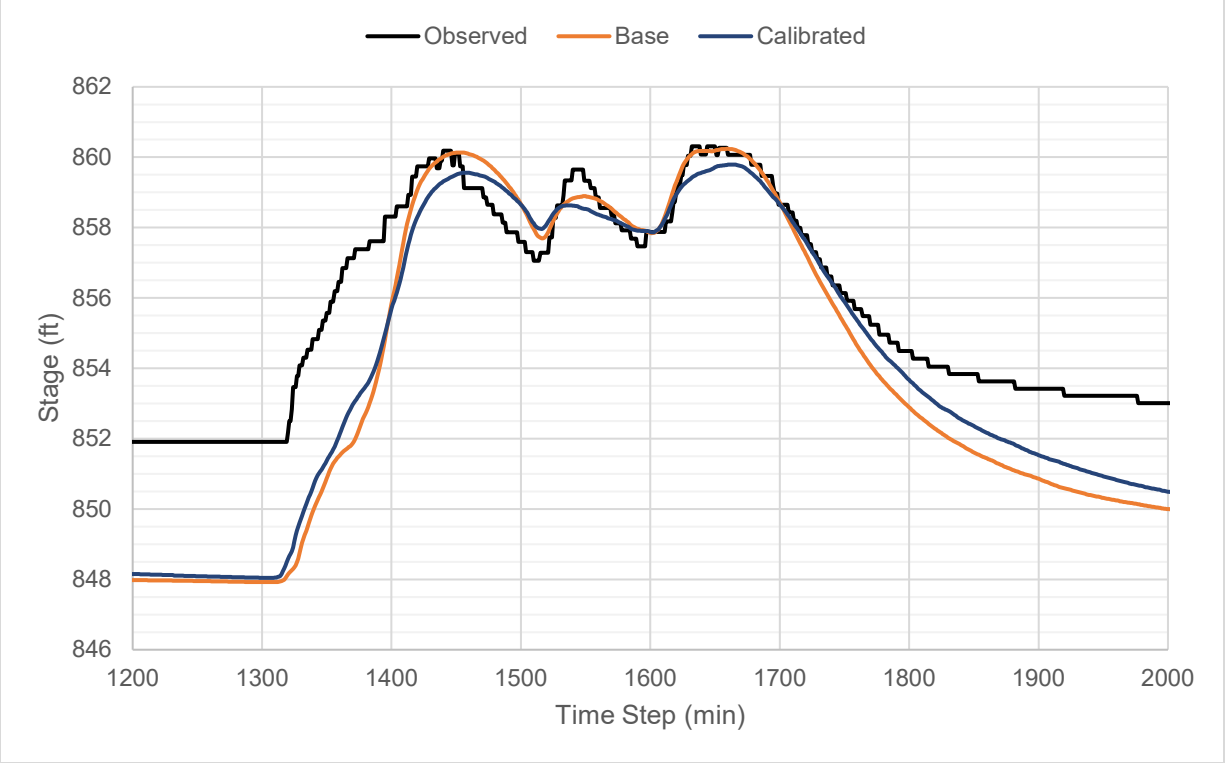


Figure B 5: Base & Calibrated Model Results at TU07-Metcalf Ave @ Turkey Creek

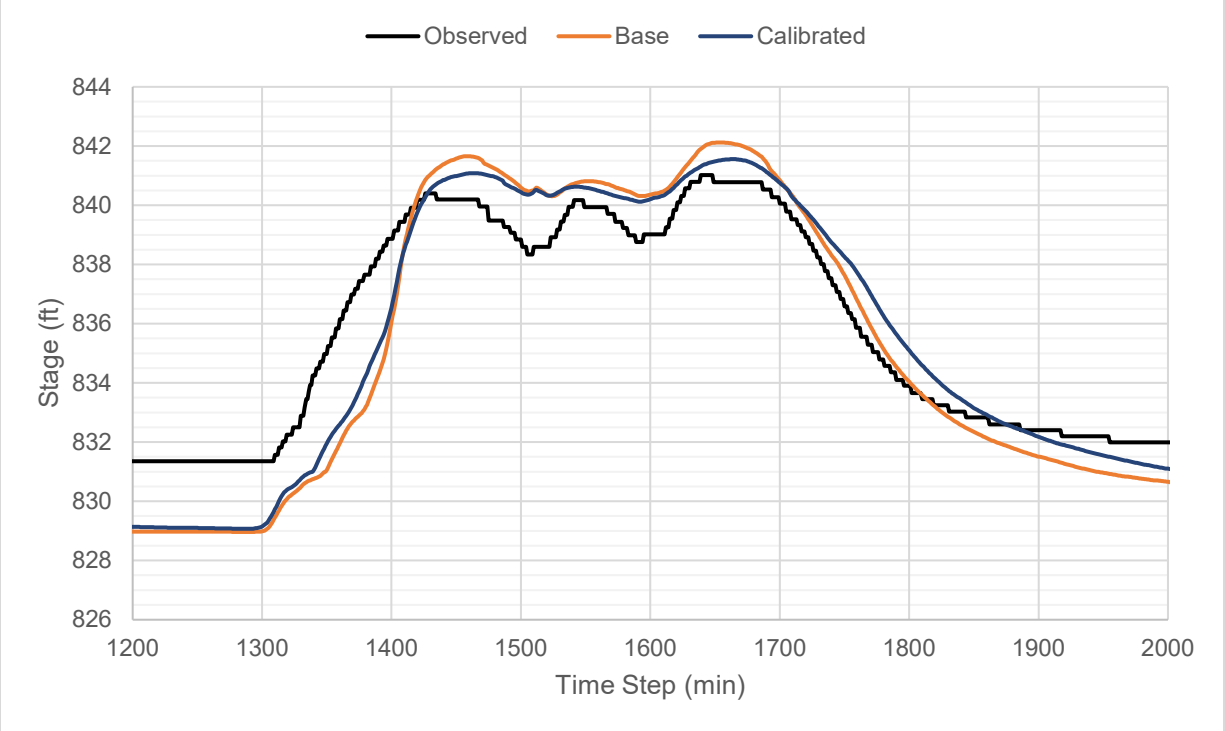


Figure B 6: Base & Calibrated Model Results at TY08-Lamar Ave @ Turkey Creek

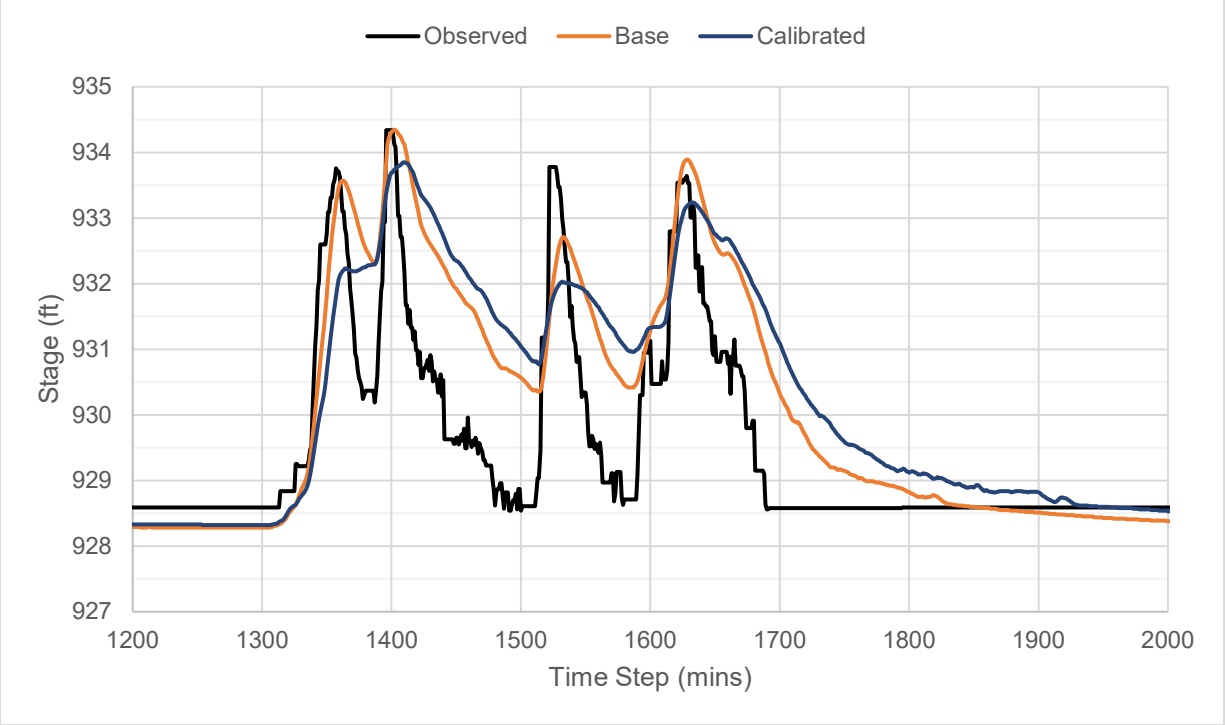


Figure B 7: Base & Calibrated Model Results at BR02-Porter Park @ Brush Creek

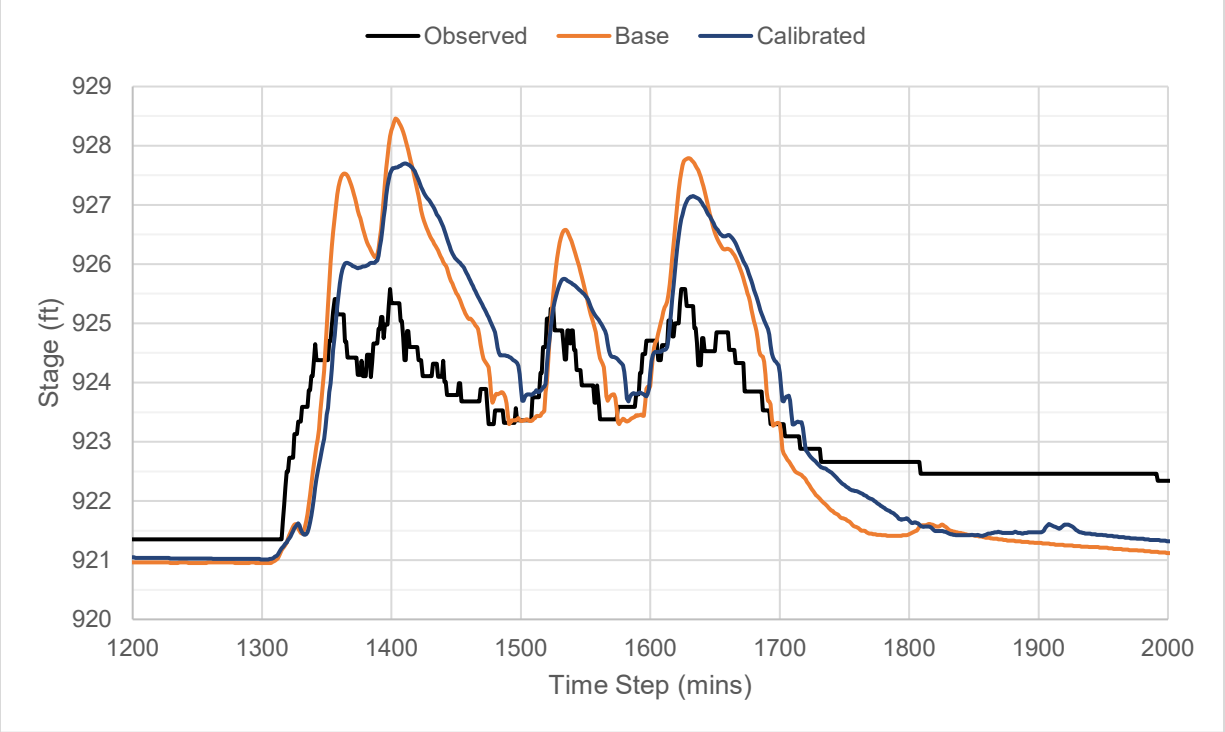


Figure B 8: Base & Calibrated Model Results at BR03-74th ST @ Brush Creek

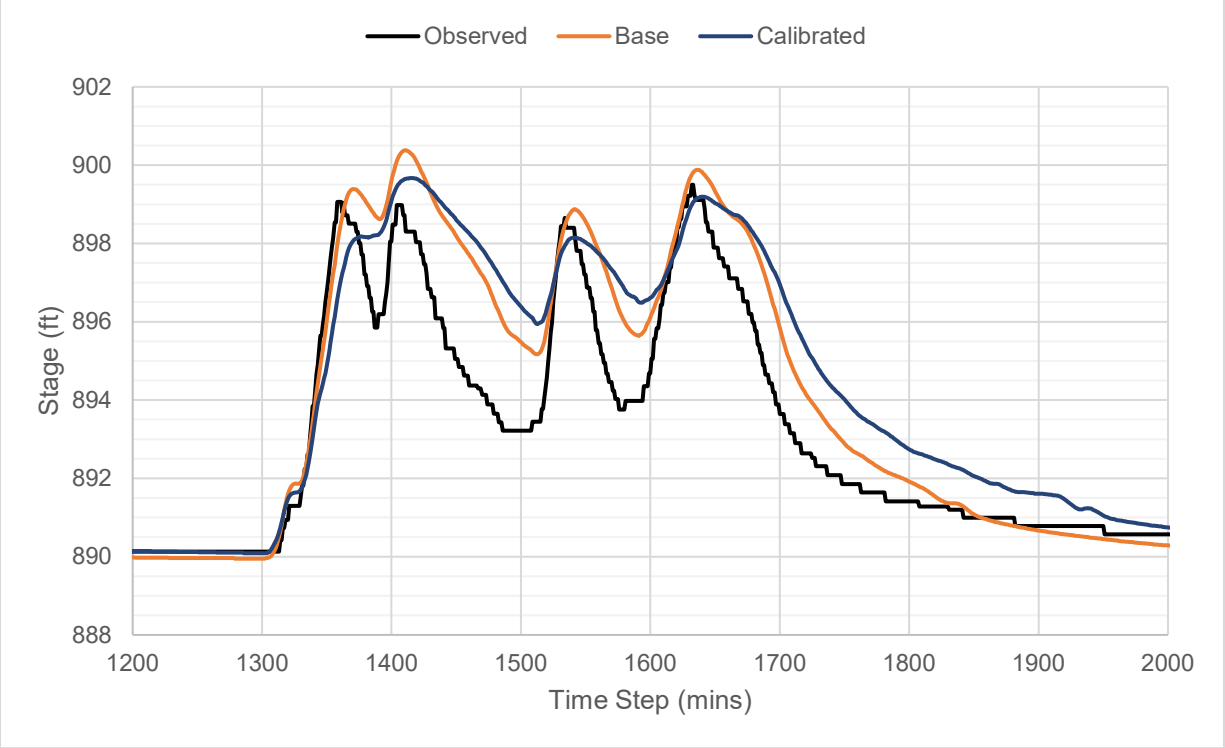


Figure B 9: Base & Calibrated Model Results at BR04-66th St & Mission Rd Master @ Brush Creek

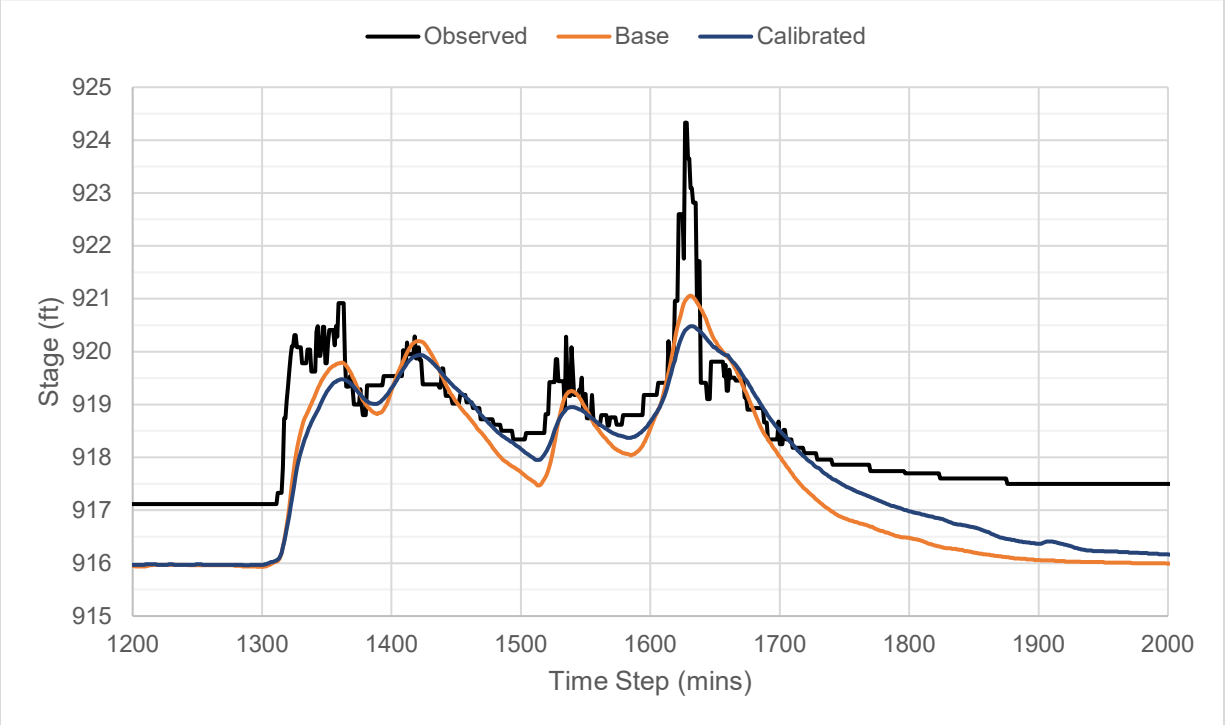


Figure B 10: Base & Calibrated Model Results at BR06-Martway @ Rock Creek

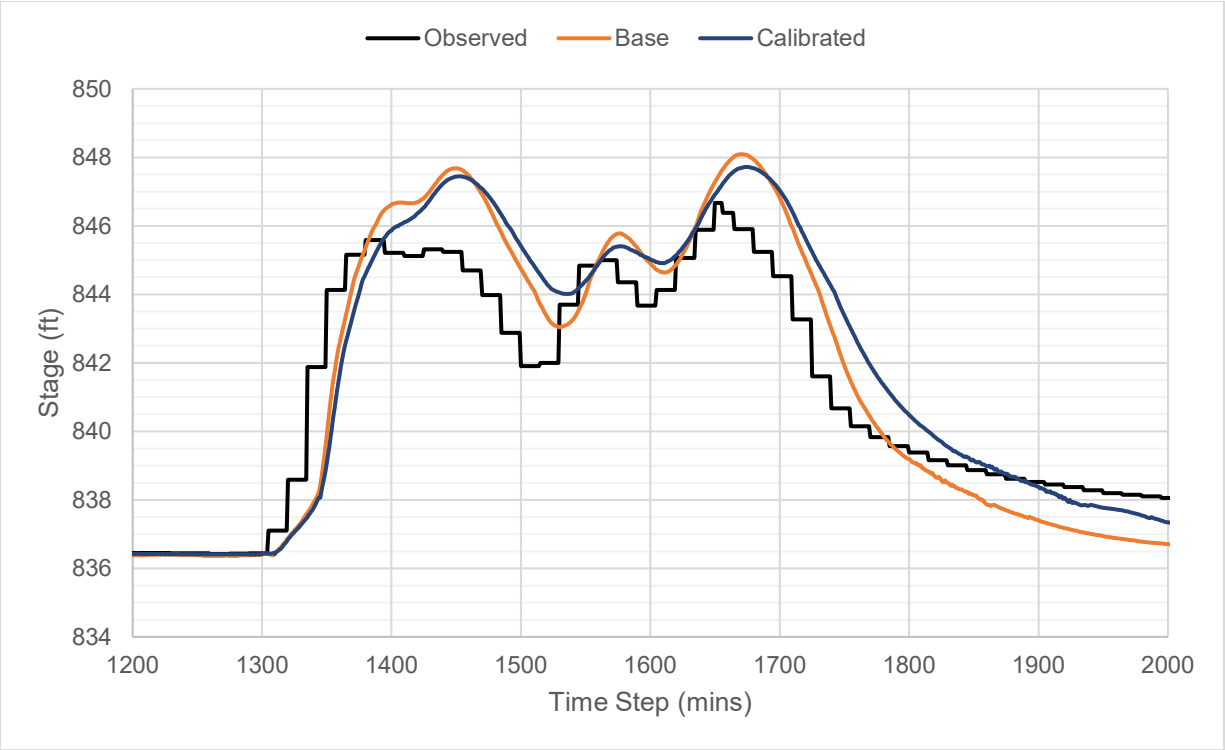


Figure B 11: Base & Calibrated Model Results at USGS-06893557 Brush Creek at Ward Parkway

Appendix C – Base & Calibrated Model Results 2020 Event

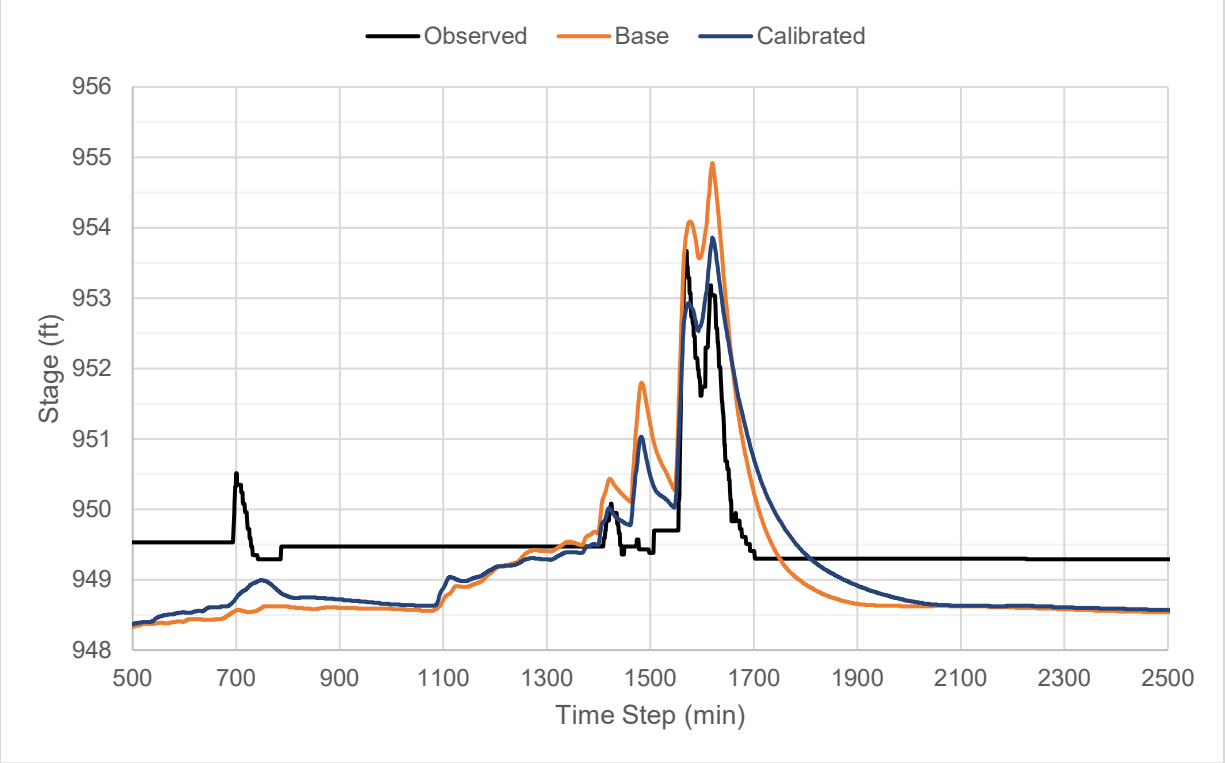


Figure C 1: Base & Calibrated Model Results at TU02-69th St @ Turkey Creek

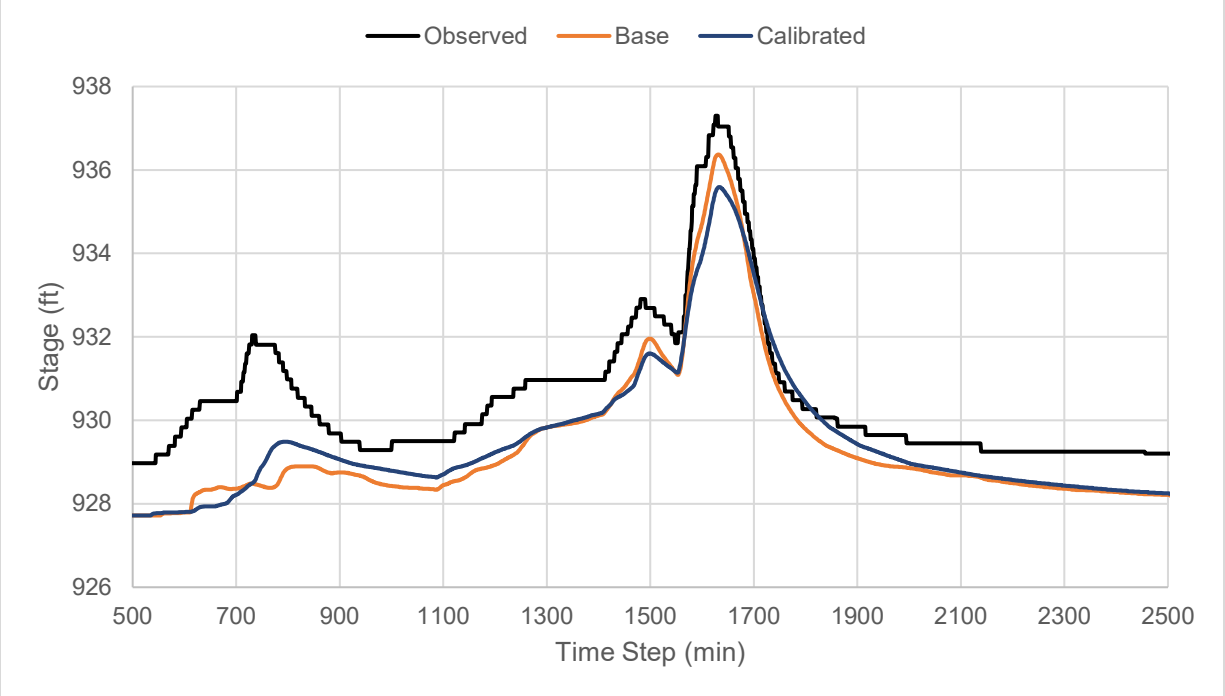


Figure C 2: Base & Calibrated Model Results at TU03-67th St @ Turkey Creek

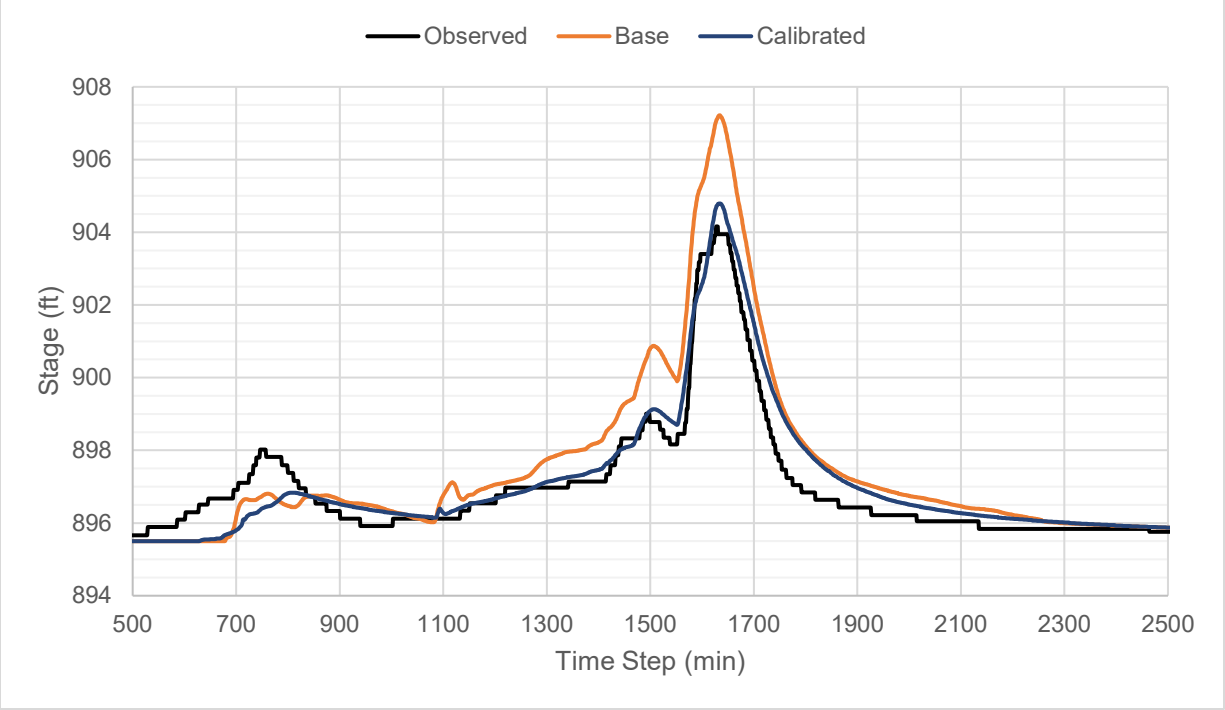


Figure C 3: Base & Calibrated Model Results at TU04-Johnson Dr @ Turkey Creek

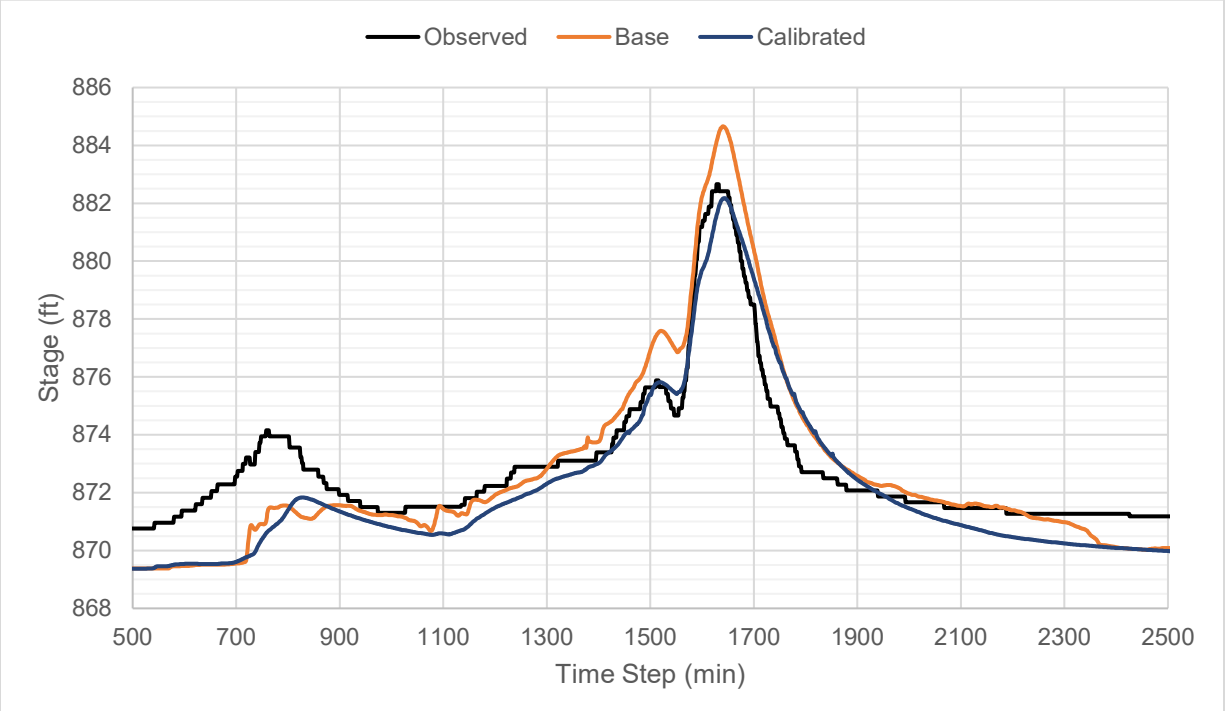


Figure C 4: Base & Calibrated Model Results at TU-06Antioch Rd On-Ramp @ Turkey Creek

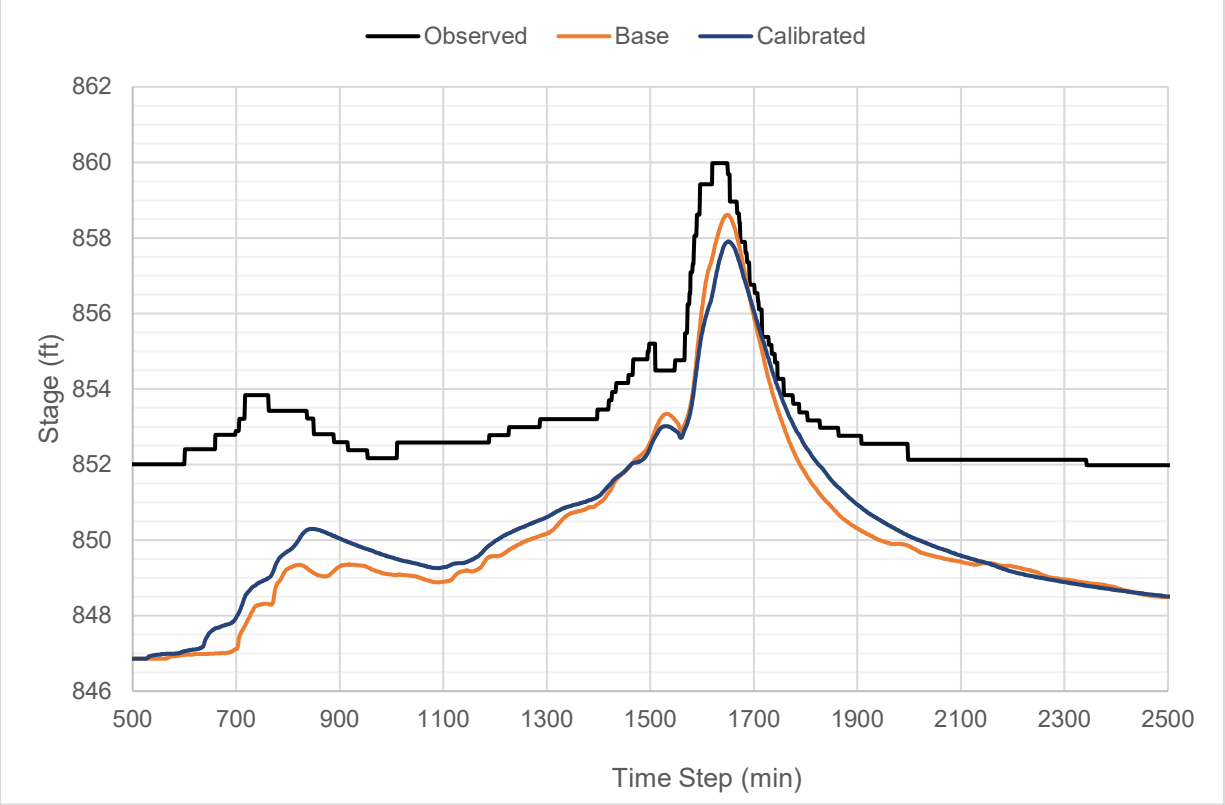


Figure C 5: Base & Calibrated Model Results at TU07-Metcalf Ave @ Turkey Creek

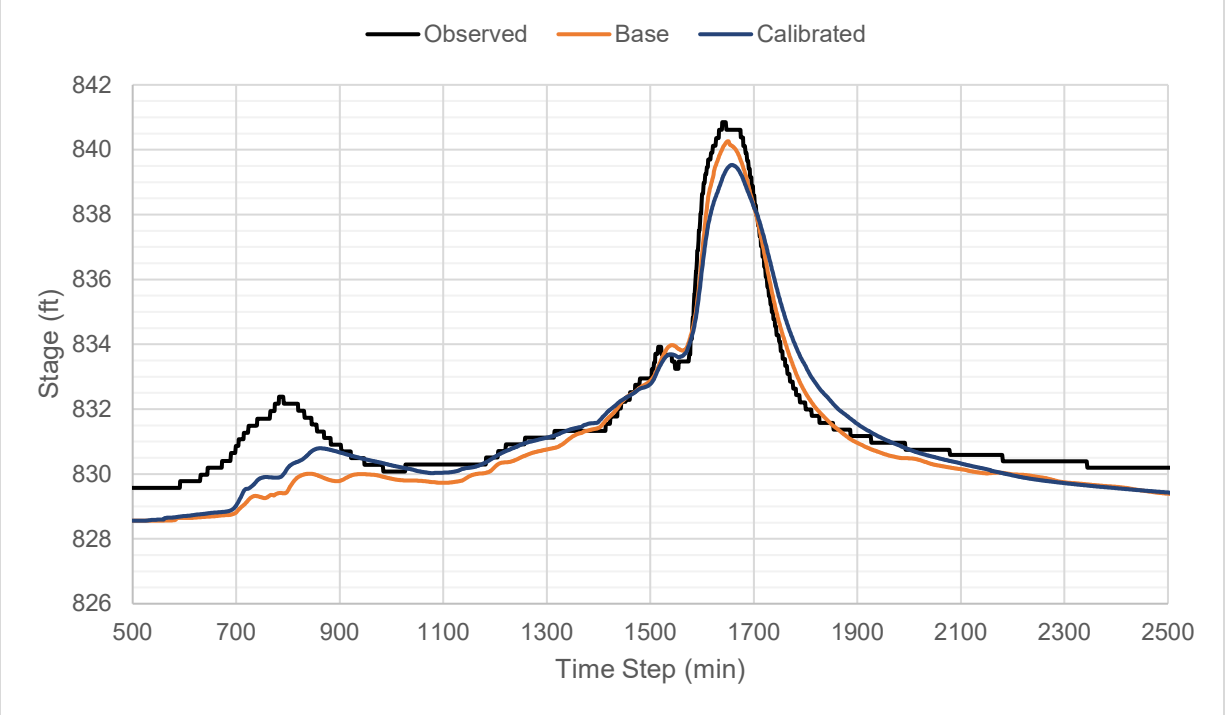


Figure C 6: Base & Calibrated Model Results at TU08-Lamar Ave @ Turkey Creek

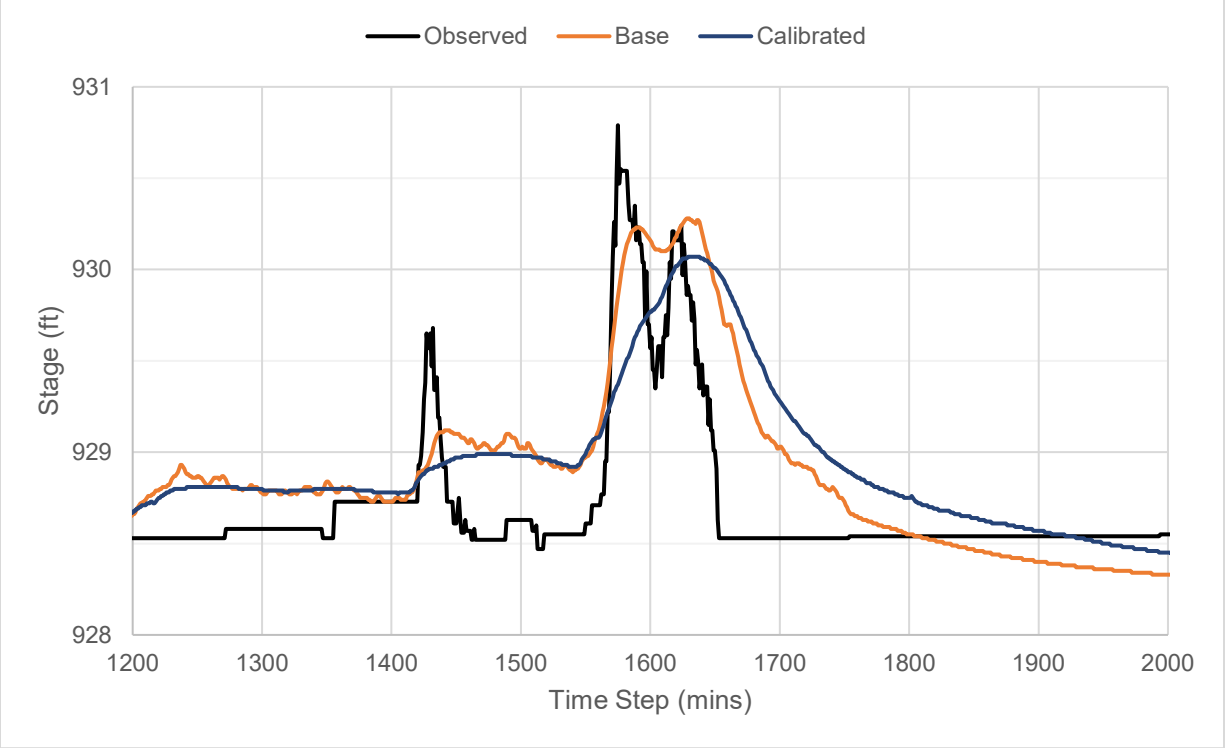


Figure C 7: Base & Calibrated Model Results at BR02-Porter Park @ Brush Creek

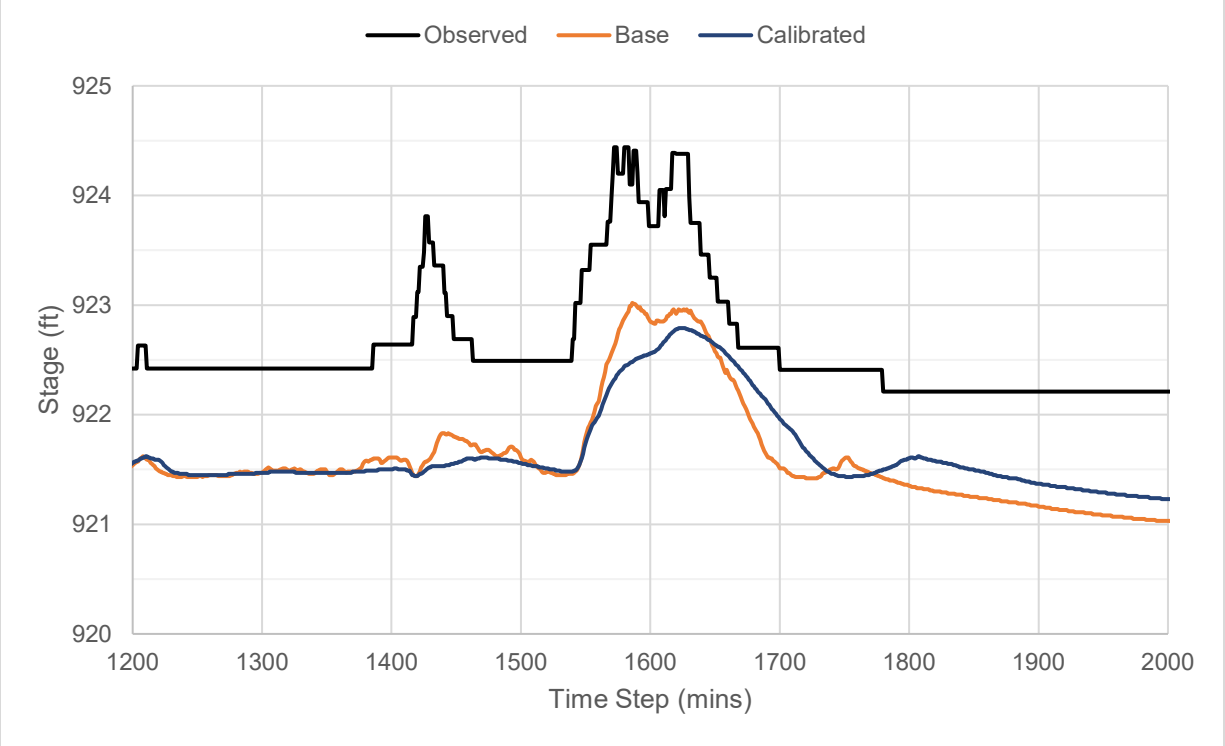


Figure C 8: Base & Calibrated Model Results at BR03-74th St @ Brush Creek

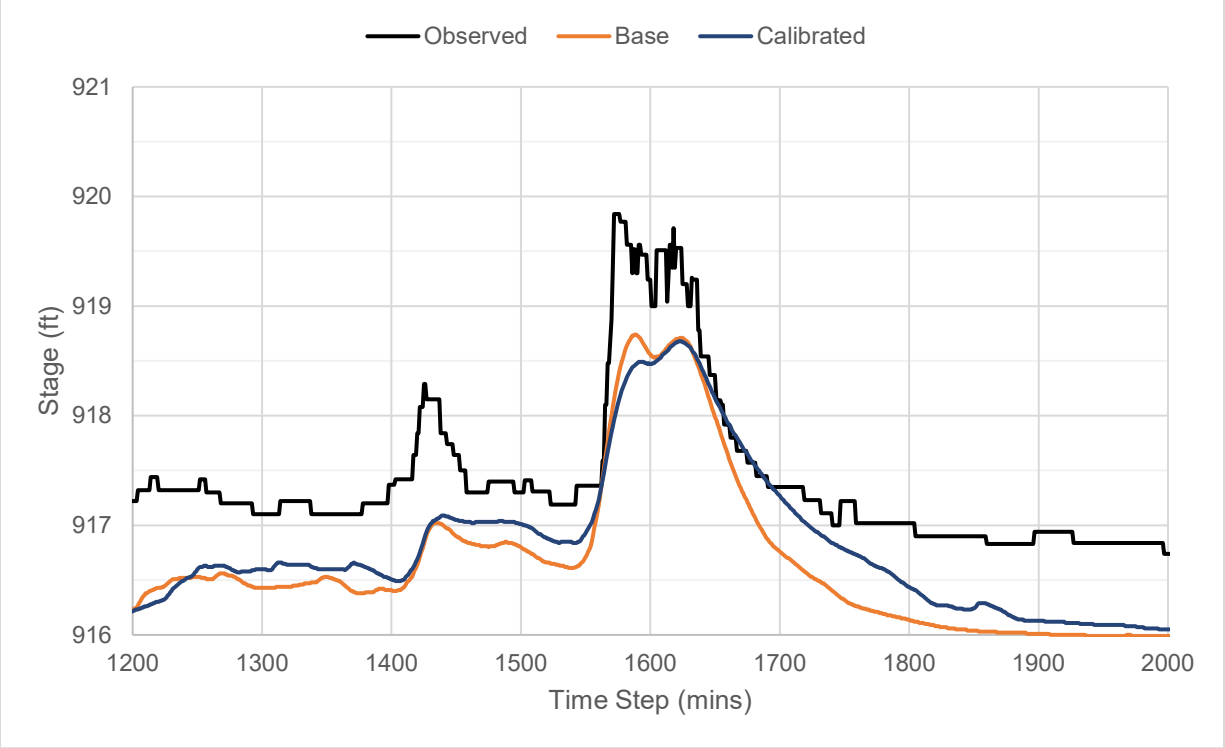


Figure C 9: Base & Calibrated Model Results at BR06-Martway @ Rock Creek

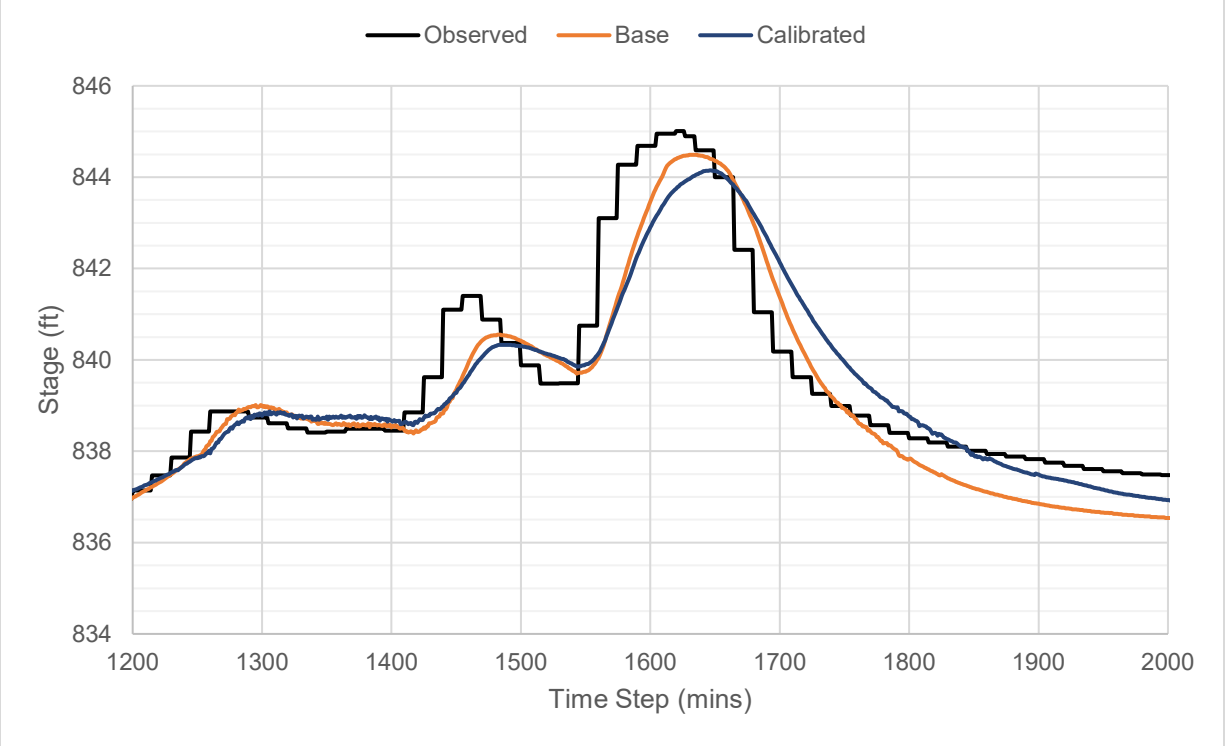


Figure C 10: Base & Calibrated Model Results at USGS-06893557 Brush Creek at Ward Parkway

Appendix D – July 2017 Validation

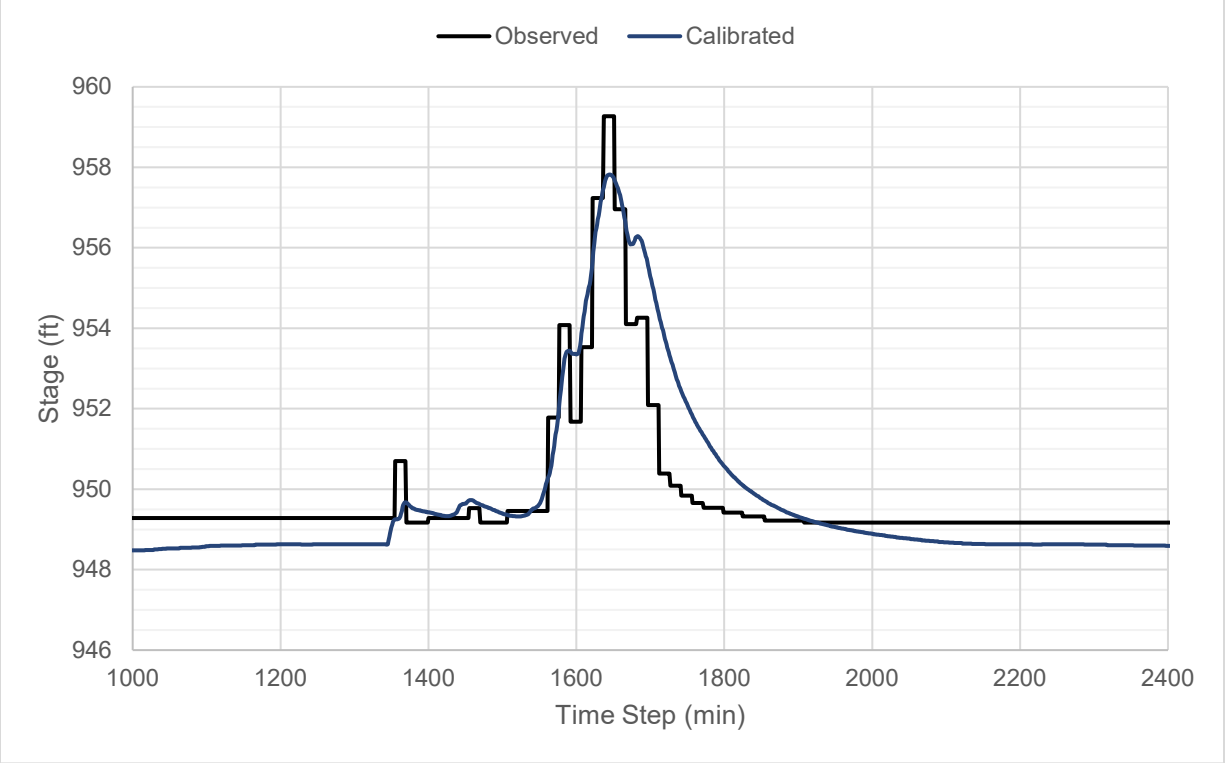


Figure D 1: Calibrated Model Results at TU02-69th St @ Turkey Creek

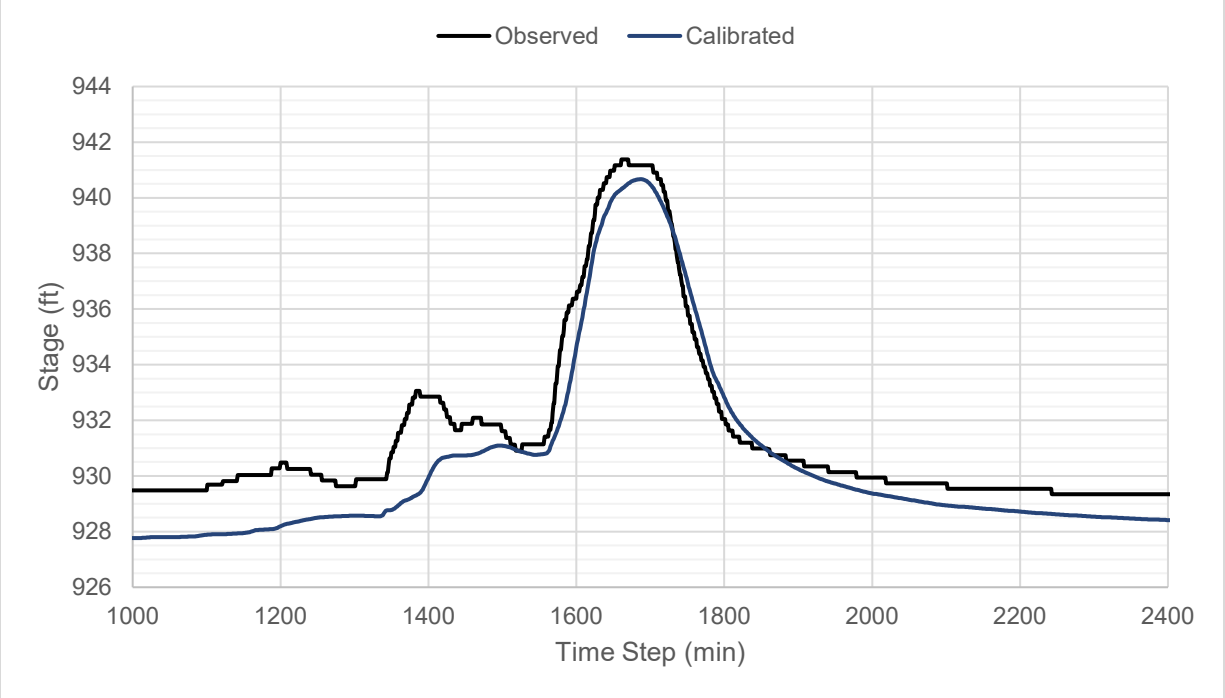


Figure D 2: Calibrated Model Results at TU03-67th St @ Turkey Creek

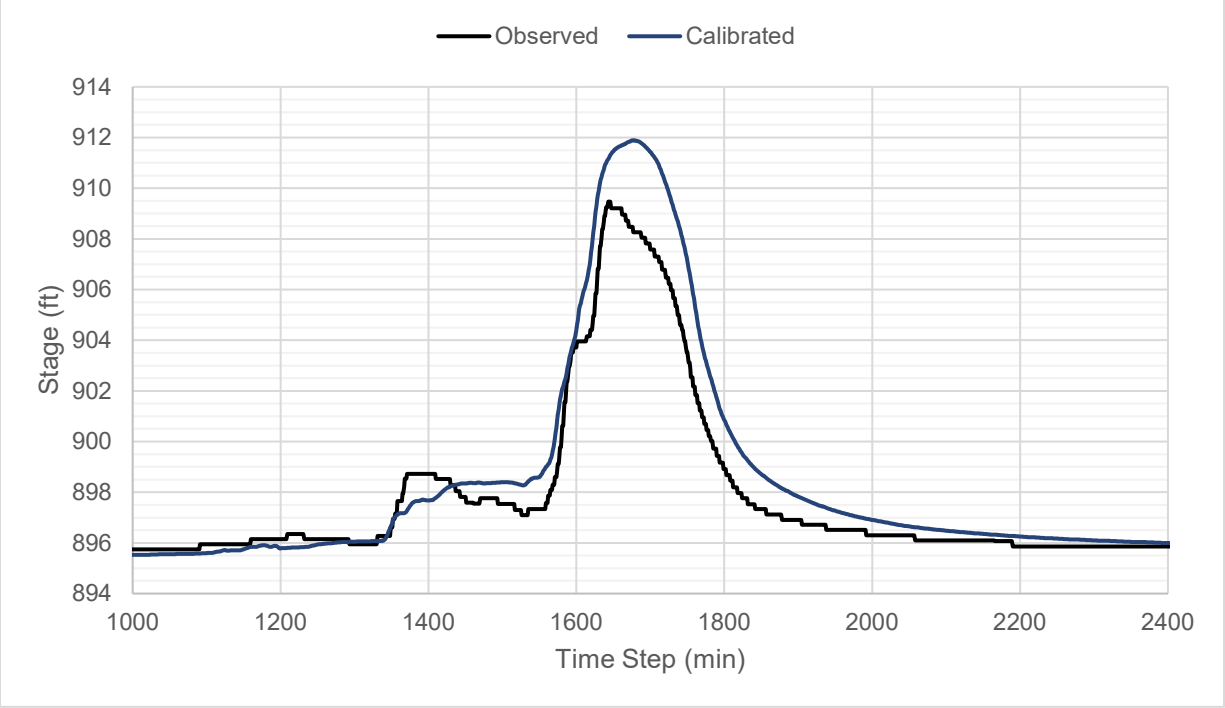


Figure D 3: Calibrated Model Results at TU04-Johnson Dr @ Turkey Creek

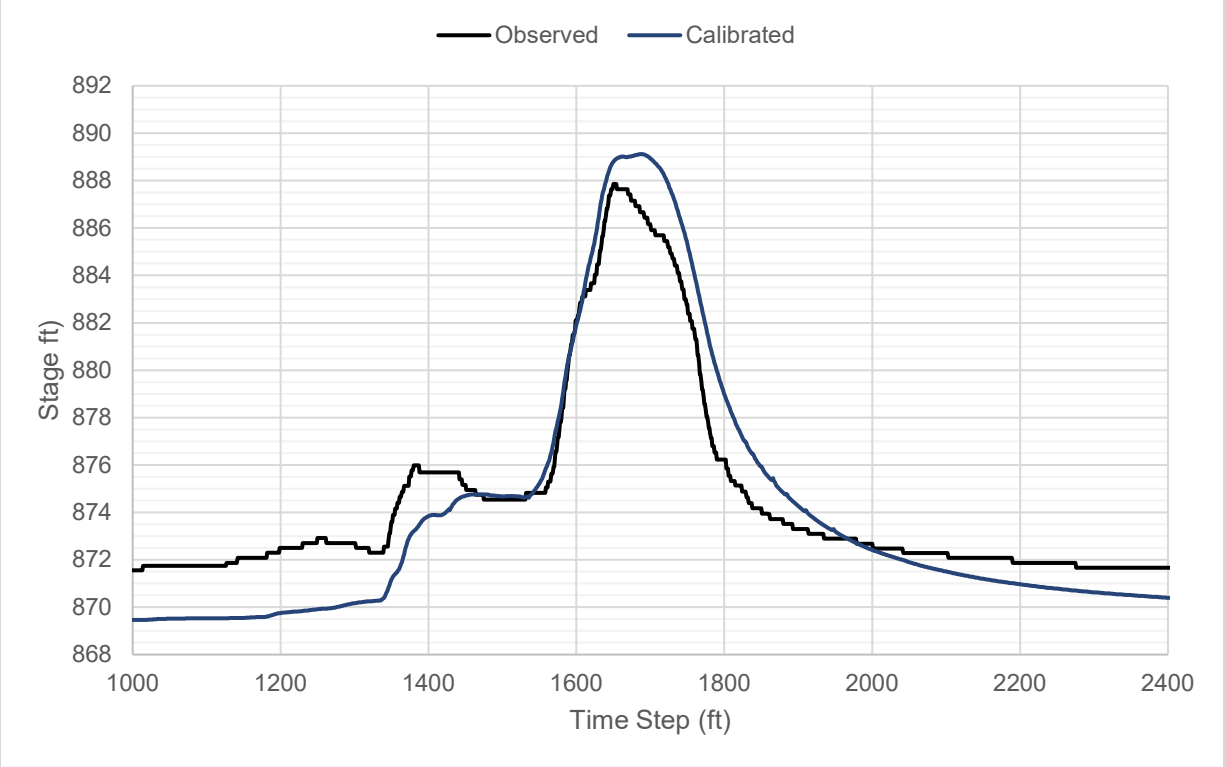


Figure D 4: Calibrated Model Results at TU-06Antioch Rd On-Ramp @ Turkey Creek

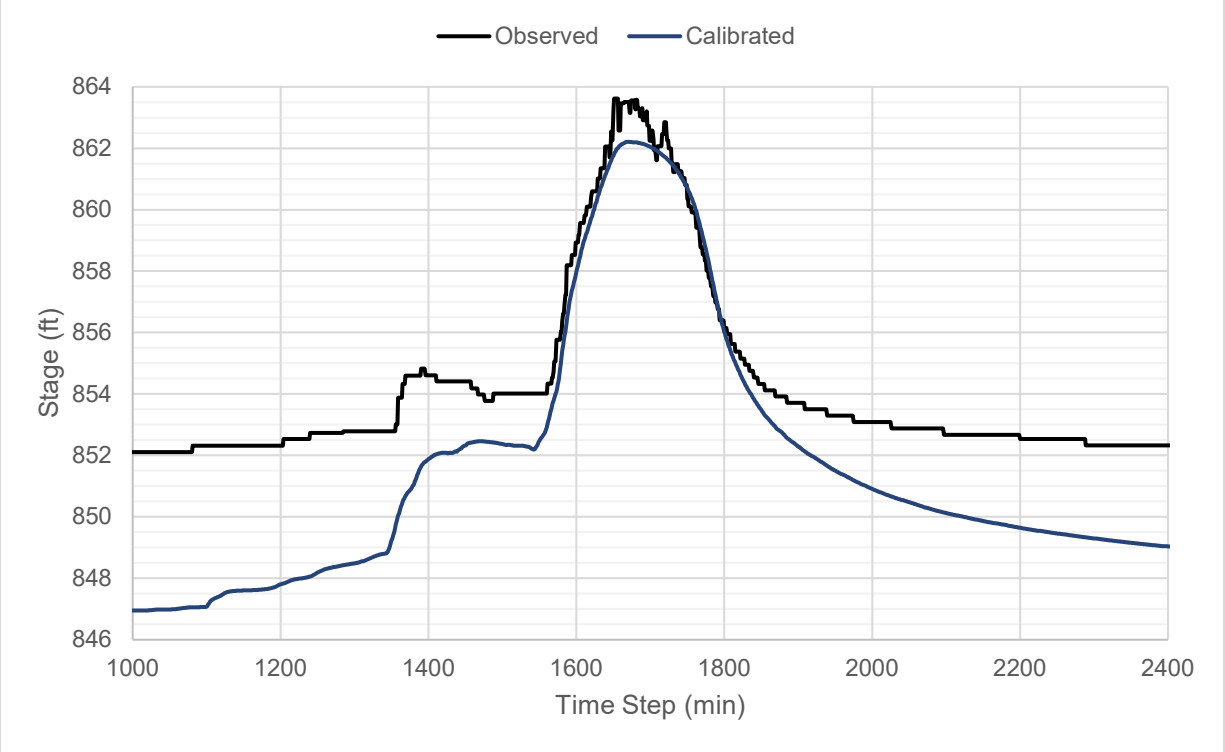


Figure D 5: Calibrated Model Results at TU07-Metcalf Ave @ Turkey Creek

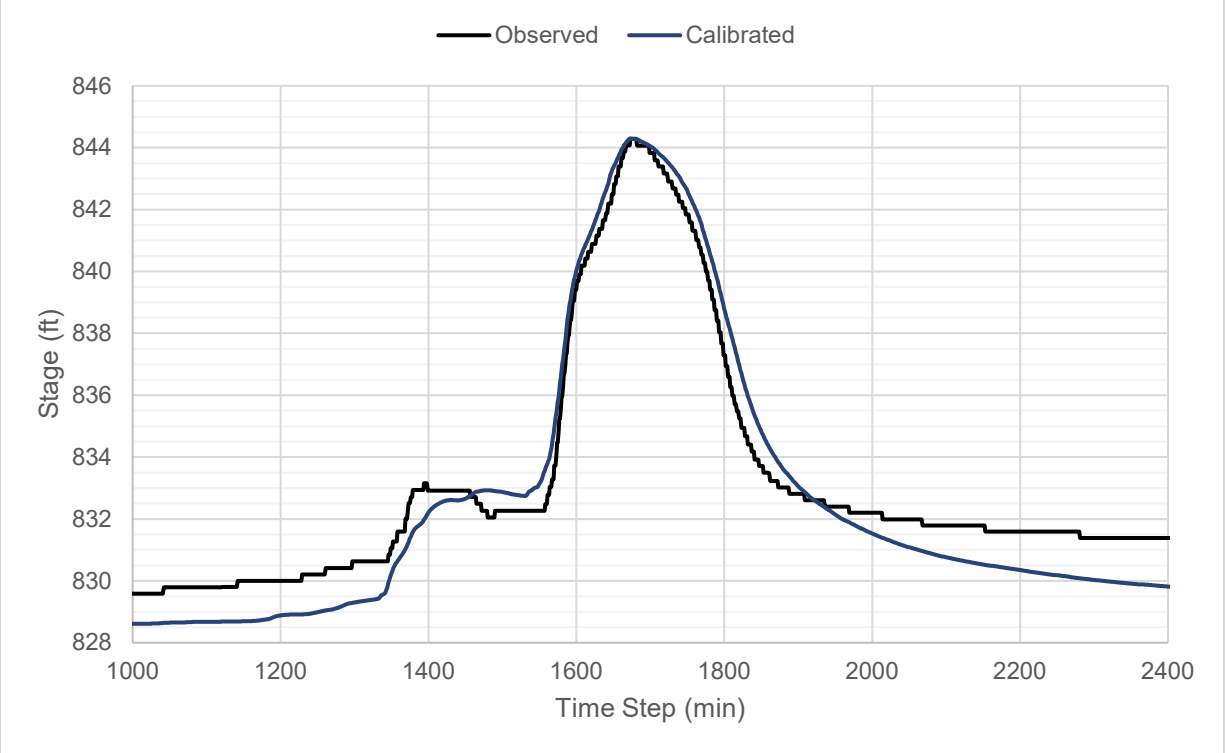


Figure D 6: Calibrated Model Results at TU08-Lamar Ave @ Turkey Creek

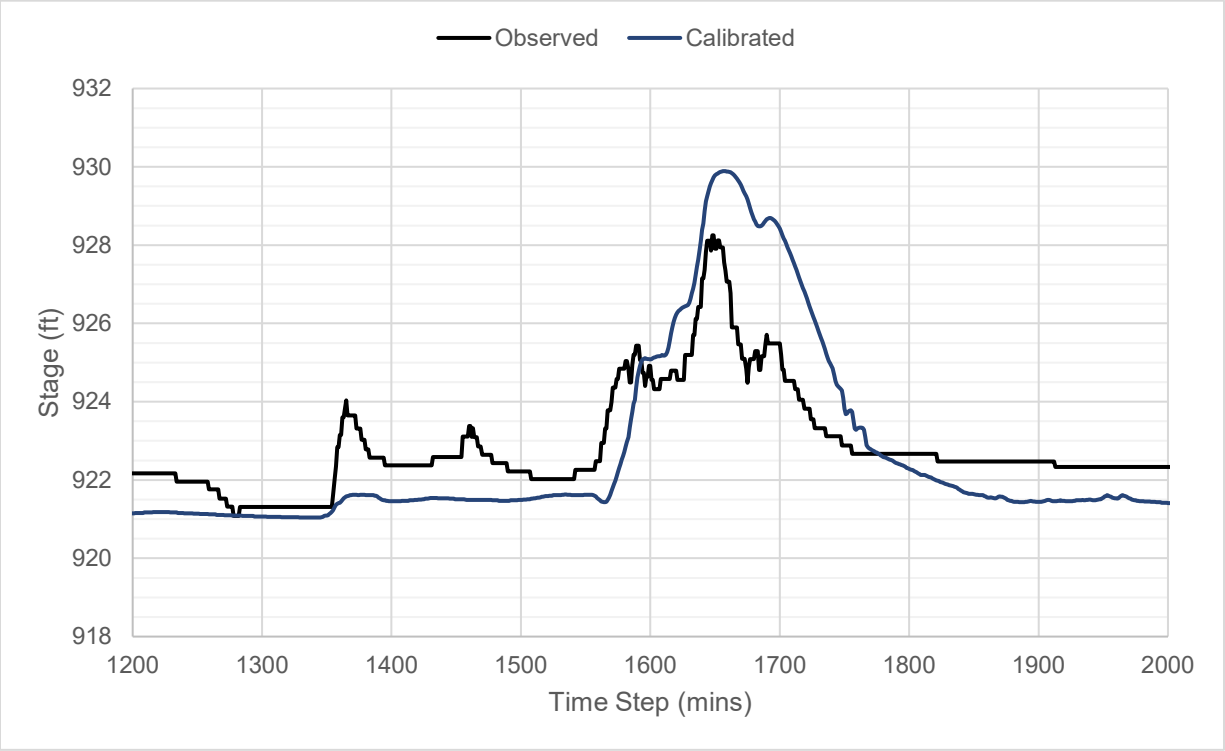


Figure D 7: Calibrated Model Results at BR03-74th St @ Brush Creek

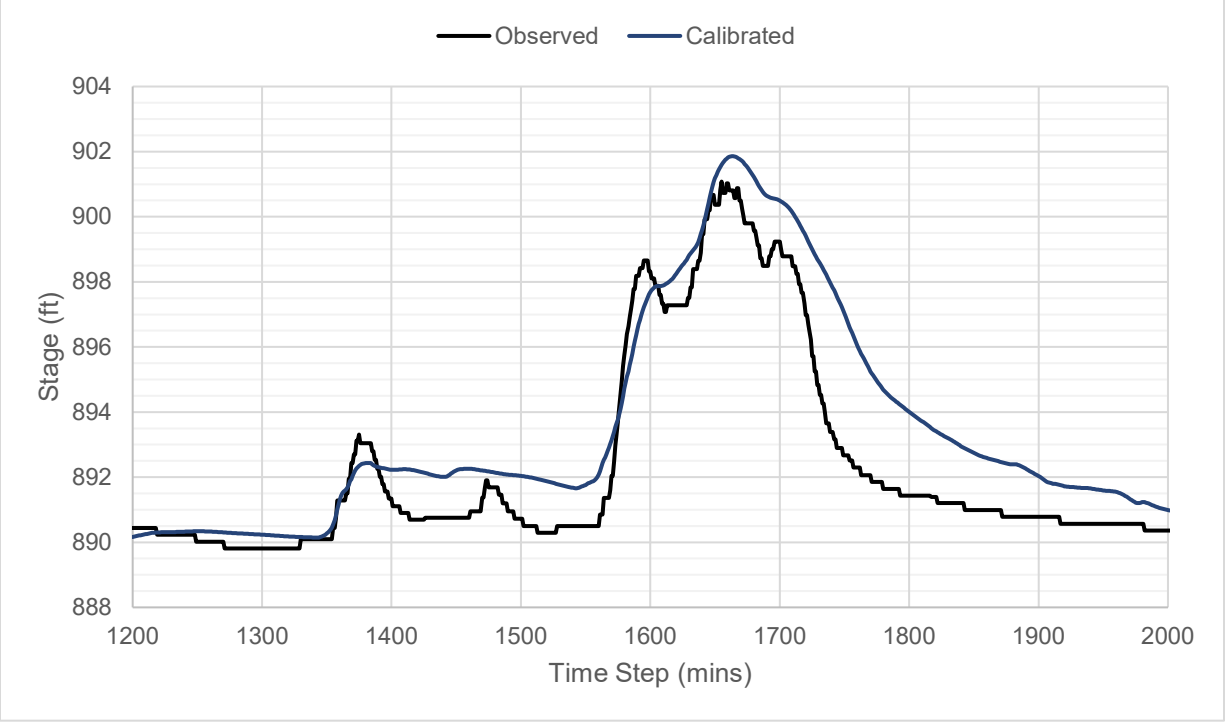


Figure D 8: Calibrated Model Results at BR04-66th St & Mission Rd Master @ Brush Creek

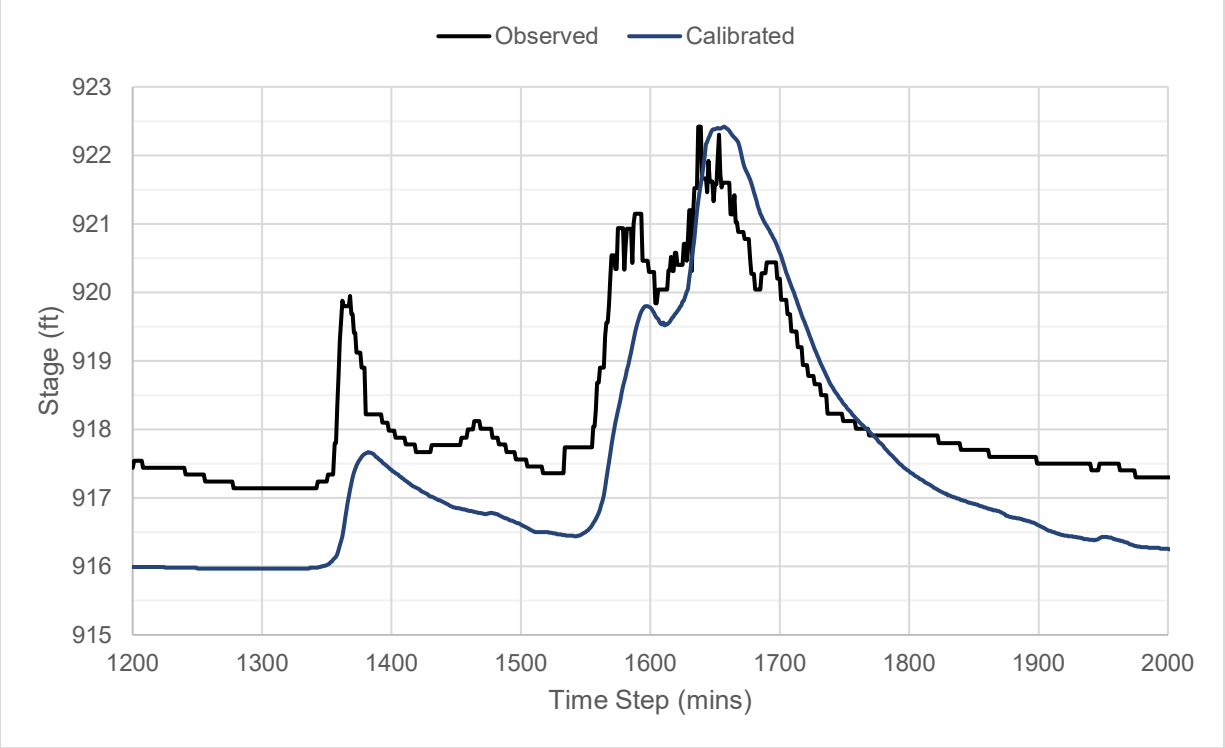


Figure D 9: Calibrated Model Results at BR06-Martway @ Rock Creek

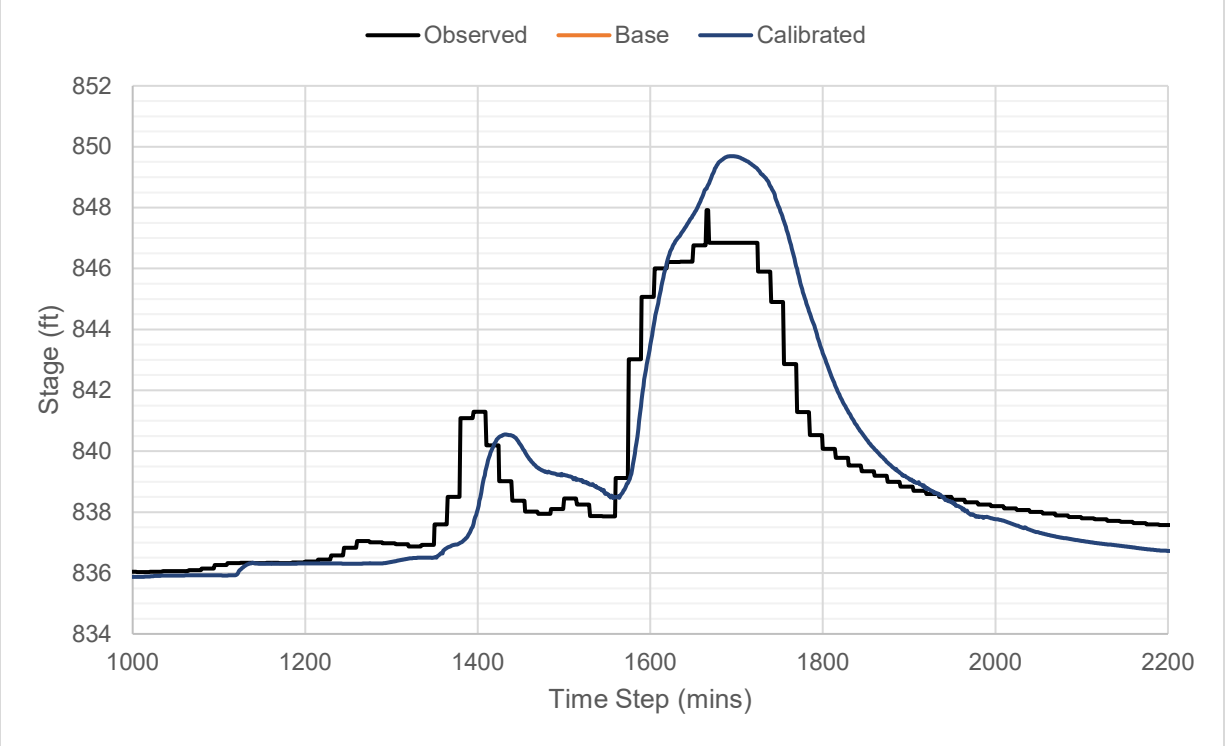


Figure D 10: Calibrated Model Results at USGS-06893557 Brush Creek at Ward Parkway

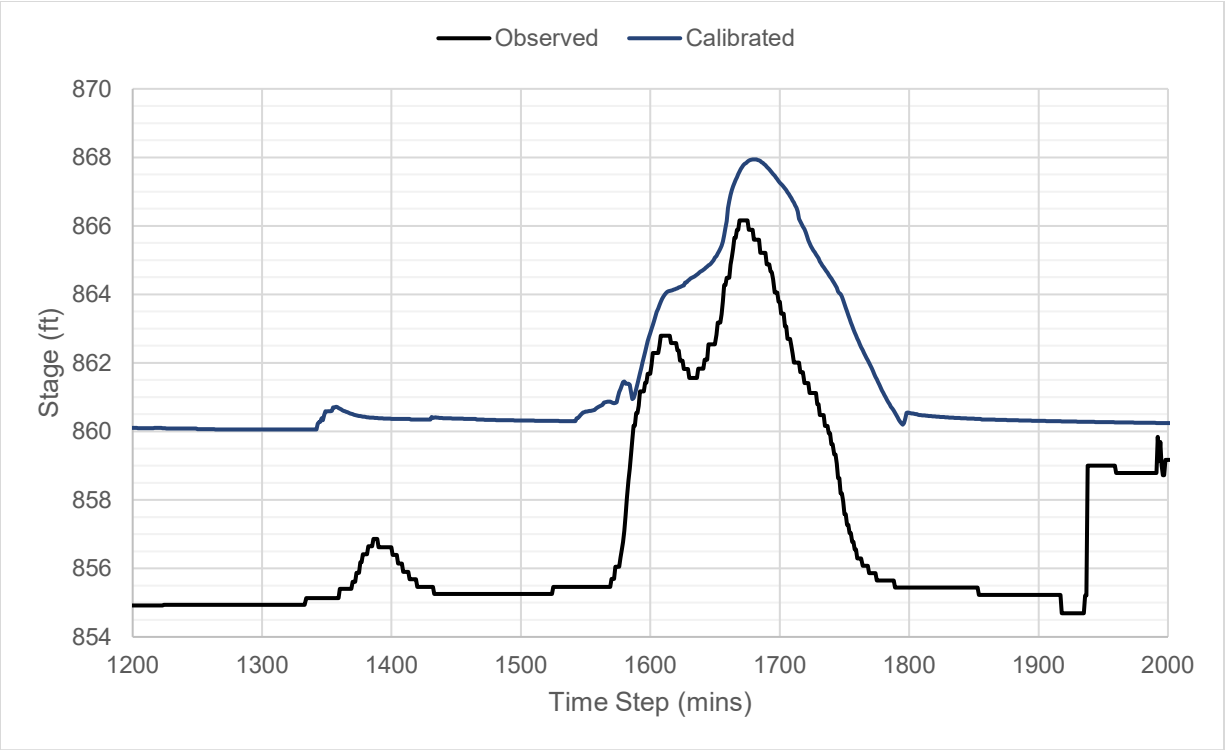


Figure D 11: Calibrated Model Results at BR09-Belinder Ave @ Brush Creek Master

Appendix E - HEC-SSP Gage Analysis

Bulletin 17C (Java) Frequency Analysis
06 Feb 2024 01:51 PM

Skew Option: Use Weighted Skew
Regional Skew: -0.3
Regional Skew MSE: 0.14

Plotting Position Type: Hirsch-Stedinger

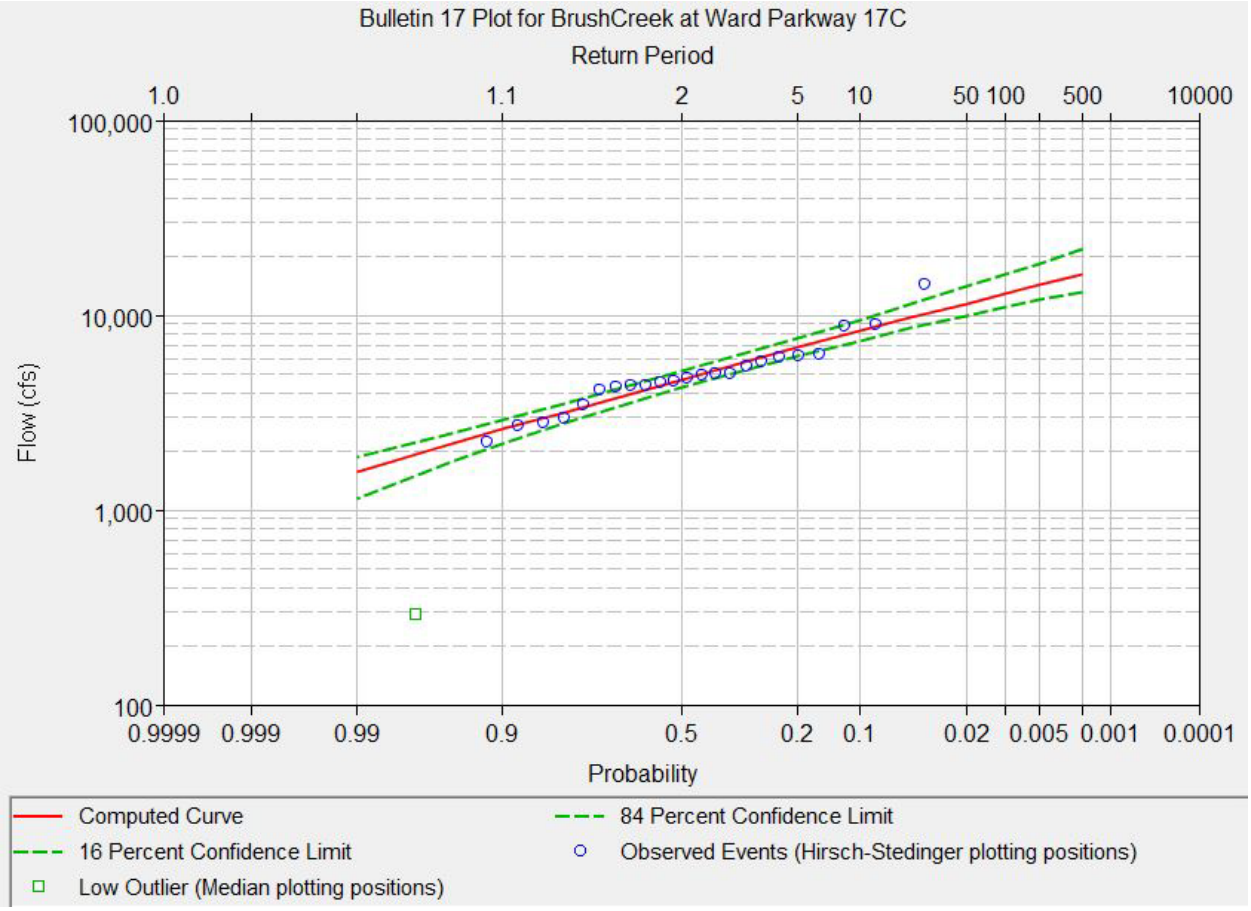
Upper Confidence Level: 0.84
Lower Confidence Level: 0.16

Display ordinate values using 1 digits in fraction part of value

--- End of Input Data ---

<< Frequency Curve >>
Brush Creek, KCMO-Ward Parkway in Kansas City, MO-FLOW-ANNUAL PEAK

Computed Curve FLOW, CFS	Variance Log(EMA)	Percent Chance Exceedance	Confidence Limits 0.84 FLOW, CFS	Confidence Limits 0.16 FLOW, CFS
16,191.6	0.01100	0.200	13,214.6	21,827.7
14,298.3	0.00833	0.500	11,959.5	18,496.3
12,890.4	0.00661	1.000	10,975.9	16,176.7
11,497.7	0.00514	2.000	9,956.9	14,010.9
9,665.0	0.00359	5.000	8,541.6	11,351.7
8,265.5	0.00270	10.000	7,401.6	9,462.8
6,820.7	0.00205	20.000	6,171.3	7,637.7
4,683.4	0.00171	50.000	4,247.0	5,154.3



Bulletin 17C (Java) Frequency Analysis
06 Feb 2024 01:47 PM

Skew Option: Use Weighted Skew
Regional Skew: -0.125
Regional Skew MSE: 0.252

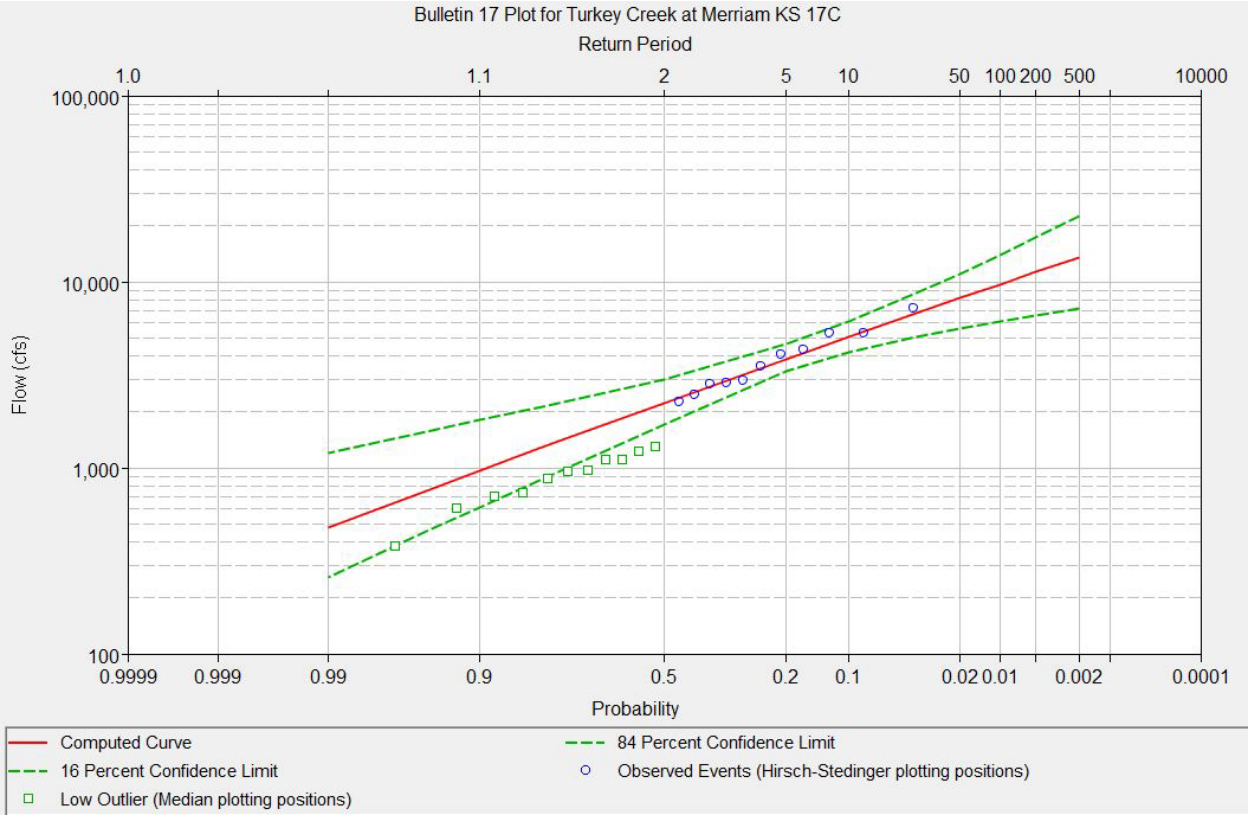
Plotting Position Type: Hirsch-Stedinger

Upper Confidence Level: 0.84
Lower Confidence Level: 0.16

--- End of Input Data ---
<< Frequency Curve >>
Turkey Creek at Merriam KS-MERRIAM, KS-FLOW-ANNUAL PEAK

Computed Curve Log(EMA) FLOW, CFS	Variance	Percent Chance Exceedance	Confidence Limits 0.84 FLOW, CFS	0.16

13,598.7	0.05371	0.200	7,171.3	22,583.8
11,291.8	0.03821	0.500	6,576.5	17,251.3
9,678.4	0.02799	1.000	6,106.6	13,893.5
8,171.4	0.01920	2.000	5,595.6	11,048.7
6,763.2	0.01206	4.000	5,027.9	8,660.6
5,036.2	0.00582	10.000	4,171.6	6,110.9
3,811.1	0.00453	20.000	3,286.0	4,604.1
2,222.3	0.01222	50.000	1,714.4	3,004.0



Appendix F – Stage Comparison

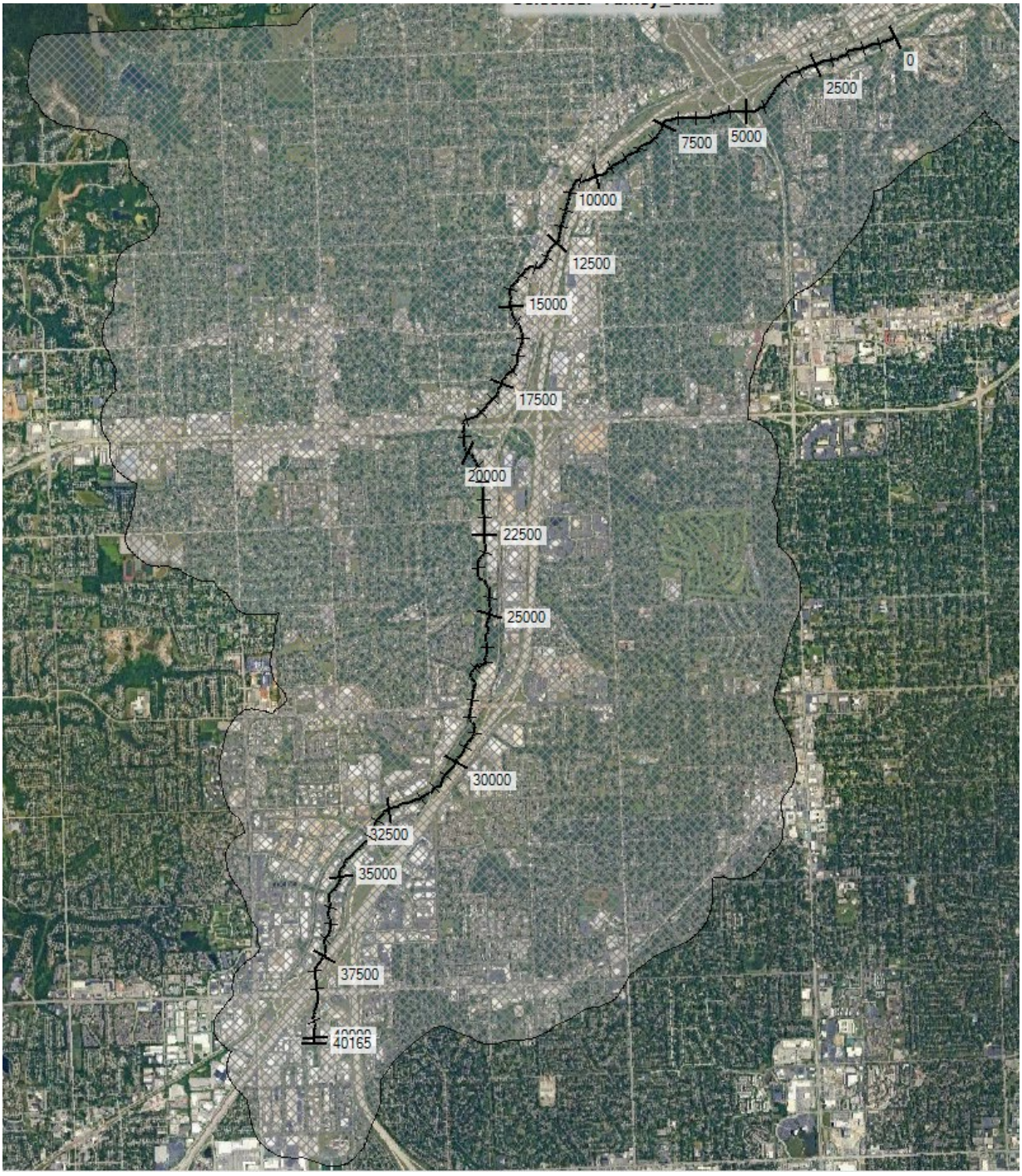


Figure F 1: Turkey Creek Profile for Stage Comparison

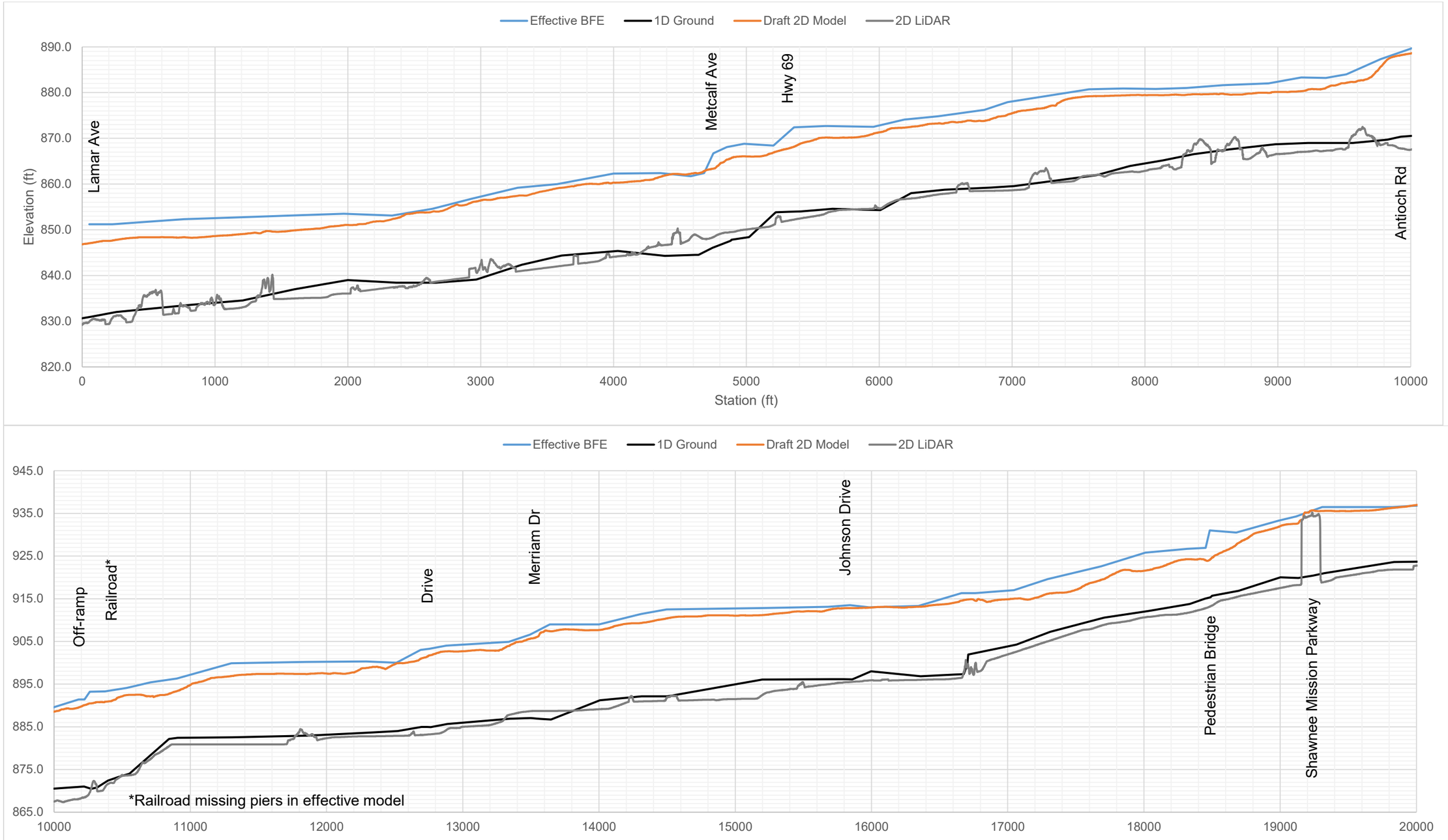
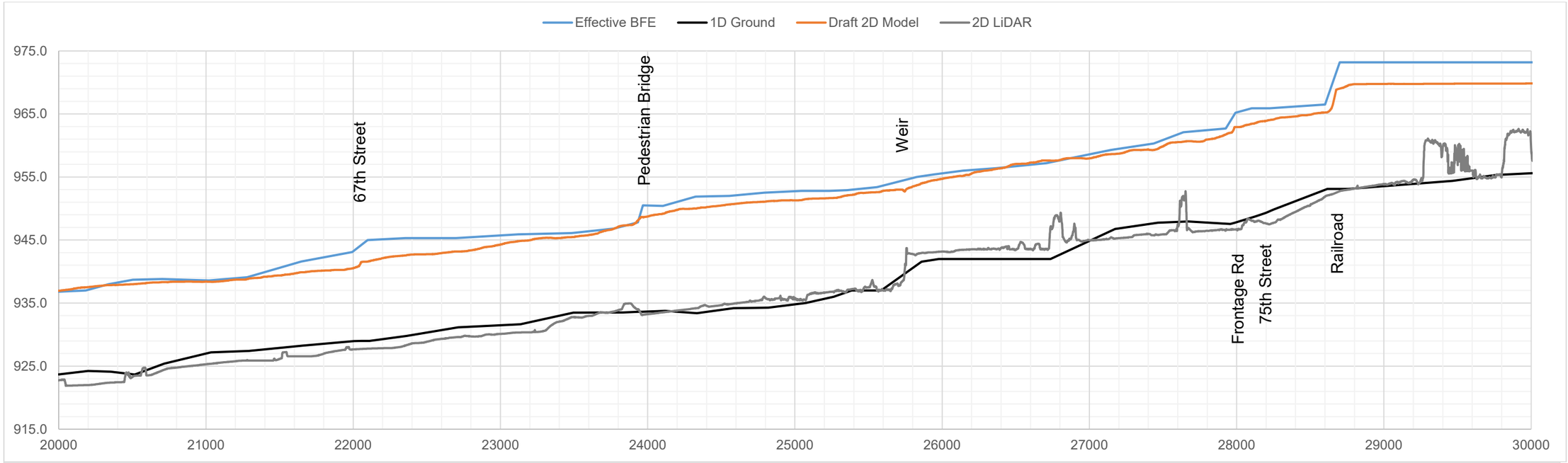


Figure F 2: Turkey Creek Profile Station 0 to 20,000 feet



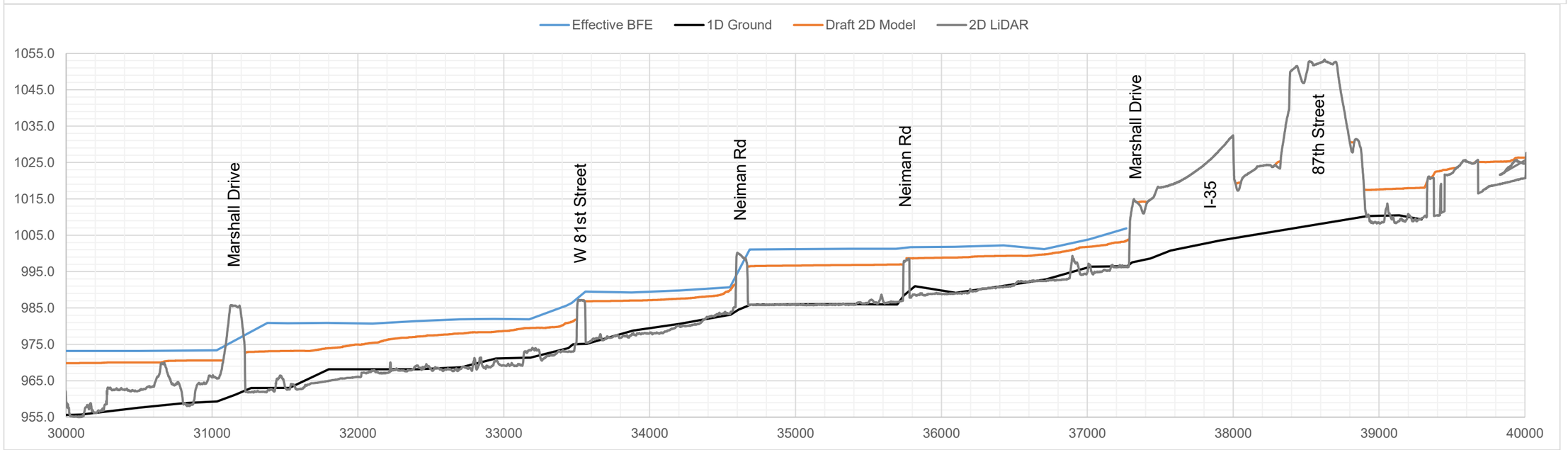


Figure F 3: Turkey Creek Profile Station 20,000 to 40,000 feet

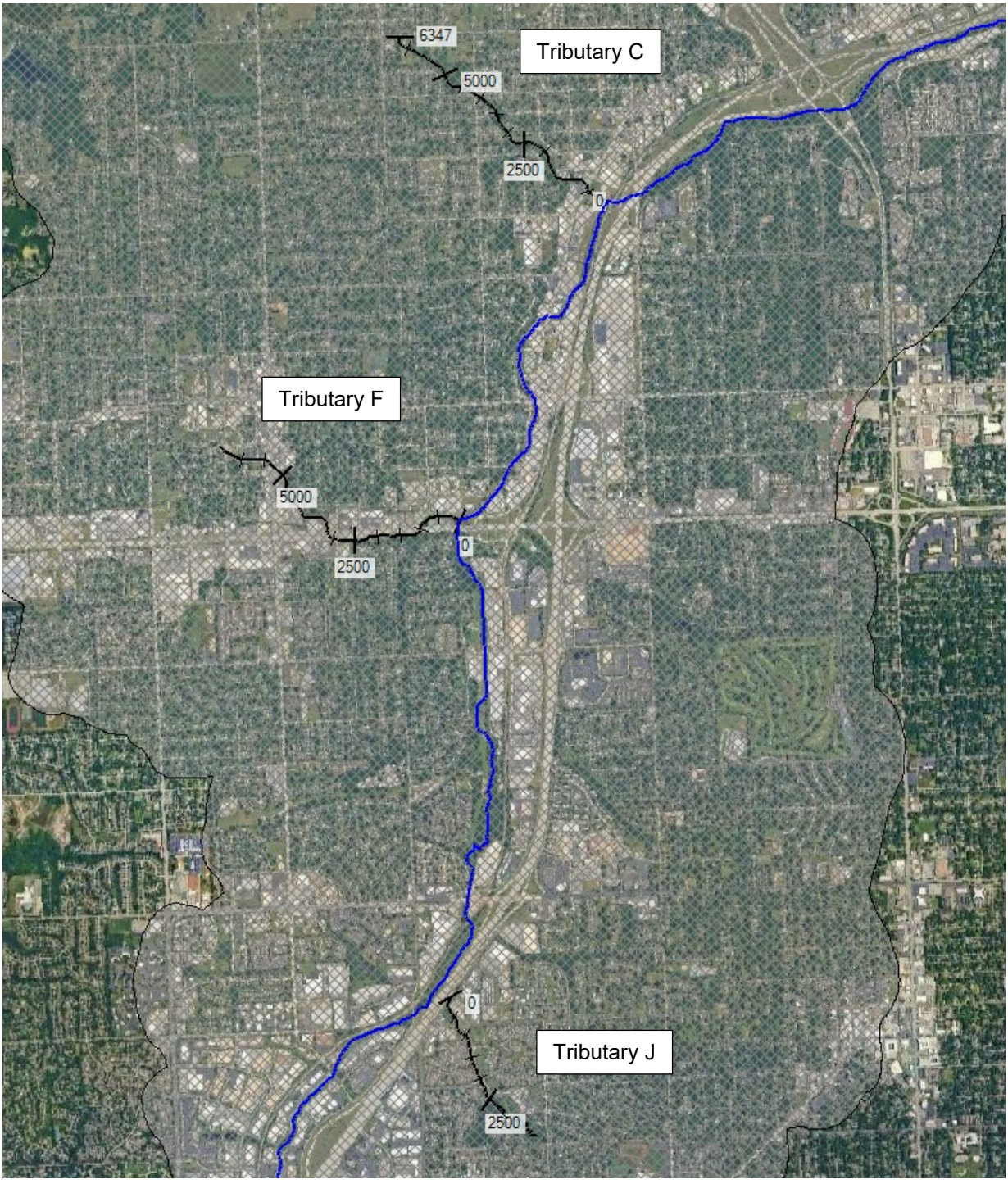


Figure F 4: Turkey Creek Tributaries C, F, and J Profiles for Stage Comparison

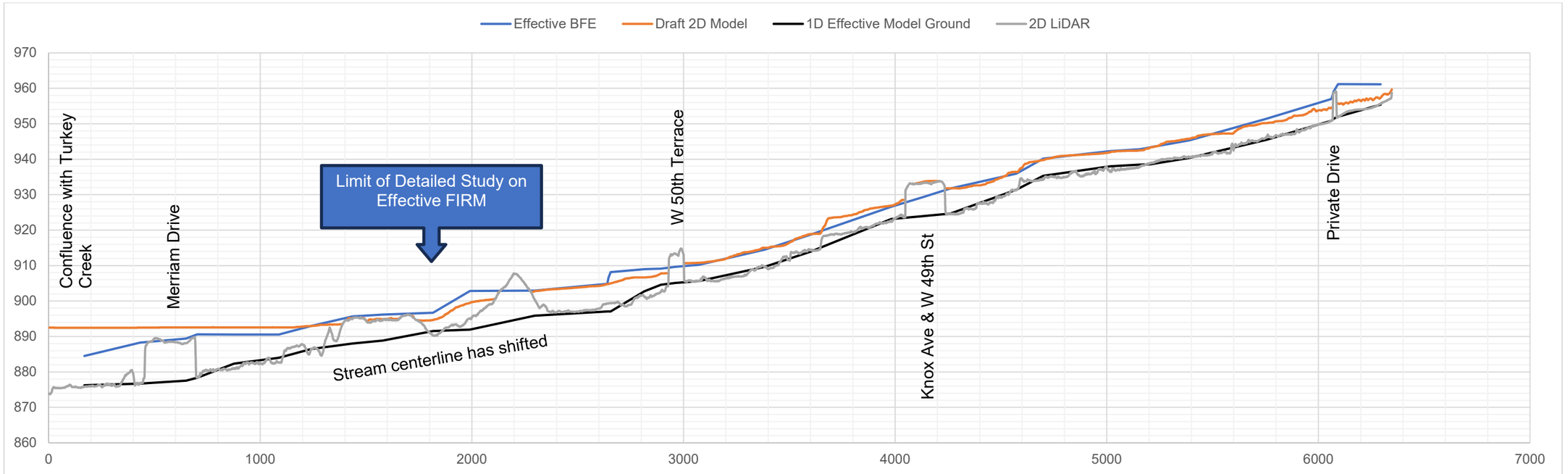


Figure F 5: Turkey Creek Tributary C Profile

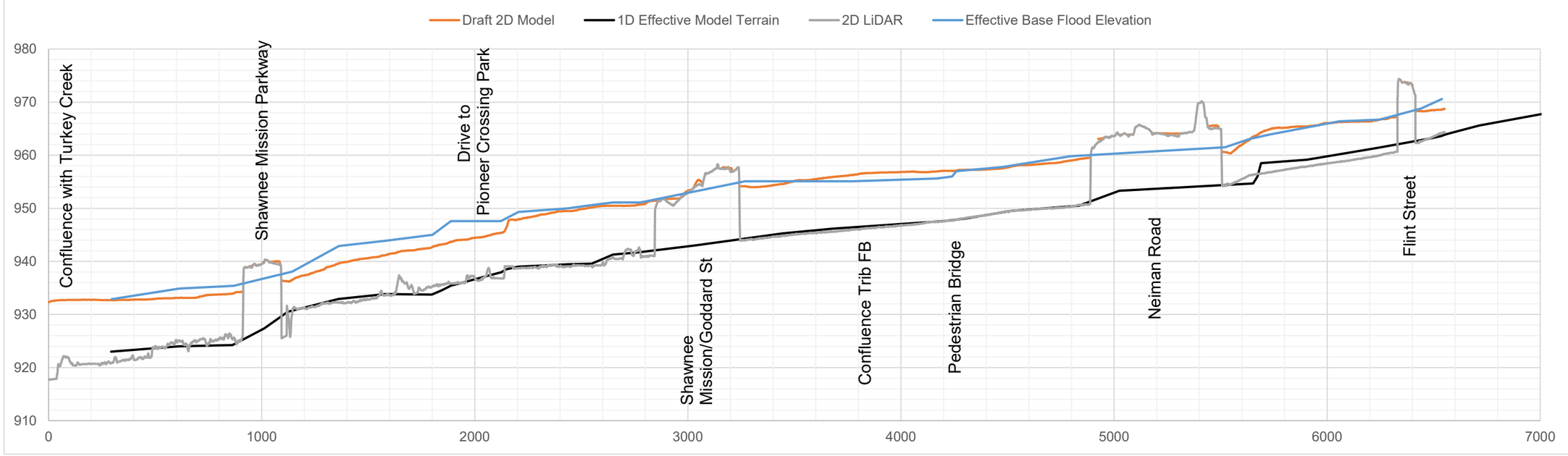


Figure F 6: Turkey Creek Tributary F Profile

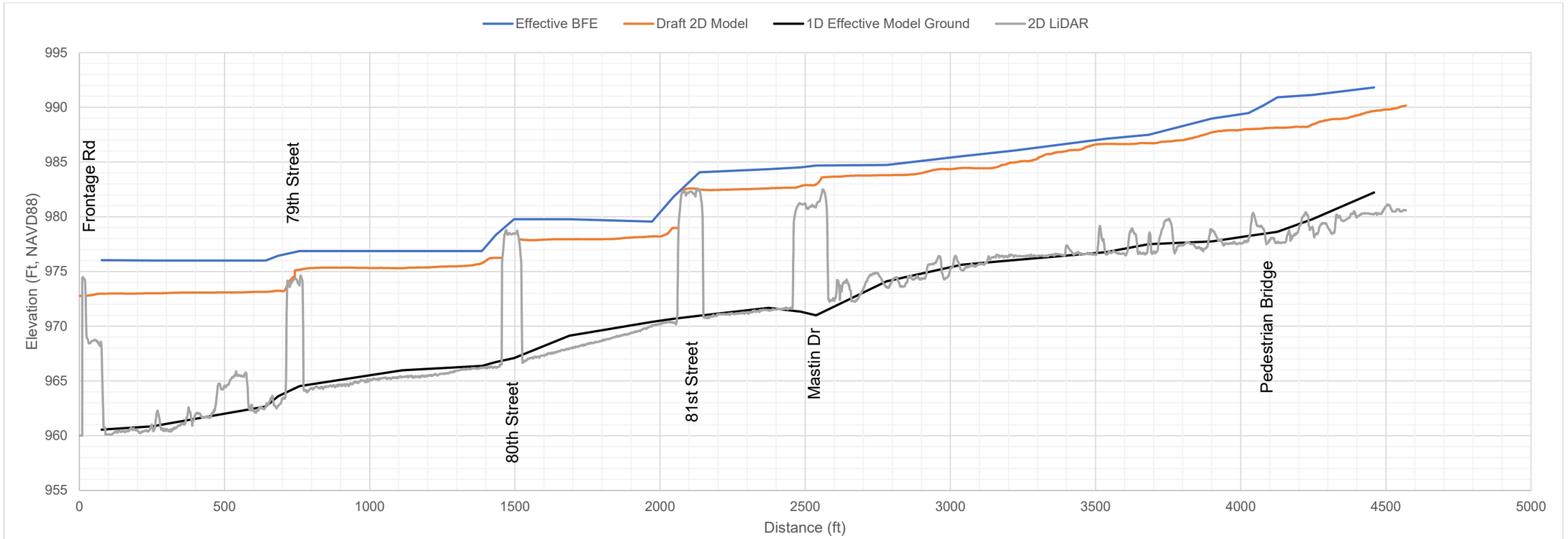


Figure F 7: Turkey Creek Tributary J Profile



Figure F 8: Brush Creek and Rock Creek Profiles for Stage Comparison

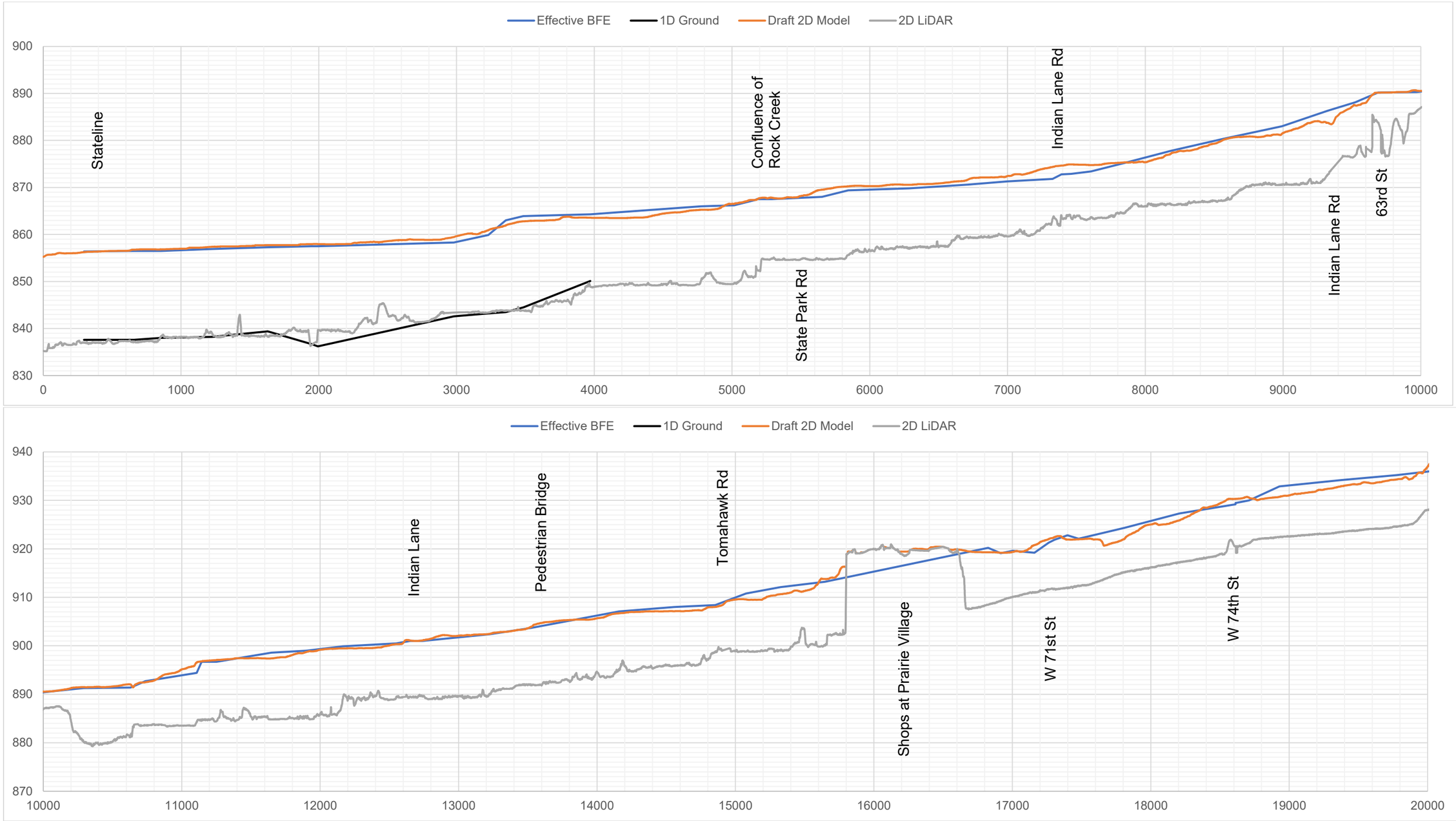


Figure F 9: Brush Creek Profile from Station 0 to 20,000 Feet

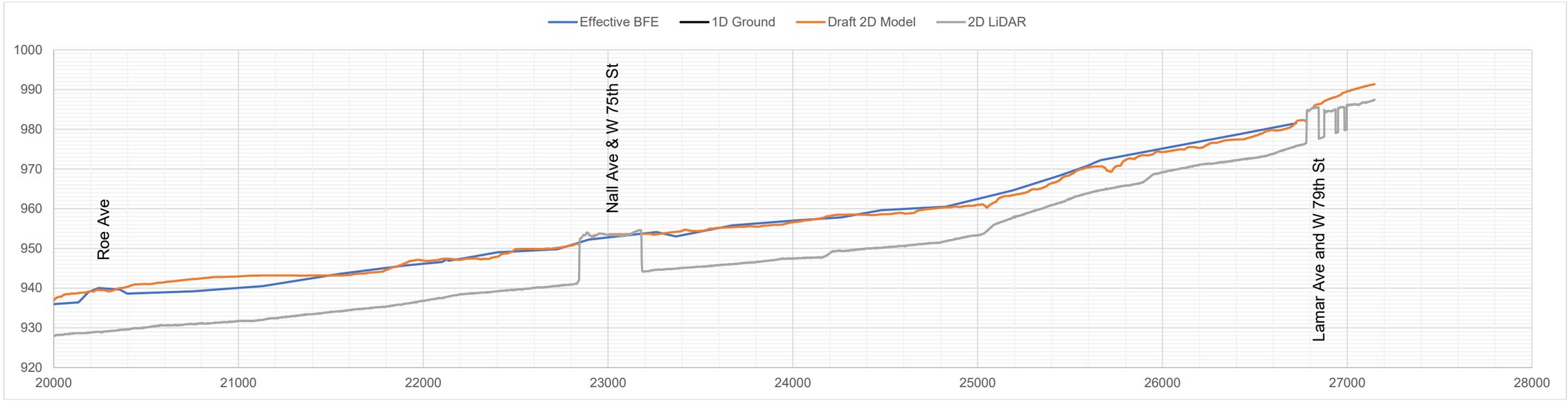


Figure F 10: Brush Creek Profile from Station 20,000 to 28,000 Feet

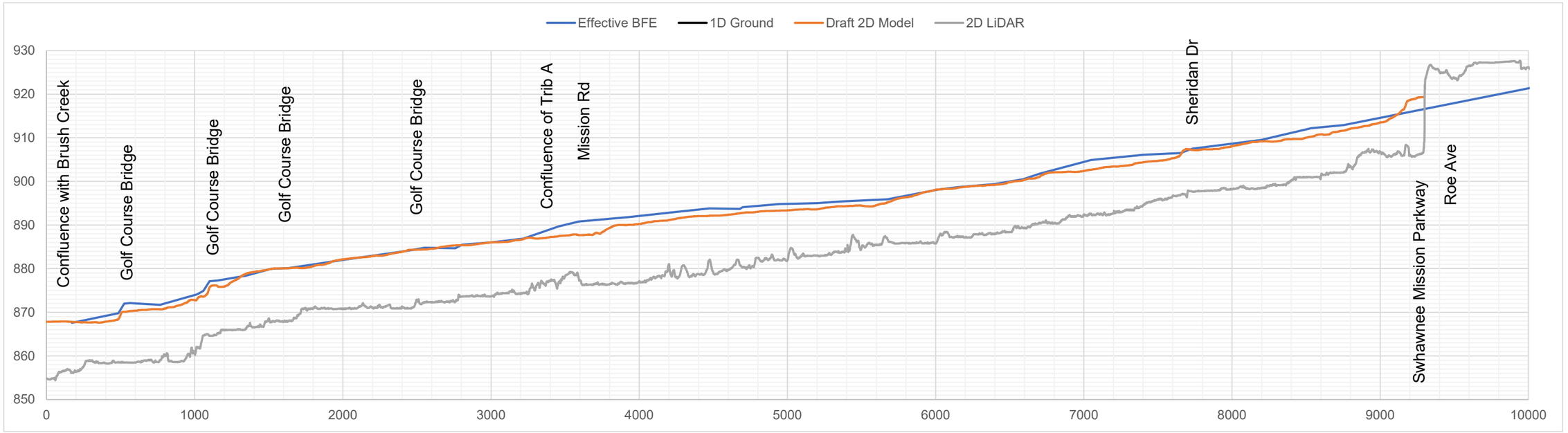


Figure F 11: Rock Creek Profile from Station 0 to 10,000 Feet

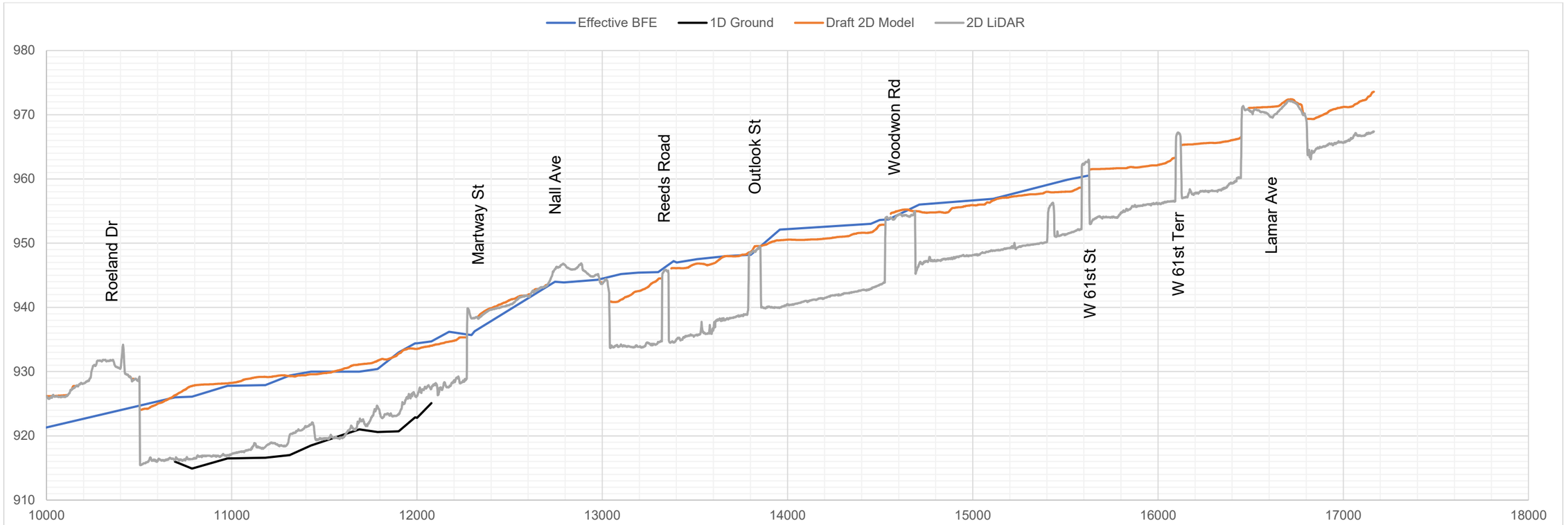


Figure F 12: Rock Creek Profile from Station 10,000 to 18,000

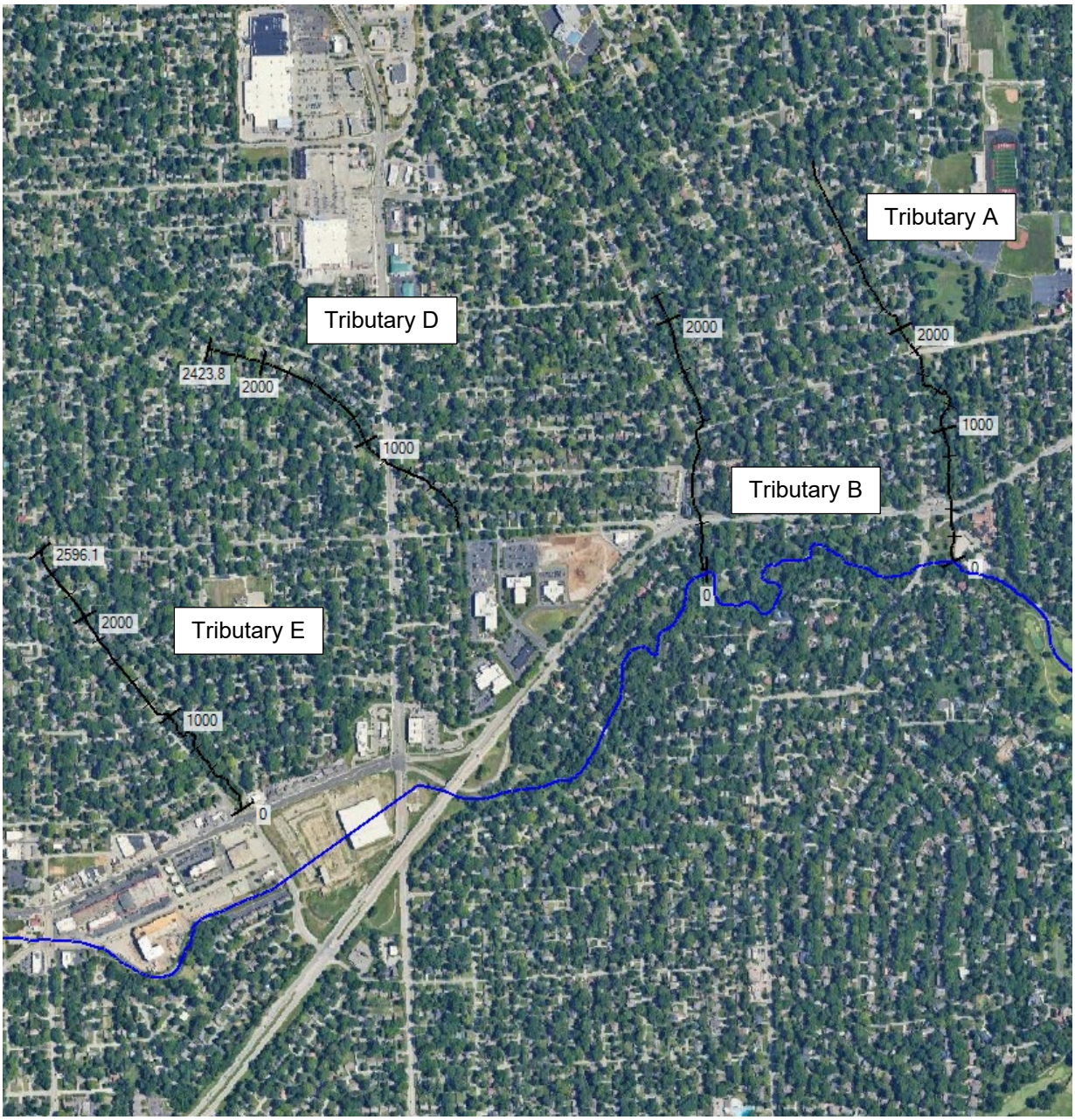


Figure F 13: Rock Creek Tributaries Profiles for Stage Comparison

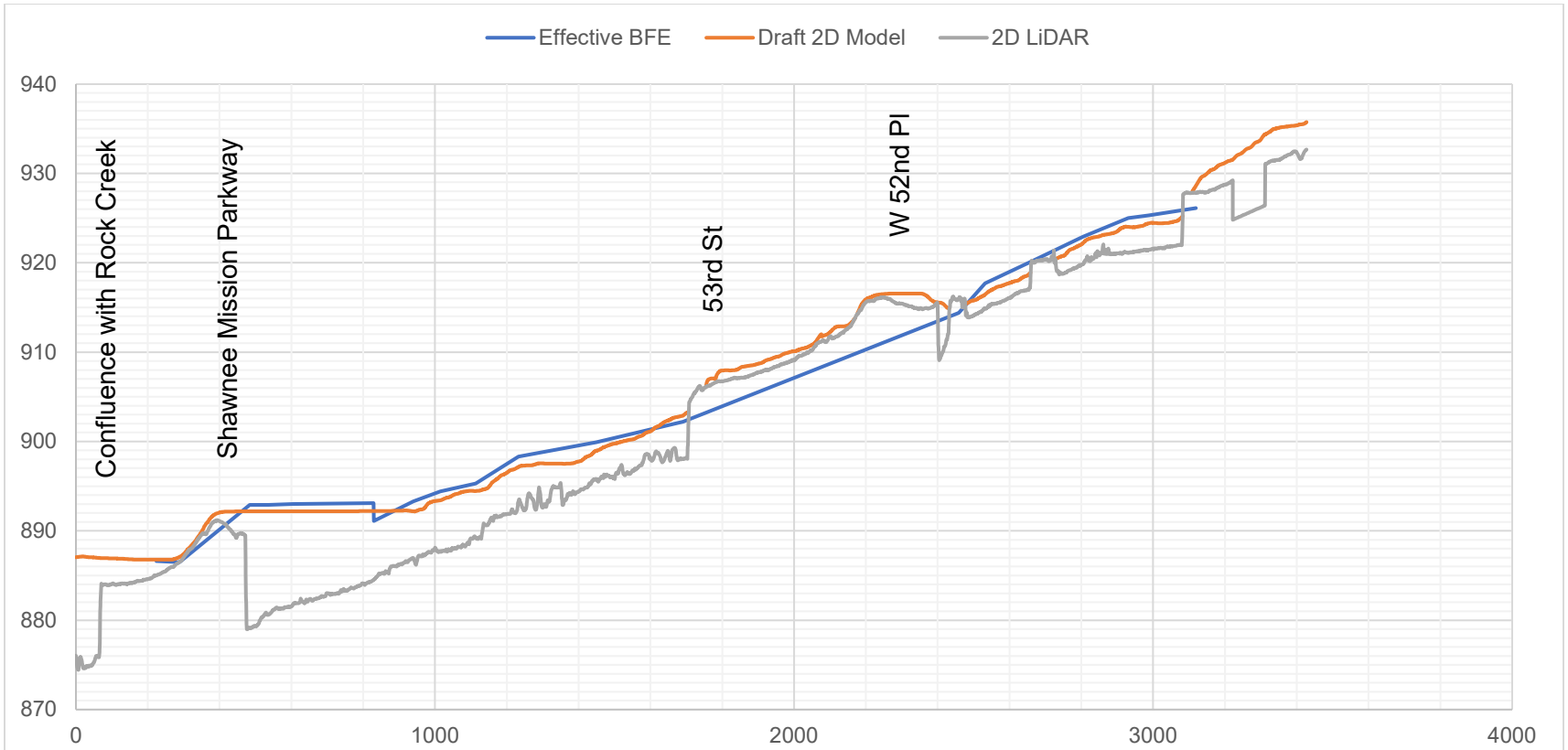


Figure F 14: Rock Creek Tributary A Profile

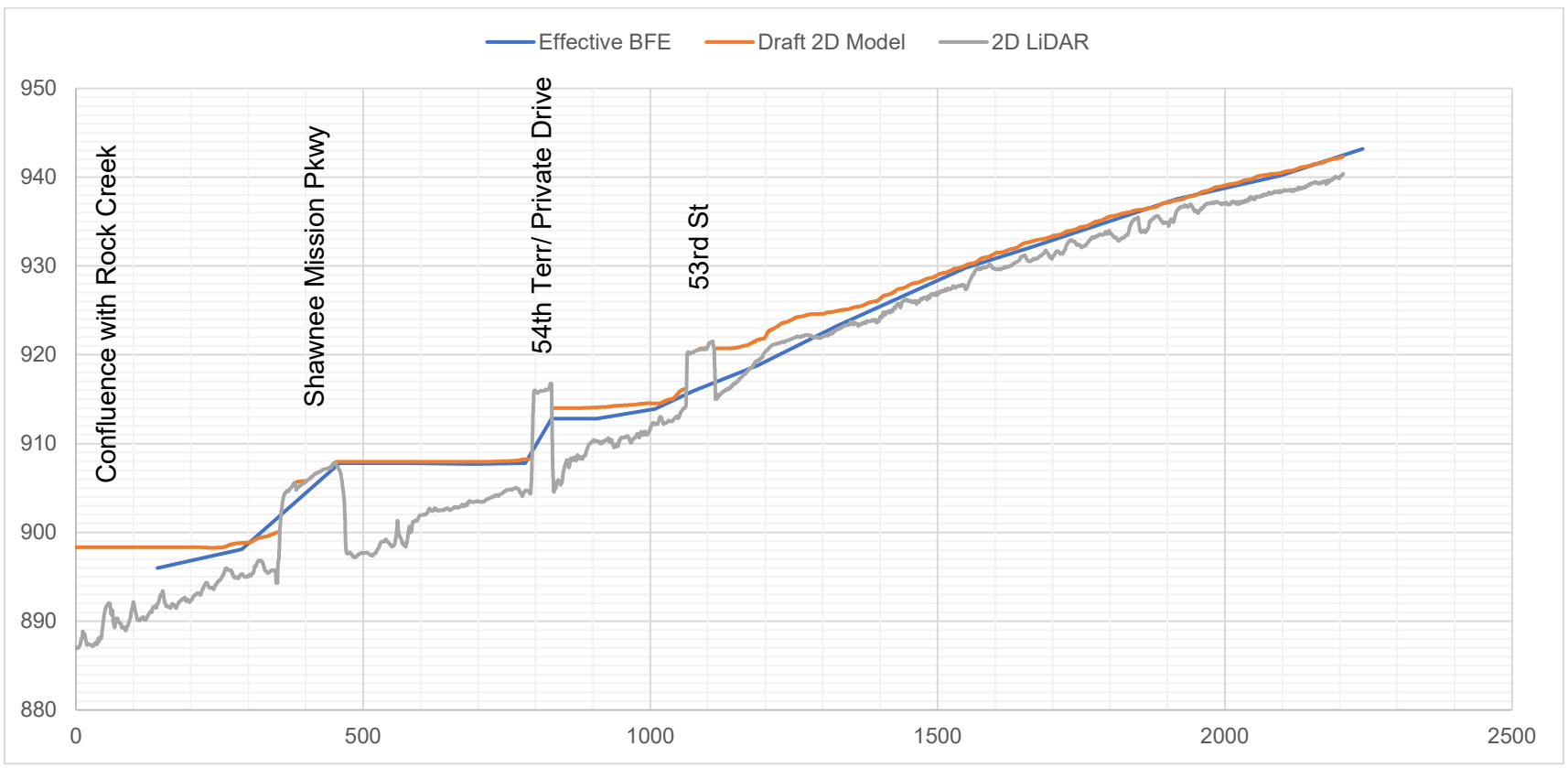


Figure F 15: Rock Creek Tributary B Profile

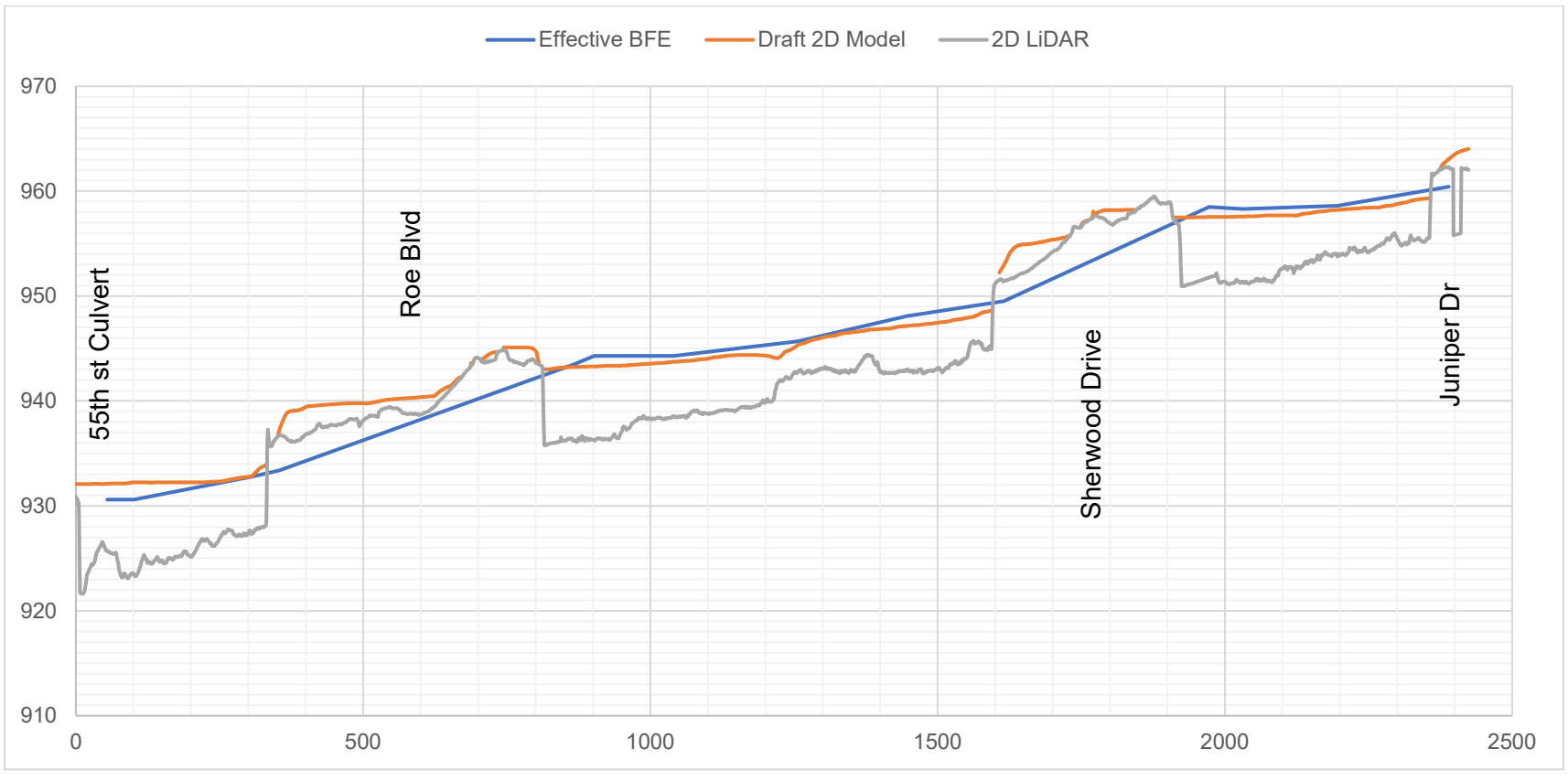


Figure F 16: Rock Creek Tributary D Profile

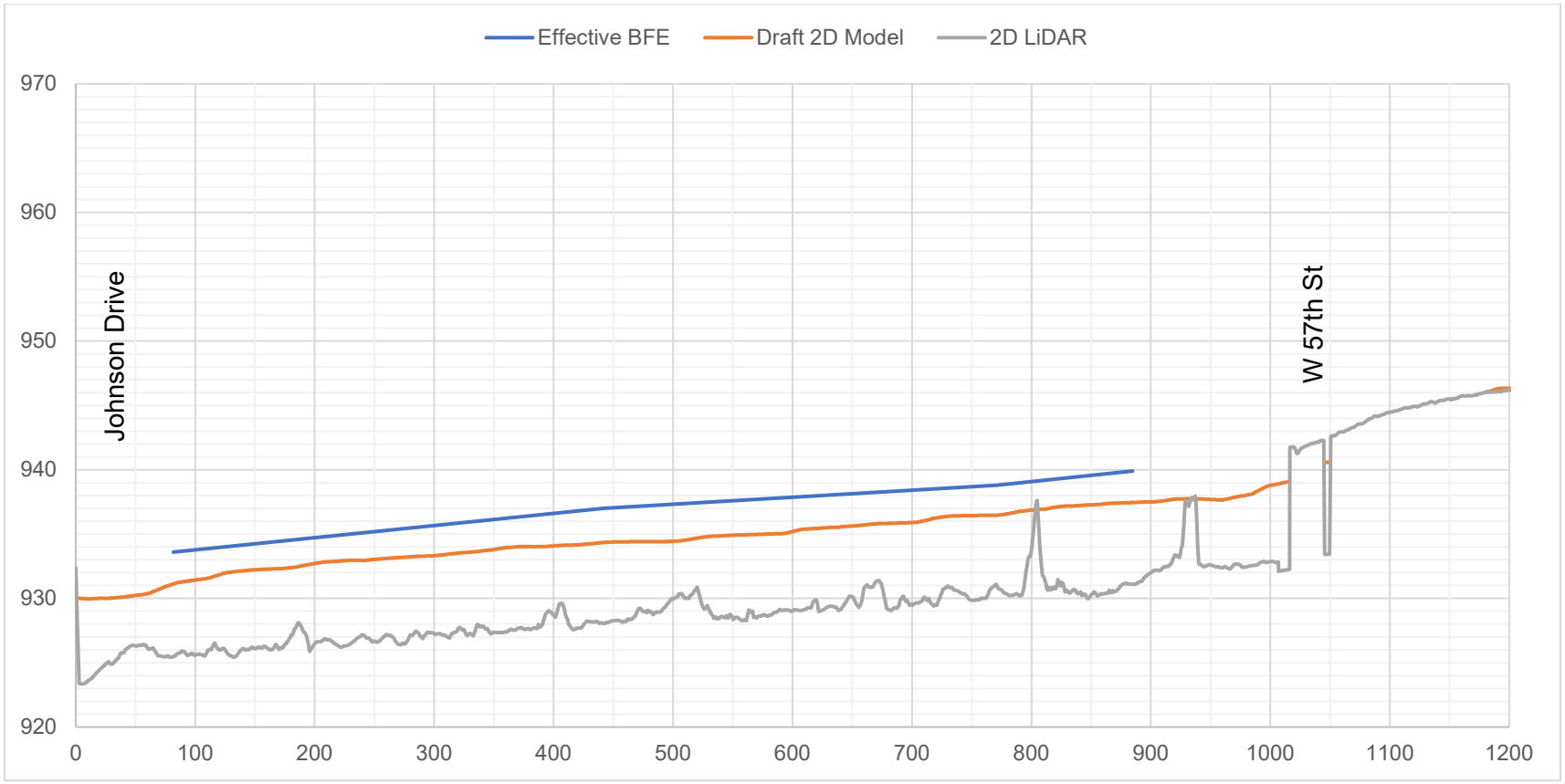


Figure F 17: Rock Creek Tributary E Profile

APPENDIX E.8 – BRIDGE DRAWINGS

REVISED PLANS 05/04/2010

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	1	33

KDOT Proj. No. 46 N-0425-01
FED. Proj. No. BHM- N042 (501)
CITY Proj. No. GK1001

BRIDGE
SEEDING

INDEX OF SHEETS

1 Title Sheet
2 Typical Sections
3 Plan and Profile
4 Drainage Data Sheet
5-10 Bridge Sheets
11 Supports and Spacers for Reinforcing Steel
12 Summary of Quantities Sheet
13 Seeding
14-20 Temporary Erosion and Pollution Control
21 Construction Sequence
22-32 Traffic Control
33 Drainage Area Sheet

STATE OF KANSAS

DEPARTMENT OF TRANSPORTATION

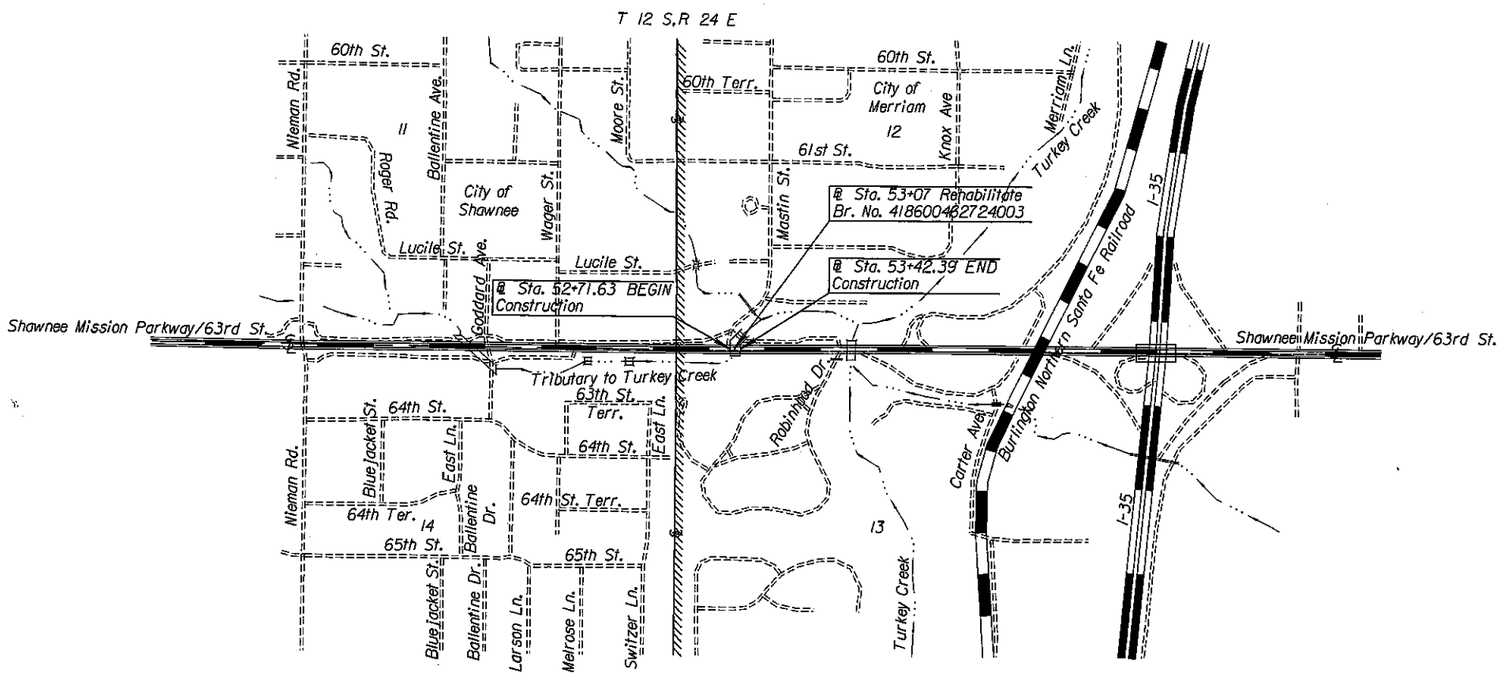
PLAN AND PROFILE OF PROPOSED

STATE HIGHWAY

FEDERAL AID PROJECT

CITY OF MERRIAM

TRIBUTARY TO TURKEY CREEK BRIDGE REHABILITATION



DESIGN DESIGNATION

AADT	41590 (2004)
AADT	
DHV	9%
D	40/60
T	2%
V	45 MPH
C of A	None
Clear Zone	22 ft

CONVENTIONAL SIGNS

COUNTY LINE		CENTER LINE OF PROJECT	
CITY LIMITS		TERRACE	
STATE OR NATIONAL LINE		CULVERTS	
TOWNSHIP, SECTION or GRANT LINE		DROP INLET & STORM SEWER	
PROPERTY LINE		ACCESS CONTROL	
HIGHWAY FENCE		POWER POLE	
EXISTING FENCE		TELEPHONE POLE	
GUARD FENCE		MARSH	
CONSTRUCTION LIMITS		HEDGE	
RIGHT OF WAY LINE		TREES	
TRAVELED WAY		PROFILE ELEVATION	
RAILROADS		STREAM or CREEK	

GROSS LENGTH OF PROJECT	70.76	FT.	
EXCEPTIONS			
ADDITIONS			
NET LENGTH OF PROJECT	0.0	FT.	0.0 MILES
NET LENGTH OF BRIDGES	70.76	FT.	0.013 MILES
NET LENGTH OF ROAD	0.0	FT.	0.0 MILES

EDWARD WILLIAM BULLIN LICENSED 9679 KANSAS PROFESSIONAL ENGINEER
4-5-10

1" = 1000'

Traffic to be carried through construction.

RECOM. FOR APPROVAL-DATE

CHIEF, BUREAU OF LOCAL PROJECTS
KANSAS DEPARTMENT OF TRANSPORTATION

APPROVED - DATE 4-8-10

CITY ENGINEER, CITY OF MERRIAM
APPROVED FOR ONE YEAR FROM THIS DATE.

APPROVED - DATE

STATE TRANSPORTATION ENGINEER
KANSAS DEPARTMENT OF TRANSPORTATION

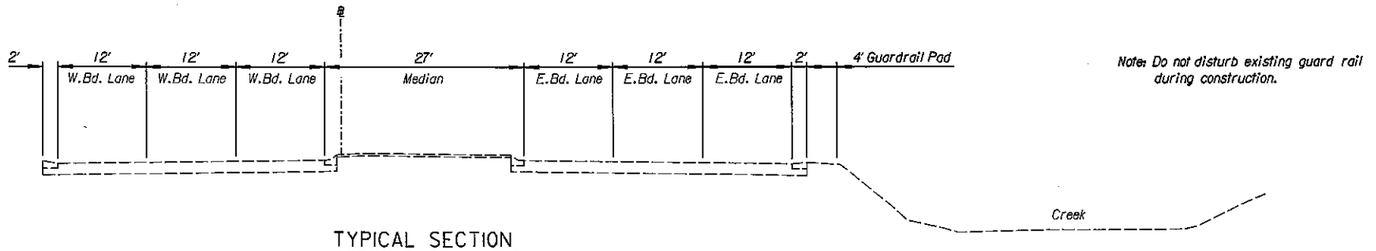
DATE	BY

SURVEY
CADD TECHNICIAN
DESIGNERS
SQUAD

Drawn By : cluedke Plotted : 10/28/2009
File : G:\11694\MicroHighways\11694l300.dgn

2010

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	2	33



TYPICAL SECTION
EXISTING SHAWNEE
MISSION PARKWAY

Note: Do not disturb existing guard rail during construction.

Guide to designer: Use this sheet when subgrading in rock or shale is not required.
See Soils and Geology Reports for additional plan notes.

Drawn By : cluedke Plotted : 10/29/2009
File : G:\11694\Microstation\highways\11694202.dgn

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL SECTION
SHAWNEE MISSION PARKWAY

FHWA APPROVAL		APP'D.	
DESIGNED	DETAILED	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	3	33

GENERAL NOTE

THE GEOLOGICAL INFORMATION SHOWN ON THESE PLANS IS FROM STUDIES MADE IN THE FIELD AND REPRESENTS THE BEST INFORMATION AVAILABLE TO THE KANSAS DEPARTMENT OF TRANSPORTATION.

AT BORROW AREA LOCATIONS ADJACENT TO THE RIGHT OF WAY, UTILITY POLES MAY BE SET AT THE PERMANENT LOCATIONS PRIOR TO CONSTRUCTION AS APPROVED BY THE ENGINEER PROVIDED A MINIMUM VERTICAL CLEARANCE, IN ACCORDANCE WITH THE NATIONAL ELECTRICAL SAFETY CODE, IS OBTAINED. THE CONTRACTOR WILL BE REQUIRED TO WORK AROUND THESE POLES TO COMPLETE THE WORK.

ALL BORROW TO BE OBTAINED FROM AREAS PROVIDED BY THE CONTRACTOR SHALL BE APPROVED BY THE ENGINEER, BOTH AS TO SUITABILITY OF MATERIAL AND SITE LOCATION, LOCATIONS WHICH, IN THE OPINION OF THE ENGINEER, CONTAIN UNSUITABLE MATERIAL OR WILL LEAVE AN UNSIGHTLY APPEARANCE ON THE PROJECT WILL NOT BE APPROVED.

CHANNELS SHALL BE CUT AT BOX CULVERTS (UNLESS OTHERWISE NOTED) TO FLOW LINE ELEVATIONS AND TO A WIDTH OF ONE FOOT OUTSIDE OF EACH OUTSIDE WALL AND WITH SLOPES 2 TO 1 PRIOR TO CONSTRUCTION OF THE CULVERT.

EMBANKMENT QUANTITIES FOR INITIAL CONSOLIDATION AND SETTLEMENT SHOWN IN THE EARTHWORK QUANTITIES ARE SUBSIDIARY TO OTHER EARTHWORK ITEMS. MATERIAL FOR THE EMBANKMENT IS INCLUDED IN THE EXCAVATION QUANTITIES.

EXCAVATION REQUIRED FOR PLACING SELECT SOIL IS INCLUDED IN THE COMMON EXCAVATION QUANTITIES.

WHERE EASEMENTS ARE SHOWN ON RAILROAD RIGHT OF WAY, THE CONTRACTOR SHALL BE REQUIRED TO WORK AROUND AND NOT DISTURB THE RAILROAD COMMUNICATION OR SIGNAL POLES OR LINES.

EXCAVATION SHOWN TO BE WASTED SHALL BE WASTED ON SITES PROVIDED BY THE CONTRACTOR. THESE SITES SHALL BE APPROVED BY THE ENGINEER AS TO SUITABILITY, APPEARANCE, AND SITE LOCATION. LOCATIONS THAT, IN THE OPINION OF THE ENGINEER, WILL LEAVE AN UNSIGHTLY APPEARANCE WILL NOT BE APPROVED.

ALL TREES, HEDGE ROWS, SHELTERBELTS, AND WOODY SHRUBS NOT SHOWN TO BE REMOVED AND LOCATED BETWEEN THE CONSTRUCTION LIMITS AND THE RIGHT-OF-WAY LINE OR EASEMENT LINE SHALL BE SPARED UNLESS DIRECTED BY THE ENGINEER TO BE REMOVED. ALL TREES WINTHIN THE APPROPRIATE CLEAR ZONE SHALL BE REMOVED.

Project Survey Control
Project Coord. x 0.99992054= NGS State Plane Coord. NAD 83
Vertical Datum: Sea Level Datum of NAVD 1988
Datum Bench Mark: BM 176 Elev.= 938.95

BM 176 (Johnson County Vertical Control Network):
2" Aluminum Bernsten CD-2 Monument Stamped "BM 176" on the southeast corner of bridge abutment located at the southeast corner of Mastin and Shawnee Mission Parkway. Travel west 0.5 miles from the intersection of I-35 and Shawnee Mission Parkway to the intersection of Mastin

Utility Owner Information

—G— *Kansas Gas Service*
Lyn Leet
11404 W.89th St.
Overland Park, KS 66214
913-599-8940

—W— *Johnson Co. Water District 1*
Jan Hardie
10747 Renner Boulevard
Lenexa, KS 66219
913-895-5776

— — — *Kansas City Power and Light*
Randy Jamison
P.O. Box 418679
Kansas City, MO 64141-9679
816-245-3727

═ ═ ═ *Johnson Co. Wastewater*
Charles McAllister
1148 Nall Avenue
Mission, KS 66201
913-715-8537

Horizontal Control Point #100 (Elev.=934.00)
Project Coordinates: N= 265602.94 E= 2249152.23

1.	Set 0.5" iron bar with GBA Cap at S.E. corner of Mastin and 62nd St. Terr.		
2.	South face of Power Pole.	7.7 ft	N.N.E.
3.	Back of Curb.	29.2 ft	N.
4.	Centerline of Sanitary Sewer Manhole.	40.5 ft	N.W.

Horizontal Control Point #101 (Elev.=939.67)
Project Coordinates: N= 265348.70 E= 2248881.43

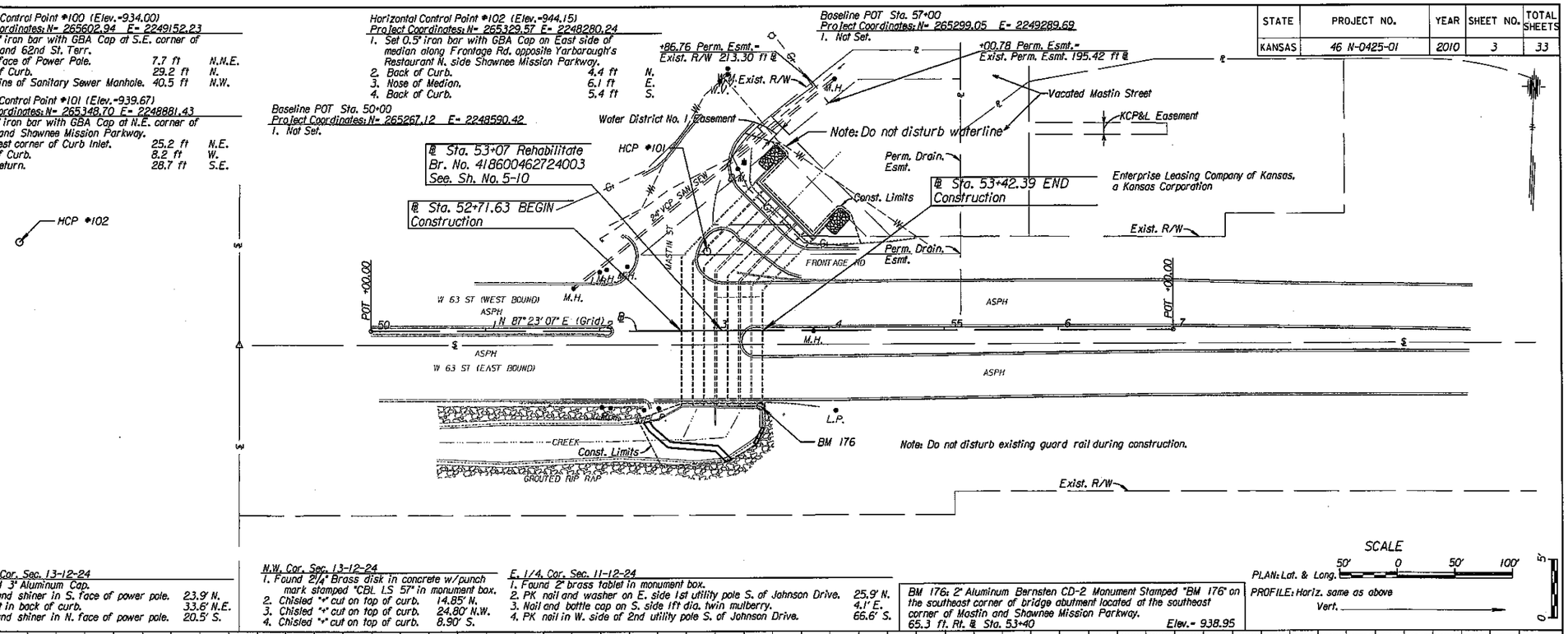
1.	Set 0.5" iron bar with GBA Cap at N.E. corner of Mastin and Shawnee Mission Parkway.		
2.	Northwest corner of Curb Inlet.	25.2 ft	N.E.
3.	Back of Curb.	8.2 ft	W.
4.	Curb Return.	28.7 ft	S.E.

Horizontal Control Point #102 (Elev.=944.15)
Project Coordinates: N= 265329.57 E= 2248280.24

1.	Set 0.5" iron bar with GBA Cap on East side of median along Frontage Rd. opposite Yarbarough's Restaurant N. side Shawnee Mission Parkway.		
2.	Back of Curb.	4.4 ft	N.
3.	Nose of Median.	6.1 ft	E.
4.	Back of Curb.	5.4 ft	S.

Baseline POT Sta. 50+00
Project Coordinates: N= 265267.12 E= 2248590.42
1. Not Set.

Baseline POT Sta. 57+00
Project Coordinates: N= 265299.05 E= 2249289.69
1. Not Set.



W. 1/4 Cor. Sec. 13-12-24
1. Found 3" Aluminum Cap.

2.	Nail and shiner in S. face of power pole.	23.9' N.
3.	"+" cut in back of curb.	33.6' N.E.
4.	Nail and shiner in N. face of power pole.	20.5' S.

N.W. Cor. Sec. 13-12-24
1. Found 2 1/4" Brass disk in concrete w/punch mark stamped "CBL LS 57" in monument box.

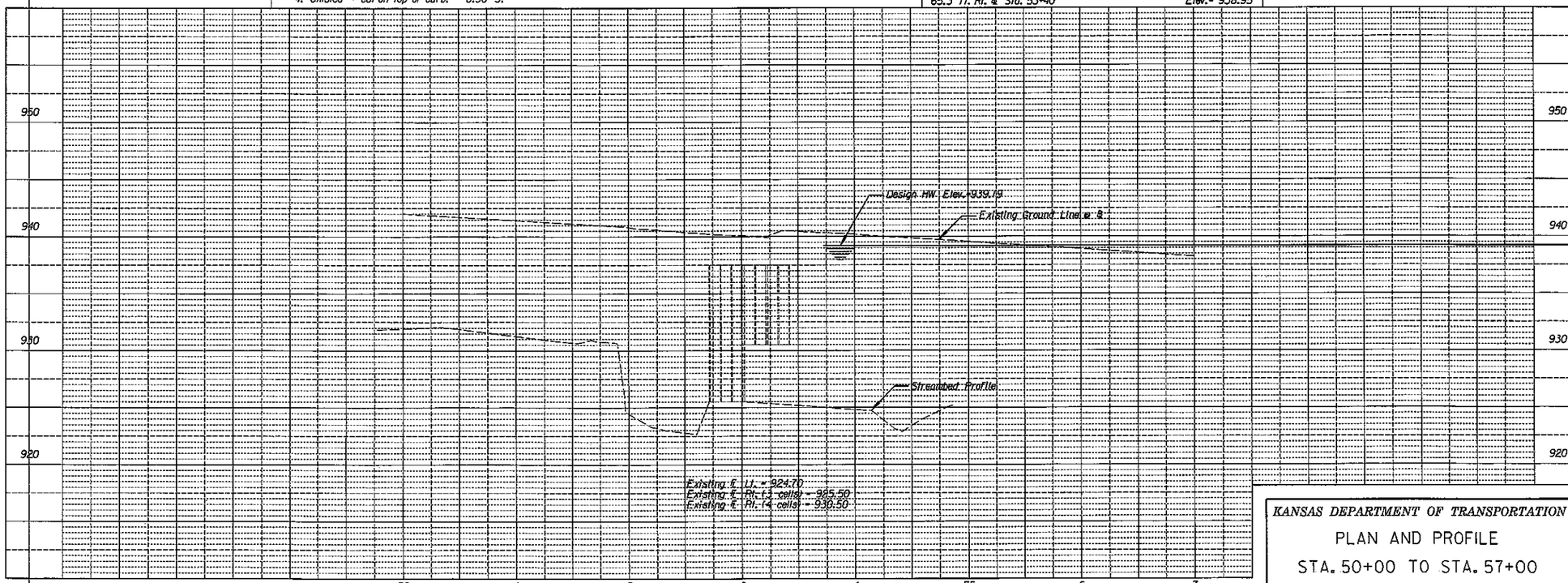
2.	Chisled "+" cut on top of curb.	14.85' N.
3.	Chisled "+" cut on top of curb.	24.80' N.W.
4.	Chisled "+" cut on top of curb.	8.90' S.

E. 1/4 Cor. Sec. 11-12-24
1. Found 2" brass tablet in monument box.

2.	PK nail and washer on E. side 1st utility pole S. of Johnson Drive.	25.9' N.
3.	Nail and bottle cap on S. side 1ft dia. twin mulberry.	4.1' E.
4.	PK nail in W. side of 2nd utility pole S. of Johnson Drive.	66.6' S.

BM 176: 2" Aluminum Bernsten CD-2 Monument Stamped "BM 176" on the southeast corner of bridge abutment located at the southeast corner of Mastin and Shawnee Mission Parkway.
65.3 ft. Rt. @ Sta. 53+40 *Elev.= 938.95*

SCALE
50' 0 50' 100'
PLAN: Lat. & Long.
PROFILE: Horiz. same as above
Vert.



KANSAS DEPARTMENT OF TRANSPORTATION
PLAN AND PROFILE
STA. 50+00 TO STA. 57+00

BY	DATE
C. LUEDKE	12/1/08
C. LUEDKE	5/26/09

REFERENCES NOTED
REFERENCES CHECKED

Drawn By : cluedke Plotted : 10/28/2009
File : G:\11694\MicroHighways\116942701.dgn

3

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	4	33

SITE DATA							HYDROLOGY				HYDRAULICS	
DRAINAGE STRUCTURE NUMBER	LOCATION STATION	DRAINAGE AREA NUMBER	DRAINAGE AREA ACREAGE	RECURRENCE INTERVAL YEARS	A. H. W. ELEVATION FEET	REMARKS	METHOD	DATA	Q_{cfs}	REMARKS	H. W. ELEVATION FEET	REMARKS
	53+07	1	1747	50	939.50	Road overtopping at Sta. 53+62	HEC-RAS 3.1.3	Tc= 55 min, Tlag)= 25 min, CN=76	6013	Turkey Creek Johnson County Watershed Study	939.19	Inlet Control. Design H.W. Elev. located Sta. 53+62

Drawn By : clsedke Plotted : 10/28/2009
File : G:\11694\Micro\Highways\drainagedatasht.dgn

NO.	DATE	REVISIONS	BY	APP'D
2	10-8-90	Detailed on CADD, no change	R.J.S.	J.O.B.
1	5-29-80	Assigned standard number	W.L.H.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

DRAINAGE DATA SHEET

~~RD653~~

FHWA APPROVAL	01-17-91	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED B.N.B.
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. R.J.S.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	5	33

Revised Plans

GENERAL NOTES

CONSTRUCTION SPECIFICATIONS: All work shall be in accordance with the current edition of the Standard Specifications for State Road and Bridge Construction, Kansas Department of Transportation.

LOW AREA: This item shall consist of all labor, materials and equipment necessary to fill the area indicated on the "Construction Layout" sheet and shall be included in the bid item "Concrete (Grade 4.0)(AE)".

Clean and power wash the limestone under the Low Area as required to remove organic material. This shall be subsidiary to the bid item "Concrete (Grade 4.0)(AE)"

The top surface shall be sloped at a min of 4% towards the center of the channel and shall be roughened to produce 1" tall protrusions with a maximum of 3" clear between protrusions.

RETAINING WALL (MODULAR CONCRETE): This item shall consist of all labor, materials and equipment necessary to install 2'-6" (H) x 5'-0" (L) x 2'-6" (W) modular concrete blocks. The concrete blocks shall have an approved interlocking system and shall be fabricated from Concrete (Grade 2.5)(AE) and shall be paid for per square foot of wall face.

The leveling pad shall be constructed from Concrete (Grade 4.0) (AE). All material, labor and equipment necessary to construct the leveling pad and remove, repair and/or replace grouted riprap, shall be subsidiary to the bid item "Retaining Wall (MBW)."

WEIR WALL: This item shall consist of all labor, materials and equipment necessary to construct the weir wall as detailed on the "Weir Wall Plan and Details" sheet and shall be included in the bid item "Concrete (Grade 4.0)(AE)". All reinforcing steel required in the weir wall shall be paid for under the bid item " Reinforcing Steel (Grade 60)".

Clean and power wash area where weir wall is to be installed as required to remove all organic material and roughen existing limestone prior to placing concrete. This shall be subsidiary to the bid item "Concrete (Grade 4.0)(AE)".

The horizontal and sloped surfaces of the weir wall shall be roughened to produce 1" tall protrusions with a maximum of 3" clear between protrusions.

CORE DRILLING: This item shall consist of all labor, material and equipment necessary to core drill the fill holes and install the casing and cap and shall be subsidiary to the bid item "Concrete (Grade 4.0)(AE)".

CONCRETE: Bevel all exposed edges of all concrete with a 3/4-inch triangular molding, except where noted on the plans.

SELF CONSOLIDATING CONCRETE: All concrete designated as self consolidating concrete shall meet "Section 401" of the Standard Specifications "AND" the following requirements:

Quality Assurance:

A pre-job conference will be held at the jobsite to discuss site specific conditions. Those present shall include the Ready Mix Supplier, the Contractor, the Engineer and the Testing Agency. No self consolidating concrete placement will be allowed prior to this meeting.

The self consolidating concrete manufacturer shall make available a qualified individual to monitor concrete quality and performance as specified.

Slump Flow Characteristics:

The self consolidating concrete slump flow characteristics required shall be determined by the manufacturer and shall be included in the mix design.

The slump flow characteristics shall be measured per ASTM C1611.

Mockup:

The General Contractor, Concrete Sub-Contractor, Ready-Mix Supplier and Testing Agency shall submit a mock up procedure, that simulates field conditions as closely as possible, to the engineer for approval. Once approved, the concrete shall be placed and tested according to the approved procedure. No self consolidating concrete placement will be allowed until the mockup is placed, reviewed and approved by the Engineer.

Surface Preparation:

Immediately before concrete placement, thoroughly wet moisture-absorbing material that will be in contact with the concrete, without developing standing water. Ensure that concrete steel forms and rebars are not steaming hot prior to pouring concrete.

REINFORCING STEEL: All reinforcing steel dimensions are to the centerline of bars unless otherwise noted. All reinforcing steel shall conform to the requirements of ASTM A615, Grade 60.

EXISTING DIMENSION VERIFICATION: Verify, by field measurement, the as built dimensions of the existing structure and submit such verification in writing to the Engineer. The verification will include sketches, drawings, photographs and descriptions as needed to clearly define the as built dimensions that will be incorporated in the new construction.

CONCRETE SURFACE REPAIR: Remove loose and unsound wall and top slab concrete as designated by the Engineer to whatever depth is necessary to reach sound concrete. See KDOT Specifications.

All materials removed from the existing structure shall become the property of the Contractor and removed from the site.

AREA PREPARED FOR PATCHING: Remove loose and unsound bottom slab concrete as designated by the Engineer to whatever depth is necessary to reach sound concrete. See KDOT Specifications.

All materials removed from the existing structure shall become the property of the Contractor and removed from the site.

BROKEN CONCRETE: Waste broken concrete and debris removed from the existing structure on sites provided by the Contractor and approved by the Engineer.

UTILITIES: Field verify locations of all utilities adjacent to the structure prior to the start of construction. Care shall be taken to avoid damage to existing utilities or their conduit encasements. Damage to existing utilities shall be repaired at the Contractor's expense.

QUANTITIES: Items not listed separately in the Summary of Quantities are subsidiary to other items in the proposal.

DIMENSIONS: All dimensions shown on the design plans are horizontal dimensions unless otherwise noted. Make necessary allowances for roadway grade and cross slope.

⚠1 SHOT-CRETE: This item shall consist of all labor, materials and equipment necessary to fill the areas indicated on the "Weir Wall Plan and Details" sheet and shall be paid for per square feet as "Concrete Surface Repair".

Tree removal as shown on the plan is included in the bid item "Clearing and Grubbing".

RECAPITULATION OF BRIDGE QUANTITIES		
ITEM	QUANTITY	UNIT
Concrete (Grade 4.0)(AE)	142.1	cu.yd.
Reinforcing Steel (Grade 60)	14600	lbs.
Reinforcing Steel (Repair)(Grade 60)(Set Price)	1	lbs.
Area Prepared for Patching (Partial Depth)	10	sq.yd.
Area Prepared for Patching (Full Depth)	5	sq.yd.
⚠1 Concrete Surface Repair	~~170~~ 933	sq.ft.
Riprap (Heavy Stone) (1/4 ton)	55	cu.yd
Bedding for Slope Protection	18	cu.yd.
Retaining Wall (MBW)	113	sq.ft.
⚠1 ~~Shot Crete~~	~~763~~ 0	~~sq.ft.~~

DESIGN DATA:

DESIGN SPECIFICATIONS:
AASHTO Specifications, 2002 Edition and latest Interim Specifications. Load Factor Design

DESIGN LOADING:
HS20-44

UNIT STRESSES:
Concrete (Grade 2.5)(AE) f'c = 2.5 ksi
Concrete (Grade 4.0)(AE) f'c = 4 ksi
Reinforcing Steel(Grade 60) fy = 60 ksi

CONSTRUCTION SEQUENCE

These plans have been developed based on a sequence of construction for completing the RCB repairs as follows:

1. Patch the delaminated areas of top slab, bottom slab and walls of all cells, full length of the existing RCB.
2. Core Fill Holes through the pavement at the spacing shown to allow placement of self consolidating concrete for the Top Slab Repair. Spacing of fill holes shown is based on information from area self consolidating concrete producers and contractors to meet traffic handling constraints and provide a quality finished product.
   a. Fill holes to be cased and capped during construction to prevent loss of material into the RCB and debris from falling into top slab formwork.
   b. After completion of Top Slab Repairs, fill hole casings will be removed to 12" below finish pavement and holes will be patched.
3. Construct the 6" wall in the east cell of the RCB prior to placement of the top slab in that cell.
4. Prepare reinforcing and formwork/falsework for the completion of Top Slab Repairs.
5. Construct the Top Slab Repairs - to be completed in one night of work. See Construction Sequencing and Traffic Control Plans for permissible lane closures

CONCRETE PLACING SEQUENCE: The Contractor shall submit a placing sequence for review, see Construction Sequencing and Traffic Control Plans for permissible lane closures. Submit the placing sequence to the Engineer at the Preconstruction Conference. Include the proposed rate of concrete placement in C.Y./h, the plant capacity, placement direction, construction joint location, a description of the equipment used in placing the concrete, proposed admixtures, and the quantity of concrete in each placing segment. Any additional cost for the Contractor's plan of placing concrete, including admixtures, shall be at the Contractor's expense and shall be considered subsidiary to the bid item "Concrete (Grade 4.0) (AE)". Approval of the Contractor's sequence is required prior to placement of concrete in the slab.

MIN. REBAR SPLICE LENGTHS	
Bar Size	Splice
#4	1'-4"
#5	1'-8"
#6	2'-0"

⚠1 5/04/10 Removed Shot-Bid Item. Added quantity to Concrete Surface Repair.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION
Br.No. 418600462724003 Sta. 53+07.00

GENERAL NOTES, QUANTITIES AND CONSTRUCTION SEQUENCING

PROJECT NO.: 46 N-0425-01 JOHNSON COUNTY

SHEET NO. OF	SCALE	APP'D	
DESIGNED NWJ	DETAILED MAR	QUANTITIES NWJ	CADD MAR
DESIGN CK. MED	DETAIL CK. MED	QUAN. CK. MED	CADD CK. MED

Std. Base File:
Plotted By: dlueske | Plot Locations: $UNIT$
File: G:\1694-MicroBridge\1694_01_Quan.dgn
Plot Date: 5/4/2010

Horizontal Control Point #100 (Elev.=934.00)
Project Coordinates: N= 265602.94 E= 2249152.23
1. Set 0.5" iron bar with GBA Cap at S.E. corner of Mastin and 62nd St. Terr.
2. South face of Power Pole. 7.7 ft N.N.E.
3. Back of Curb. 29.2 ft N.
4. Centerline of Sanitary Sewer Manhole. 40.5 ft N.W.

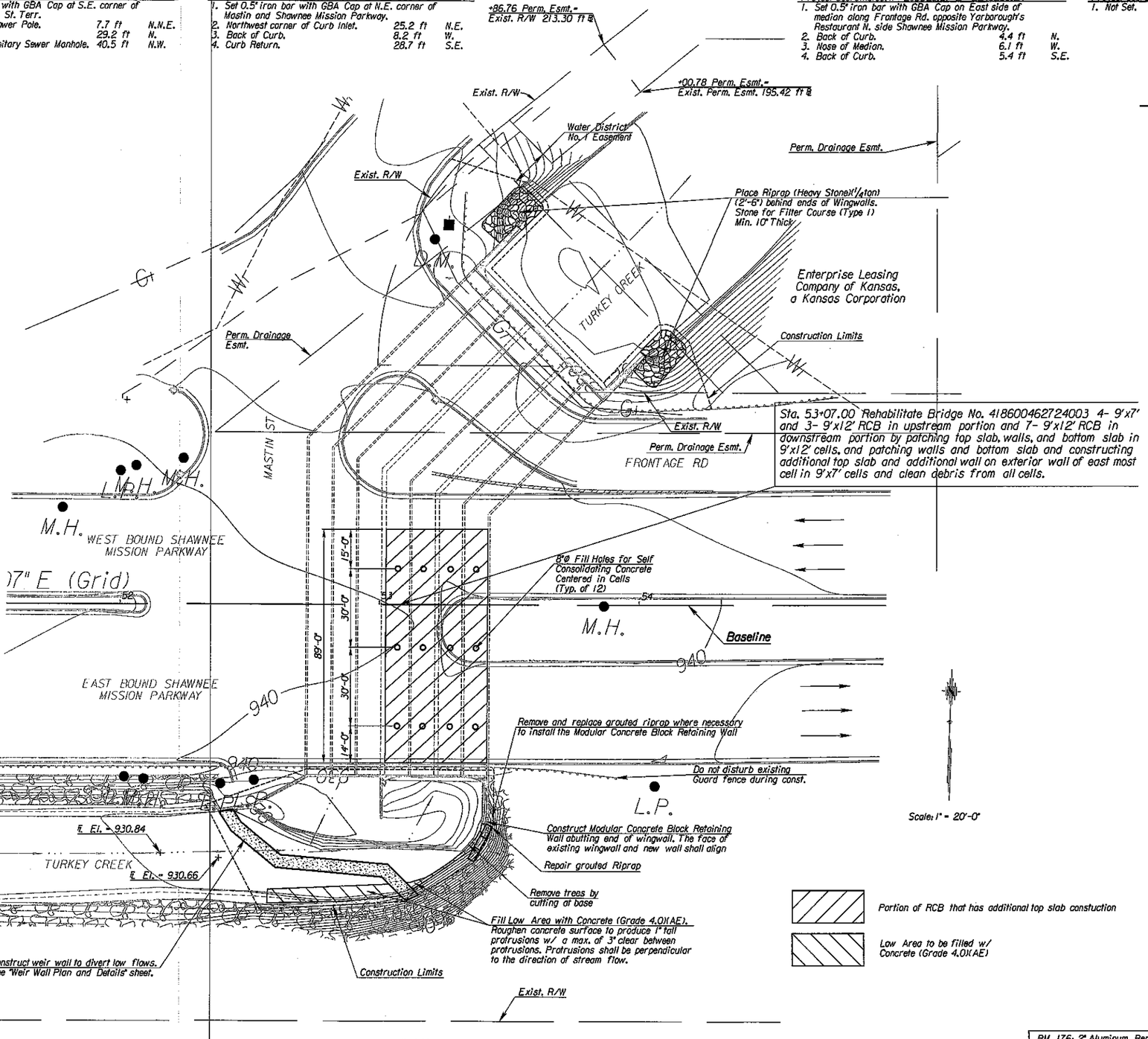
Horizontal Control Point #101 (Elev.=939.67)
Project Coordinates: N= 265348.70 E= 2248881.43
1. Set 0.5" iron bar with GBA Cap at N.E. corner of Mastin and Shawnee Mission Parkway.
2. Northwest corner of Curb inlet. 25.2 ft N.E.
3. Back of Curb. 8.2 ft W.
4. Curb Return. 28.7 ft S.E.

Horizontal Control Point #102 (Elev.=944.15)
Project Coordinates: N= 265329.57 E= 2248280.24
1. Set 0.5" iron bar with GBA Cap on East side of median along Frontage Rd. opposite Yarborough's Restaurant N. side Shawnee Mission Parkway.
2. Back of Curb. 4.4 ft N.
3. Nose of Median. 6.1 ft W.
4. Back of Curb. 5.4 ft S.E.

Baseline POT Sta. 50+00
Project Coordinates: N= 265267.12 E= 2248590.42
1. Not Set.

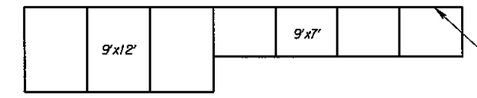
Baseline POT Sta. 57+00
Project Coordinates: N= 265299.05 E= 2249289.69
1. Not Set.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	6	33



PLAN

Scale: 1" = 20'-0"



UPSTREAM ELEVATION VIEW OF EXISTING RCB

Existing waterway opening = 576 ft²
Proposed waterway opening = 540 ft²

Sta. 53+07.00 Rehabilitate Bridge No. 418600462724003 4- 9'x7' and 3- 9'x12' RCB in upstream portion and 7- 9'x12' RCB in downstream portion by patching top slab, walls, and bottom slab in 9'x12' cells, and patching walls and bottom slab and constructing additional top slab and additional wall on exterior wall of east most cell in 9'x7' cells and clean debris from all cells.

Portion of RCB that has additional top slab constuction

Low Area to be filled w/ Concrete (Grade 4.0)(AE)

TURKEY CREEK TRIBUTARY HYDRAULIC DATA

Drainage Area	2.73 sq. mi.
Discharge Frequency	50 Yr.
Design Discharge (Q_{des})	601.3 cfs
Design High Water Elevation	939.19 ft.
Overtopping Elevation (Sta. 53+62)	939.50 ft.
Overtopping Discharge	6500 cfs
Overtopping Frequency	120 Yr.
Historic High Water Elevation	N/A
Design Velocity	6.09 ft./sec.

BM 176: 2" Aluminum Bernsten CD-2 Monument Stamped "BM 176" on the southeast corner of bridge abutment located at the southeast corner of Mastin and Shawnee Mission Parkway.
65.3 ft. Rt. @ Sta. 53+40 Elev.= 938.95

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION
Br.No. 418600462724003 Sta. 53+07.00

CONSTRUCTION LAYOUT

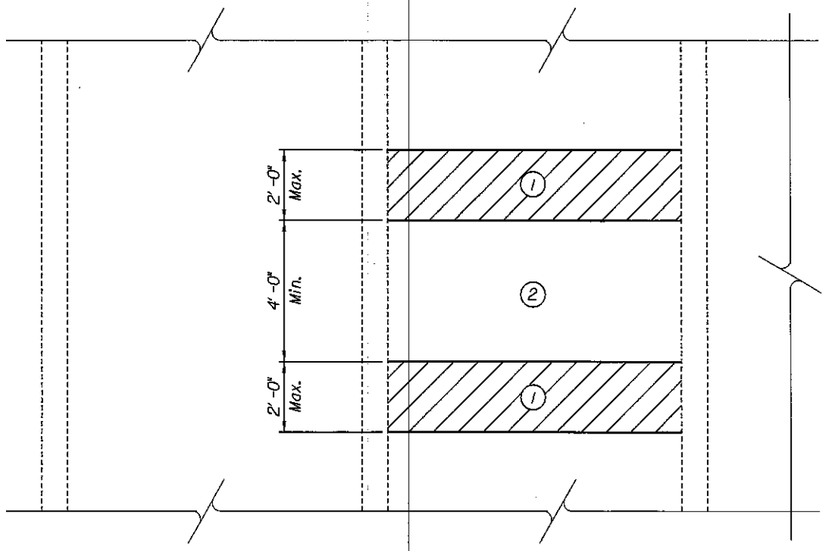
PROJECT NO.: 46 N-0425-01 JOHNSON COUNTY

SHEET NO.	OF	SCALE		APP'D	
DESIGNED	NWJ	DETAILED	MAR	QUANTITIES	NWJ
DESIGN CK.	MED	DETAIL CK.	MED	QUAN. CK.	MED

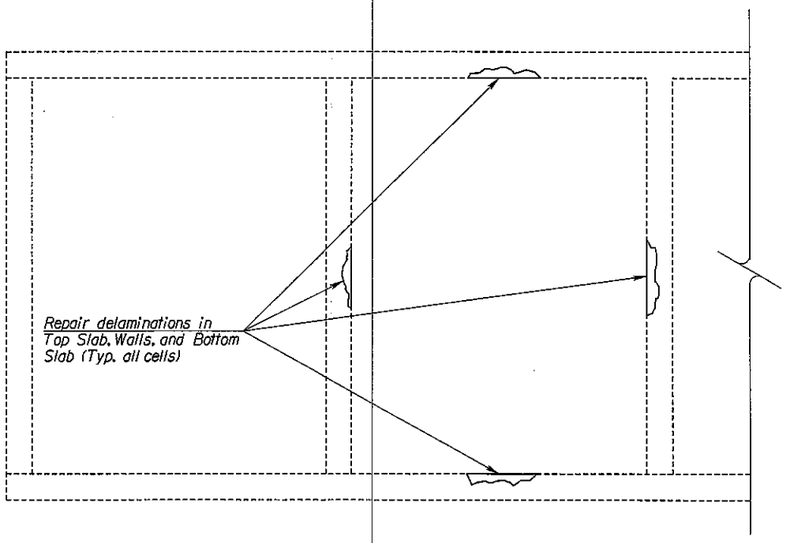
CADD MAR
CADD CK. MED

Std. Base File:
Plotted By: dbeahe | Plot Location: $UNIT$
File: GV1694\Micro\Bridge\1694_02_Con_Lay.dgn
Plot Date: 3/26/2010

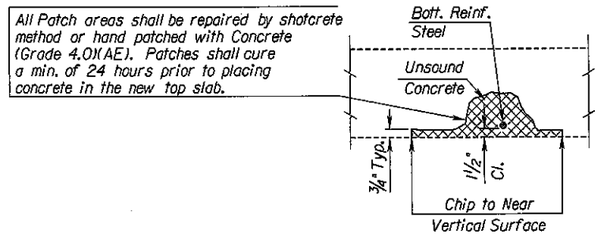
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	7	33



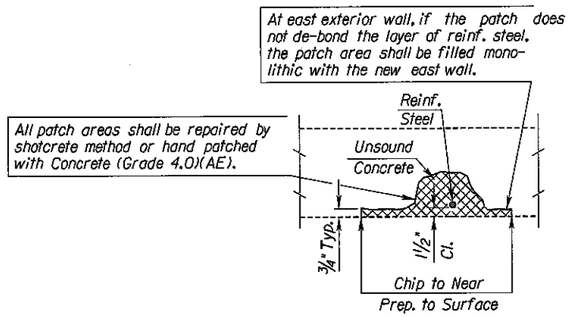
PART PLAN SHOWING SLAB & WALL PATCHING SEQUENCE



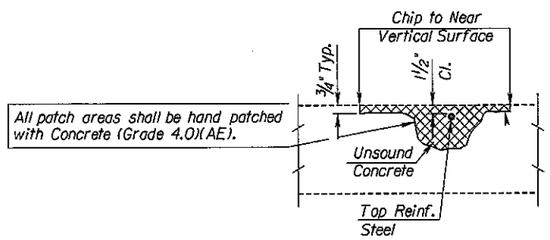
PART SECTION SHOWING PATCHING



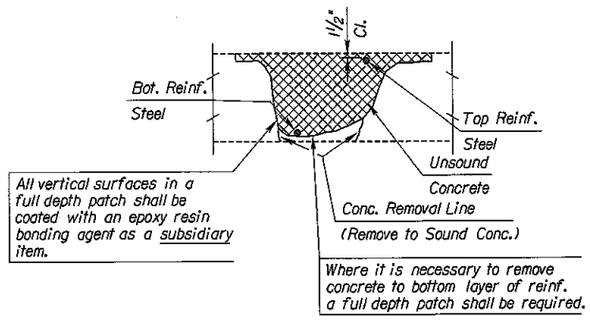
TOP SLAB PATCHING DETAIL



WALL PATCHING DETAIL



PARTIAL DEPTH FLOOR PATCH



FULL DEPTH FLOOR PATCH

GENERAL NOTES

PATCHING SEQUENCE: The concrete removal shall be completed in stages. Concrete removed in one cell of the RCB shall be patched prior to beginning concrete removal in adjacent cells. Maximum width along centerline of RCB of concrete removal with debonded reinforcing shall be 2'-0" (Area ①) with a minimum 4'-0" width (Area ②) between open areas (Area ①). Concrete removal shall not begin in Area ② until the patching concrete in adjacent Area ① has cured a minimum of 3 days.

CONCRETE SURFACE REPAIR: This item shall consist of removing unsound concrete and patches from the top slab & walls of the RCB, cleaning reinforcing bars, and filling the removed patched areas with concrete. Quantity shown is an estimate of the areas involved. The exact areas shall be determined by tapping, before, during and after chipping operation to assure that all unsound concrete has been removed. See KDOT Specifications.

The areas to be patched above the new top slab construction shall be determined by the Engineer. Areas showing distress but still capable of staying intact during construction of the new top slab need not be patched.

AREA PREPARED FOR PATCHING: This item shall consist of removing unsound concrete and patches from the bottom slab of the RCB, cleaning reinforcing bars, and filling the removed patched areas with concrete. Quantity shown is an estimate of the areas involved. The exact areas shall be determined by tapping, before, during and after chipping operation to assure that all unsound concrete has been removed. See KDOT Specifications.

FULL DEPTH PATCHING: See KDOT Specifications for method of measurement and basis of payment.

REINFORCING: Care should be exercised to prevent cutting, stretching or damaging exposed reinforcing steel. Extreme care should be exercised to avoid breaking the bond between the reinforcing steel and concrete where bars are partially exposed yet remain anchored in sound concrete. Reinforcing steel damaged, cut or deteriorated shall be replaced as directed by the Engineer. See table on General Notes sheet for minimum splice length required. Replacement of bars damaged by the Contractor shall be done so at the Contractors expense.

SUMMARY OF PATCHING QUANTITIES		
ITEM	QUANTITY	UNIT
Reinforcing Steel (Repair)(Grade 60)(Set Price)	1	lbs.
Area Prepared for Patching (Partial Depth)	10	sq.yd.
Area Prepared for Patching (Full Depth)	5	sq.yd.
Concrete Surface Repair	170	sq.ft.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br.No. 418600462724003 Sta. 53+07.00

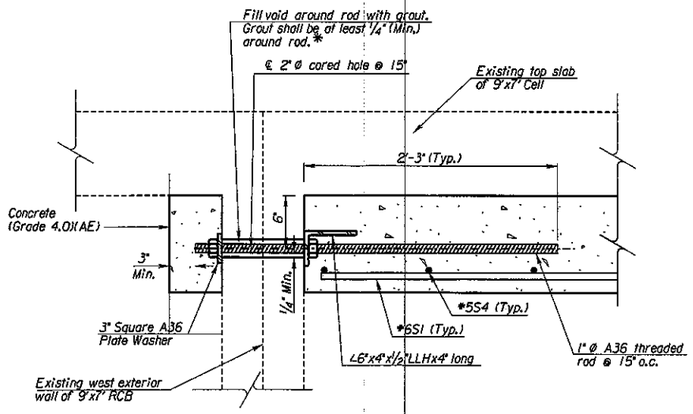
RCB REPAIR PATCHING DETAILS

PROJECT NO.: 46 N-0425-01 JOHNSON COUNTY

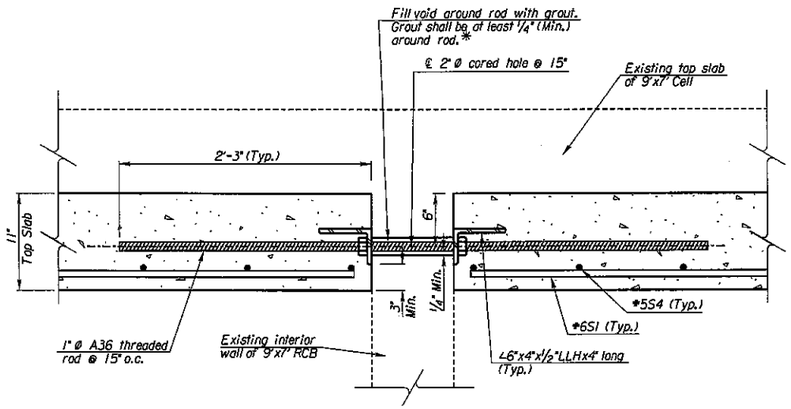
SHEET NO.	OF	SCALE		APP'D			
DESIGNED	NWJ	DETAILED	MAR	QUANTITIES	NWJ	CADD	MAR
DESIGN CK.	MED	DETAIL CK.	MED	QUAN. CK.	MED	CADD CK.	MED

Std. Base File:
Plotted By: dluedke | Plot Locations: $UNIT$
File: GA1694MicroBridge\1694_03_Repair_Details.dgn
Plot Date: 3/26/2010

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	8	33



DETAIL A



DETAIL B

Note:
All cored holes, threaded rods, nuts and angles shall be subsidiary to the bid item "Concrete (Grade 4.0)(AE)".

* Contractor shall submit a detailed grouting procedure for approval by the Engineer. The procedure shall address how grout will be placed, how proper filling will be verified, and how 1/4" (Min.) clearance around the rod will be maintained and shall be subsidiary to the bid item "Concrete (Grade 4.0)(AE)".

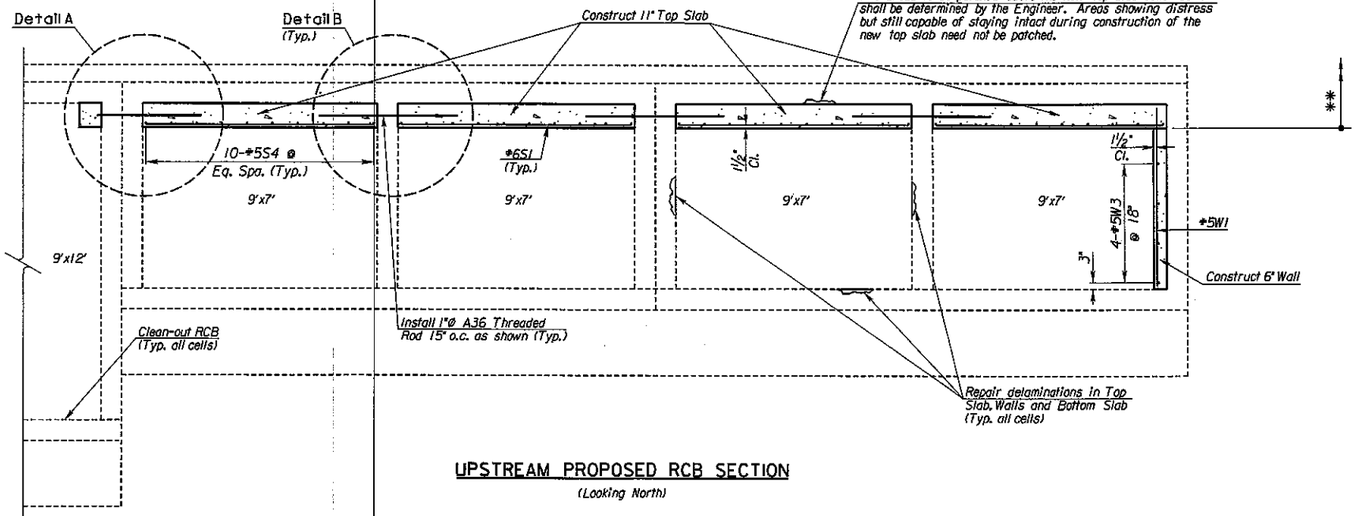
** Concrete in top slab shall be self consolidating concrete. See "General Notes, Quantities and Construction Sequencing" sheet for additional requirements.

See table on General Notes sheet for minimum splice length required.

SUMMARY OF TOP SLAB REPAIR QUANTITIES		
ITEM	QUANTITY	UNIT
Wall & Misc. - Concrete (Grade 4.0)(AE)	10.3	cu.yd.
Top Slab - Concrete (Grade 4.0)(AE)	108.2	cu.yd.
Reinforcing Steel (Grade 60)	14040	lbs.

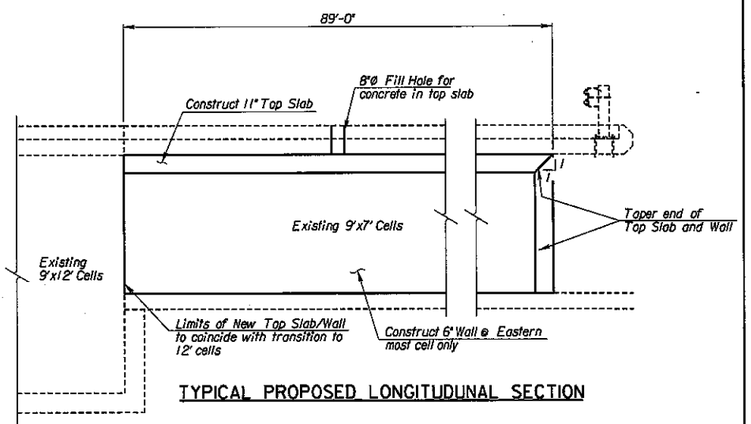
BAR SCHEDULE †						
S1				S4		
Size	Spa.	No.	Length	Size	No.	Length
6	6"	712	8'-8"	5	80	45'-2"
W1				W3		
Size	Spa.	No.	Length	Size	No.	Length
5	12"	90	6'-8"	5	8	45'-2"

† The Contractor may adjust the lengths of S4 & W3 bars, at no cost to the City, as necessary to install the reinforcing.



UPSTREAM PROPOSED RCB SECTION

(Looking North)



TYPICAL PROPOSED LONGITUDUNAL SECTION

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br.No. 418600462724003 Sta. 53+07.00

RCB TOP SLAB REPAIR DETAILS

PROJECT NO.: 46 N-0425-01 JOHNSON COUNTY

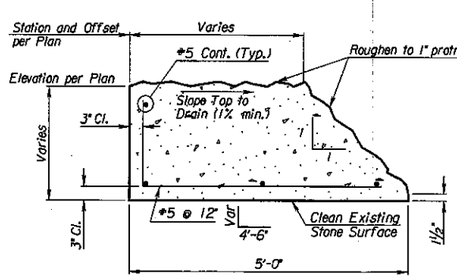
SHEET NO.	OF	SCALE		APP'D			
DESIGNED	NWJ	DETAILED	MAR	QUANTITIES	NWJ	CADD	MAR
DESIGN CK.	MED	DETAIL CK.	MED	QUAN. CK.	MED	CADD CK.	MED

Std. Base Files:
Plotted By: cluedke | Plot Location: $UNIT$
File: GV1694\MicroBridge\1694_04_Top_Slab_Details.dgn
Plot Date: 10/28/2009

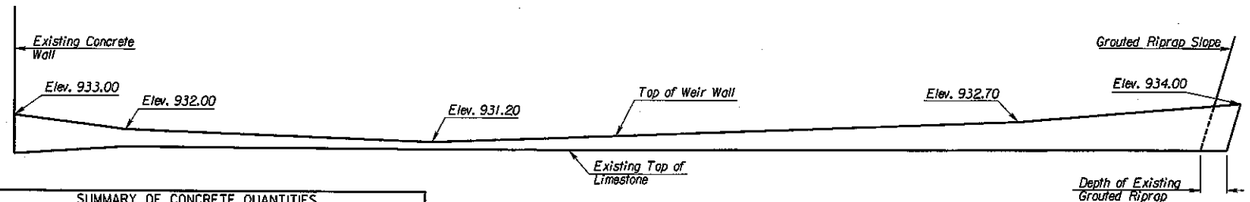
8

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	9	33

Revised Plans



TYPICAL WEIR SECTION



WEIR PROFILE

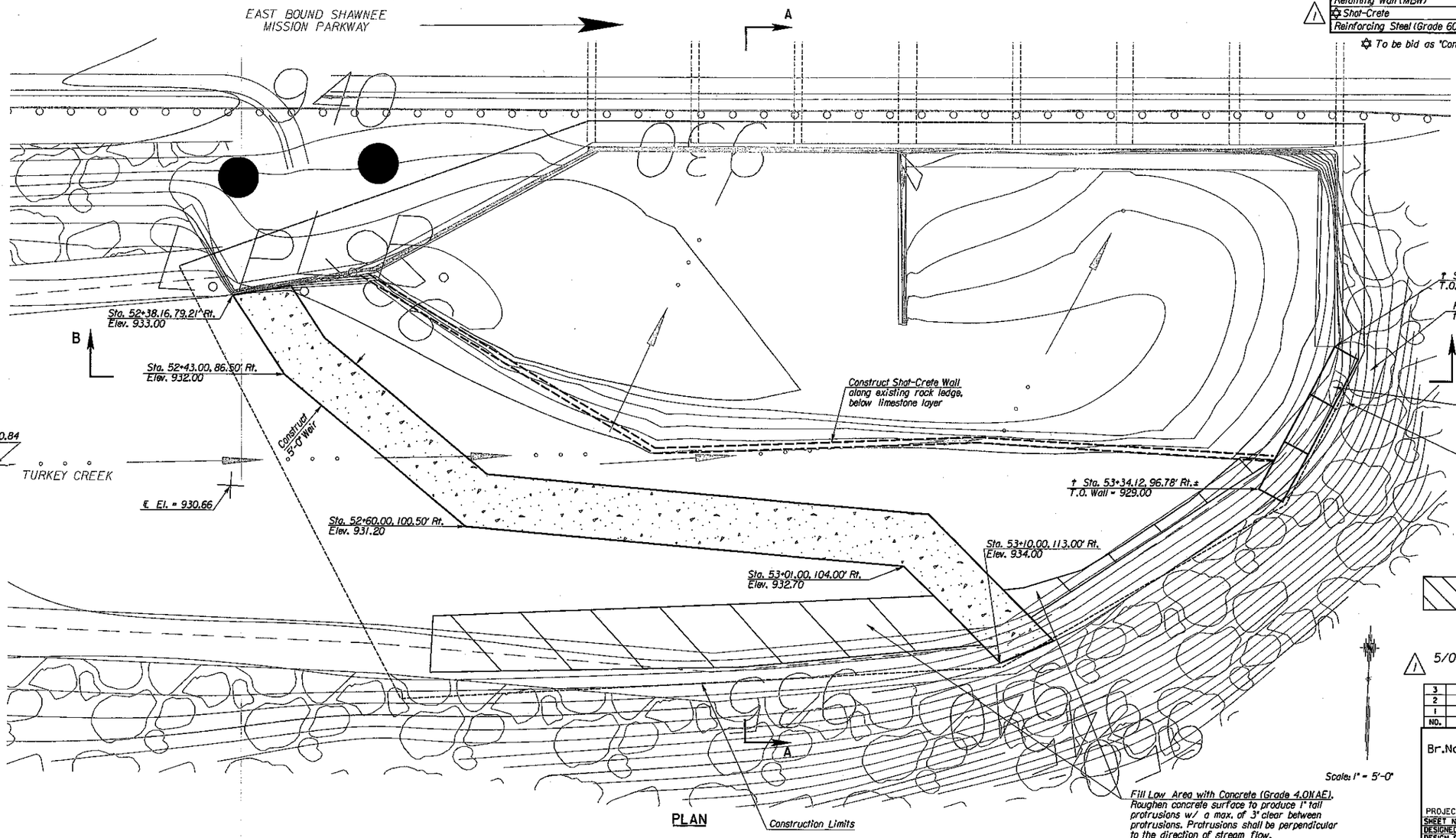
Notes:
Power wash the limestone under the Weir Wall as required to remove organic material, this shall be subsidiary to the bid item "Concrete (Grade 4.0)(AE)."

For Sections A-A and B-B, see "Miscellaneous Details Sheet".

SUMMARY OF CONCRETE QUANTITIES		
ITEM	QUANTITY	UNIT
Weir Wall	18.8	cu.yd.
Low Area	4.8	cu.yd.

SUMMARY OF CHANNEL QUANTITIES		
ITEM	QUANTITY	UNIT
Concrete (Grade 4.0)(AE)	23.6	cu.yd.
Retaining Wall (MBW)	113	sq.ft.
✡ Shot-Crete	763	sq.ft.
Reinforcing Steel (Grade 60)	560	lbs.

✡ To be bid as "Concrete Surface Repair".



PLAN

Scale: 1" = 5'-0"

1 5/04/10 Added note that Shot-Crete will be bid as Concrete Surface Repair.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br.No. 418600462724003 Sta. 53+07.00

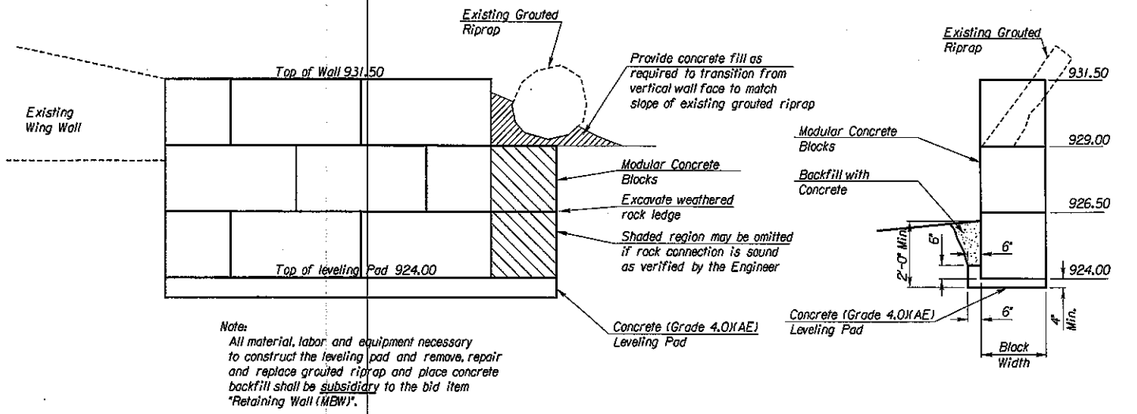
WEIR WALL PLAN AND DETAILS

PROJECT NO.: 46 N-0425-01 JOHNSON COUNTY

SHEET NO.	OF	SCALE		APP'D			
DESIGNED	NWJ	DETAILED	MAR	QUANTITIES	NWJ	CADD	MAR
DESIGN CK.	MED	DETAIL CK.	MED	QUAN. CK.	MED	CADD CK.	MED

Std. Base Files:
Plotted By: cluedke | Plot Location: $UNIT$
File: GV1694\MicroBridge\V1694_05_Weir_Con_Lay.dgn
Plot Date: 5/4/2010

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	10	33



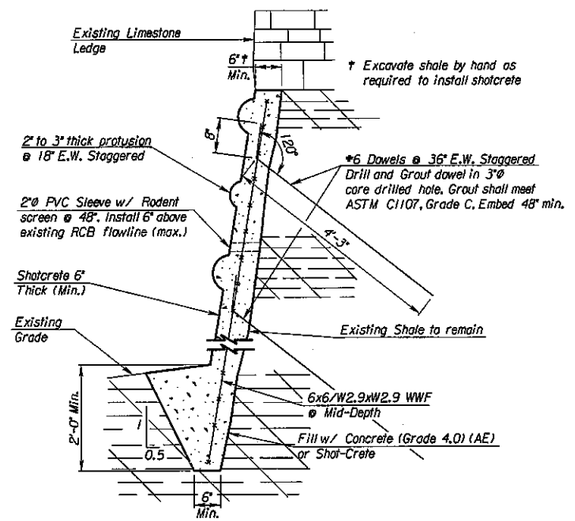
Note:
All material, labor and equipment necessary to construct the leveling pad and remove, repair and replace grouted riprap and place concrete backfill shall be subsidiary to the bid item "Retaining Wall (MBW)".

PROFILE OF RETAINING WALL (MODULAR BLOCK)

Scale: 3/8" = 1'-0"

END OF WALL ELEVATION

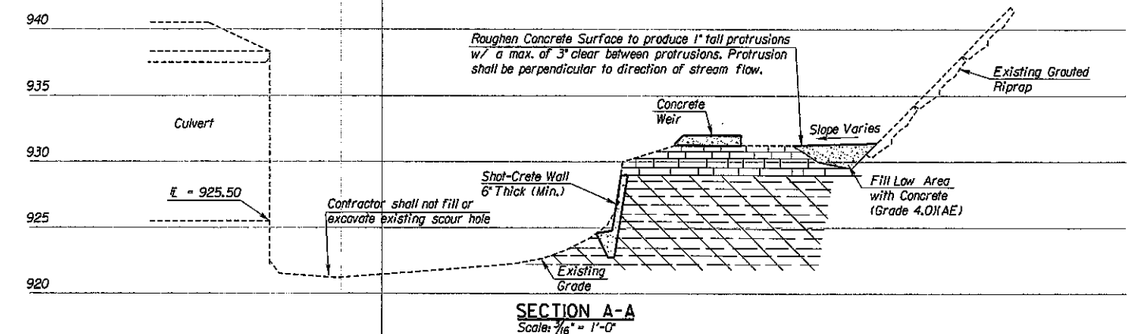
Scale: 3/8" = 1'-0"



TYPICAL SHOT-CRETE DETAIL

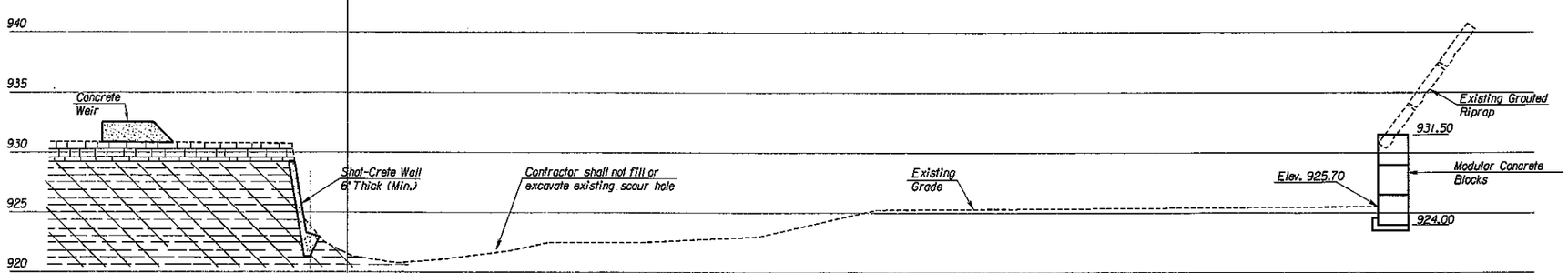
Scale: 3/4" = 1'-0"

Note:
All material, labor and equipment necessary to fabricate & core drill/grout reinforcing, excavate existing shale and fabricate/install PVC sleeve shall be subsidiary to the bid item "Shot-Crete".



SECTION A-A

Scale: 3/16" = 1'-0"



SECTION B-B

Scale: 3/16" = 1'-0"

Note:
Clear all debris and loose rock from RCB cells or obstructing the RCB openings. This shall be subsidiary to the bid item "Cleaning Existing Structures".

E.W. = Each Way

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br.No. 418600462724003 Sta. 53+07.00

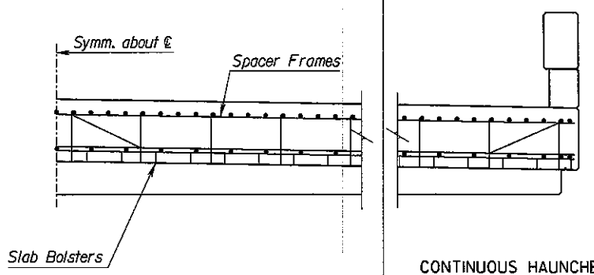
MISCELLANEOUS DETAILS

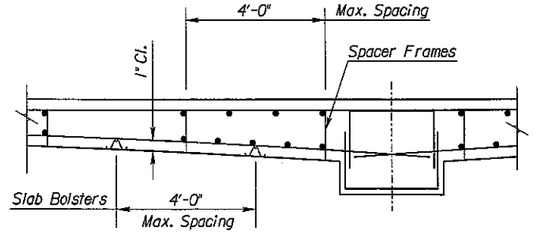
PROJECT NO.: 46 N-0425-01 JOHNSON COUNTY

SHEET NO. OF	SCALE		APP'D	
DESIGNED NWJ	DETAILED MAR		QUANTITIES NWJ	CADD MAR
DESIGN CK. MED	DETAIL CK. MED		QUAN. CK. MED	CADD CK. MED

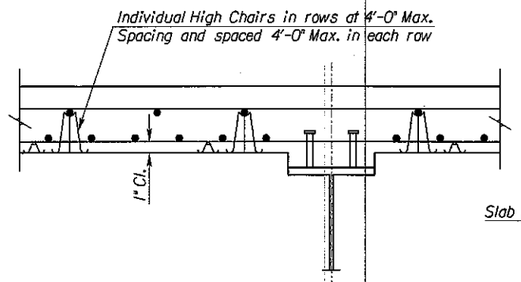
Std. Base File:
Plotted By: cluedke Plot Location: $UNIT$
File: C:\1694\Micro\Bridge\1694_06_Sections.dgn
Plot Date: 3/26/2010

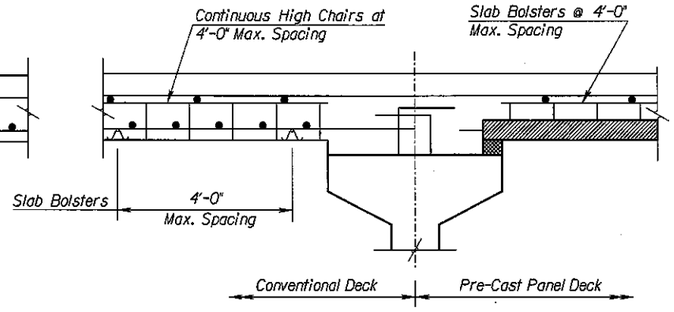
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	11	33



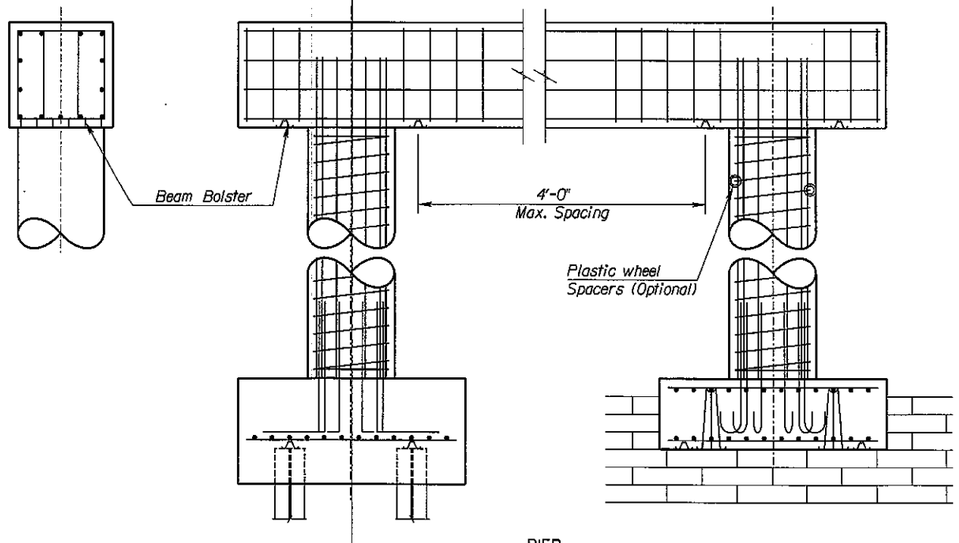


CONTINUOUS HAUNCHED SLAB

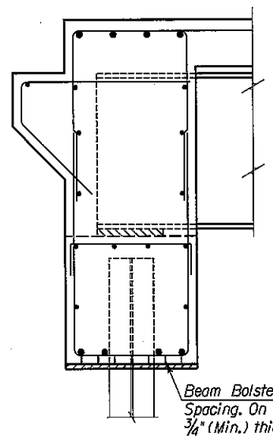




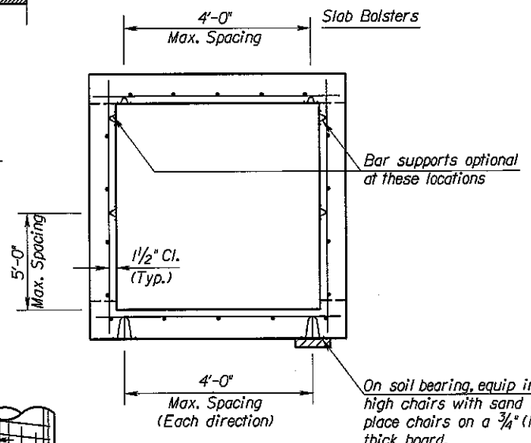
DECK GIRDERS



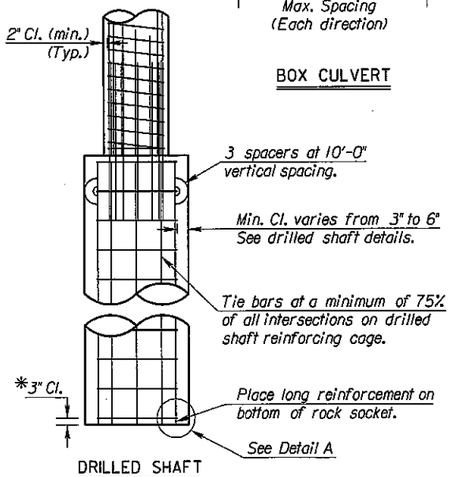
PIER



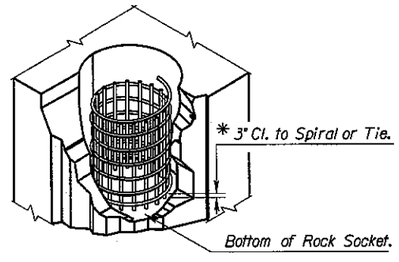
ABUTMENT



BOX CULVERT



DRILLED SHAFT



DETAIL A

* Note: Longitudinal reinforcing steel is placed on the bottom of the rock socket. Maintain 3" clearance from the bottom of rock socket to the first spiral or tie bar.

TYPICAL SECTION

Reference is made to the latest edition of the CRSI "Manual of Standard Practice" for recommended industry practices concerning reinforcing steel.

Use only the following types of bar supports:

1) Wire Bar Supports:

a) Epoxy coated reinforcing: Class I Protection
b) Non-epoxy coated reinforcing: Class 1, 2, or 3 Protection

2) Plastic Bar Supports

3) Supplementary bars

When securing epoxy coated reinforcement, use tie wires or metal clips that are epoxy or plastic coated.

Do not weld reinforcing steel to bar supports or to other reinforcing steel. Shop weld spacer frames for haunched slabs.

Tie bars at all intersections around the perimeter of each mat and at not less than 2'-0" centers or at every intersection, whichever is greater.

Where more than one length of bar support is required, lap the end legs so they are locked or tied together.

Use proper height supports to maintain the distance between the reinforcing and the formed surface or the top surface of deck slabs within 1/4" of that indicated on the plans.

Spacings shown are maximums. Use sufficient supports, as determined by the Engineer, to retain the reinforcing steel in position.

Construct any platforms, required for the support of workers and/or equipment during concrete placement, directly on the forms and not on the reinforcing steel.

Designs and arrangements of Supports or Spacers other than as shown on this sheet, may be used with the permission of the Engineer.

NO.	DATE	REVISIONS	BY	APP'D
4	12-01-05	Drilled Shaft, Spiral Steel Placement	JPJ	KEH
3	6-21-00	...Added Eco-Cost Excel Detail.	BAM	KEH
2	12-20-99	...Added Rounded Slab Bolsters...	BAM	KEH
1	12-09-99	...Revised Drilled Shaft Clearance	BAM	KEH

KANSAS DEPARTMENT OF TRANSPORTATION

SUPPORTS AND SPACERS FOR REINFORCING STEEL

BRI20

FHWA APPROVAL		12-23-05	APP'D	KENNETH E. HURST	
DESIGNED	BAM	DETAILED	GEG	QUANTITIES	CADD GEG
DESIGN CK.	LRB	DETAIL CK.	BAM	QUAN. CK.	CADD CK. BAM

Std. Base File: br120.dgn
Plotted By: cluedke
Plot Location: $UNIT$
File: G:\1694\MicroStation\Highways\Client Standards\br120.dgn
Plot Date: 10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	12	33

RECAPITULATION OF BRIDGE QUANTITIES

BRIDGE NUMBER	STATION	SEE SHEET NO.
41B600462724003	53+07	5

RECAPITULATION OF ROAD QUANTITIES

ITEM	QUANTITY	UNIT
Cleaning Existing Structures	L.S.	Lump Sum
Contractor Construction Staking	L.S.	Lump Sum
Field Office and Laboratory (Type C)	1	Each
Mobilization	L.S.	Lump Sum
Mobilization (DBE)	L.S.	Lump Sum
Clearing and Grubbing	L.S.	Lump Sum

For Seeding Quantities, See Sh. No. 13
For Temporary Water Pollution Control Quantities, See Sh. No. 14
For Traffic Control Quantities, See Sh. No. 32

NO.	DATE	REVISIONS	BY	APP'D
2	1-14-08	Rem. Drainage Structure summary	S.W.K.	J.O.B.
1	1-9-91	Detailed on CADD	R.J.S	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

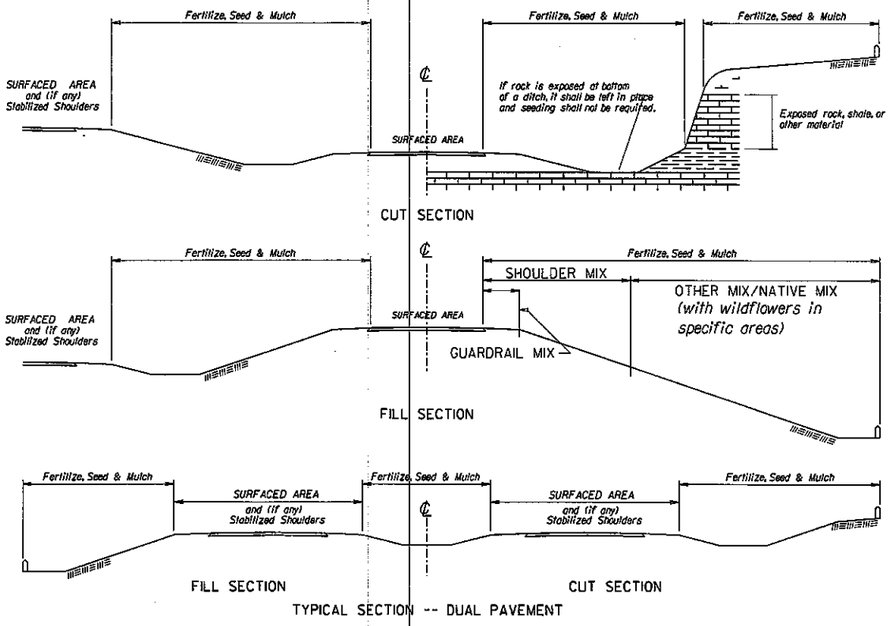
SUMMARY OF QUANTITIES

RD050

FHWA APPROVAL	5-28-08	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED B.N.B.
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. S.W.K.

Drawn By : clsedke Plotted : 10/28/2009
File : G:\11694\Micro\Highways\Client Standards\rd050.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	13	33



SEEDING PERIODS

COOL SEASON February 15 to April 20 and August 15 to Sept. 30	WARM SEASON November 15 to June 1
SPECIES	SPECIES
Bluegrasses	Bluestems
Bromegrasses	Buffalograss
Canada Wildrye	Eastern Gamagrass
Fescues	Gramas
Ryegrasses	Indiangrass
Wheatgrasses	Lovegrasses
Reed Canarygrass	Switchgrass
	Wildflower Mixes

When "Cool Season" species are mixed with "Warm Season" species, in areas of 1 acre or more, the mixture shall be seeded during the "Warm Season". In areas of less than 1 acre, the mixture of "Cool Season" and "Warm Season" species may be seeded during the "Warm or Cool Seasons".

SODDING PERIODS

March 1 to April 15
and
September 1 to November 15

GENERAL: The entire disturbed area, excepting the paved or surfaced areas, steep rocky slopes and areas of undisturbed native sod or other desirable vegetation shall be fertilized (limed when required), seeded, and mulched. Soil preparation shall conform to the Standard Specifications except as noted below.

All borrow areas shown on the plans are to be fertilized, seeded, and mulched. However, operation in borrow areas where crops are growing may be omitted when requested by the owner.

It shall not be required to till the area to bare ground prior to permanent seeding. If temporary cover has provided stable slopes with no erosion, seed the permanent grasses into the existing cover. If there has been erosion that requires repair prior to seeding, then it may be necessary to regrade the area, resulting in bare ground.

FERTILIZER: A ratio and application rate that equals or exceeds the required minimum rate per acre of N, P_2O_5, K_2O listed in Summary of Seeding Quantities will be acceptable.

MULCHING: Mulch shall be spread uniformly over all disturbed areas and punched in the soil, unless otherwise noted on the plans. The rate of application per acre, thickness in place, for the mulching material is as follows:

1¾ - 2¼ Tons per Acre = 1½" loose depth spread uniformly over acre.

Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.

Other vegetative mulches (Acceptable only with the Engineer's concurrence).

The above rate is a guide. It will be at the discretion of the Engineer to determine what rate is sufficient for adequate protection of newly seeded areas.

The amount of mulch in the bid quantities is estimated. The total mulch required shall be determined in the field. The bid item for mulching shall be paid for by one of the following ways: A) Plan quantity as shown on Summary of Quantities, Seeding Sheet or Water Pollution Control Sheet, B) Slope measurement as measured in field, or C) Drill measurement less 5% as measured at the time of seeding.

NATIVE WILDFLOWER MIX

PLS RATE	NAME	QTY (lb)
0.1	Black Eyed Susan	
1.8	Illinois Bundleflower	
0.15	Maximillian Sunflower	
0.4	Purple Prairie Clover	
2.9	Showy Partridge Pea	
0.1	Upright Prairie Coneflower	
0.3	Butterfly Milkweed	
0.1	Stiff Goldenrod	
0.05	Pinnate Prairie Coneflower	
0.1	Lance-leaf Coreopsis	
0.05	New England Aster	
0.2	Pale Purple Coneflower	
0.05	Plains Coreopsis	
0.05	Hoary Verbena	
0.3	Roundhead Lespedeza	
0.4	Thickspike Gayfeather	
0.05	Wild Bergamot	
0.2	Smooth Oxeye	
0.05	Lemon Mint	
7.35	Total (lb)	

NATIVE WILDFLOWER MIX 2

PLS RATE	NAME	QTY (lb)
0.1	Black Eyed Susan	
1.8	Illinois Bundleflower	
0.15	Maximillian Sunflower	
0.4	Purple Prairie Clover	
2.9	Showy Partridge Pea	
0.1	Upright Prairie Coneflower	
0.3	Butterfly Milkweed	
0.4	Dotted Blazing Star	
0.4	Annual Gaillardia	
0.05	Stiff Goldenrod	
0.05	New England Aster	
0.3	Missouri Evening Primrose	
0.05	Plains Coreopsis	
0.15	White Prairie Clover	
0.3	Roundhead Lespedeza	
0.05	Lemon Mint	
0.15	Pitcher Sage	
7.65	Total (lb)	

NATIVE WILDFLOWER MIX 3

PLS RATE	NAME	QTY (lb)
0.15	Black Eyed Susan	
1.9	Illinois Bundleflower	
0.15	Maximillian Sunflower	
0.05	Western Yarrow	
0.5	Black Sampson Echinacea	
0.05	Upright Prairie Coneflower	
0.3	Butterfly Milkweed	
0.4	Dotted Blazing Star	
0.75	Annual Gaillardia	
0.05	Stiff Goldenrod	
0.05	New England Aster	
0.4	Pitcher Sage	
0.01	Plains Coreopsis	
0.15	White Prairie Clover	
0.2	Purple Prairie Clover	
0.4	Leadplant	
0.02	White Heath Aster	
1	Blue Wild Indigo	
0.05	Lemon Mint	
6.58	Total (lb)	

NATIVE WILDFLOWER MIX 4

PLS RATE	NAME	QTY (lb)
1.9	Illinois Bundleflower	
0.4	Maximillian Sunflower	
0.05	Western Yarrow	
1	Black Sampson Echinacea	
0.1	Upright Prairie Coneflower	
0.1	Scarlet Globemallow	
0.4	Dotted Blazing Star	
1.1	Annual Gaillardia (Firewheel)	
0.1	Hoary Vervain	
0.3	White Prairie Clover	
0.4	Purple Prairie Clover	
0.4	Perennial Gaillardia (Blanket Flower)	
0.02	White Heath Aster	
0.05	Lemon Mint	
6.32	Total (lb)	

Package and deliver the wildflower seed separately from the grass seed mix. Package and deliver the Tall Drop Seed separately from the grass seed and the wildflower mix. Place the grass seed (except Tall Drop Seed) in the large seed box and drill (cover) seed ⅛"-¼". Place the wildflower seed in a separate seed box and drill (cover) seed 1/16" maximum. Place the Tall Drop Seed in a separate (third) seed box and place the seed (using the seed drill) on the soil surface. OPTION: Broadcast Tall Drop Seed on the soil surface.

SUMMARY OF SEEDING QUANTITIES

P.L.S. RATE/ACRE				ACRES				BID ITEM	QUANTITY	UNIT
SHLDR	OTHER			SHLDR	OTHER					
	100				0.20			Fertilizer (15-30-15)		
	180				0.20			Seed (Turf Type Tall Fescue Seed Blend)		
								Seeding	Lump Sum	L.S.
								Mulching (Permanent) (Set Price)		ac

SHLDR = Shoulder Turf Mix: Includes a 30 foot wide strip along the stabilized shoulder on each side of each traveled way, plus all median areas less than 60 feet wide.

OTHER = All other turf areas except Shoulder, Guardrail, and Native areas usually include the Native Wildflower Mix.

GRDRL = Guardrail Mix: This mix does not need mowing. Plant it below the guardrail between the edge of pavement and the edge of the 10:1 slope, and on steep slopes or non-mowable slopes as shown on the drawings.

NATIVE = The Native Mix is a 40% wildflower, 60% native grass mix. See this sheet for the locations of this high wildflower content seed mix.

NOTE: Projects of less than 1 acre shall be bid as "Seeding" by the lump sum. All disturbed areas shall be seeded, fertilized and mulched at the listed rate per acre. The acres are estimated.

NO.	DATE	REVISIONS	BY	APP'D
4	10/13/08	Revised Standard	MRM	SHS
3	12/14/05	Revised Standard	MRM	SHS
2	6/16/05	Revised Standard	SHS	SHS
1	3/01/04	Revised Standard	FCM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

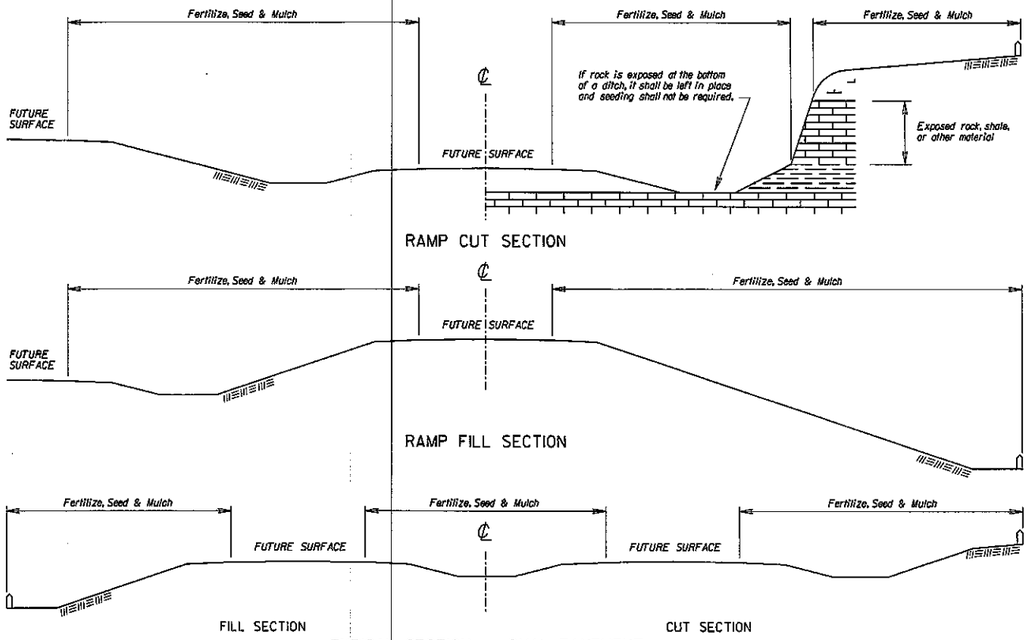
PERMANENT SEEDING
SUMMARY OF SEEDING QUANTITIES

LA850

F.H.W.A. APPROVAL	1/04/2006	APP'D	Scott H. Shields
DESIGNED MRM	DETAILED MRM	QUANTITIES	TRACED
DESIGN CK. SHS	DETAIL CK. SHS	QUAN.CK.	TRACE CK.

Std. Base Files: la850.dgn
Plotted By: dluedke
Plot Location: $UNIT$
File: GV/659/MicroV/Highways/Client Standards/la850.dgn
Plot Date: 10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	14	33



TYPICAL SECTION -- DUAL PAVEMENT

GENERAL: The entire disturbed area, excepting the paved or surfaced areas, steep rocky slopes and areas of undisturbed native sod or other desirable vegetation shall be fertilized (limed when required), seeded, and mulched. Soil preparation shall conform to the Standard Specifications.

Temporary seeding shall be done during any time of the year that the soil can be cultivated. After the temporary seeding has been completed on the entire project, a permanent seeding shall be done by another project during the normal seeding season.

MULCHING: Mulch shall be spread uniformly over all disturbed areas and punched in the soil, unless otherwise noted on the plans. The rate of application per acre, thickness in place, for the mulching materials is as follows:

1¾ - 2¼ Tons per Acre = 1½" loose depth spread uniformly over acre.

Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.

Other vegetative mulches acceptable only with the Engineer's concurrance.

The above rate is a guide. It will be at the discretion of the Engineer to determine what rate is sufficient for adequate protection of newly seeded areas.

The amount of mulch and mulch tacking slurry in the bid quantities is estimated. The total mulch and mulch tacking slurry required shall be determined in the field. The bid item for mulching and mulch tacking slurry shall be paid for by one of the following ways: A) Plan quantity as shown on Summary of Quantities, Seeding Sheet or Water Pollution Control Sheet, B) Slope measurement as measured in field, or C) Drill measurement less 5% as measured at the time of seeding.

SUMMARY OF SEEDING QUANTITIES: PROJECTS OF 1 ACRE OR MORE

P.L.S. RATE/ ACRE		ACRES		BID ITEM	QUANTITY	UNIT
CLT	Slope	CLT	Slope			
				Temporary Fertilizer (* - ** - ***)		Lb
				Temporary Seed (Canada Wildrye Grass Seed)		Lb
				Temporary Seed (Sterile Wheatgrass)		Lb
				Soil Erosion Mix		Lb
				Agricultural Limestone		Tons
				Temporary Berm		LF
				Temporary Ditch Check (Rock) (Set Price)		Cu Yd
				Temporary Ditch Check		LF
				Temporary Inlet Sediment Barrier		Each
				Curb Inlet Protection		LF
				Temporary Sediment Basin		Cu Yd
				Temporary Slope Barrier (Set Price)		LF
				Temporary Slope Drain		LF
				Temporary Stream Crossing		Each
				Sediment Removal (Set Price)		Cu Yd
				Mobilization, Emergency Erosion Control (Set Price)		Each
				Erosion Control (Class X, Type Y)		Sq Yd
				Mulch Tacking Slurry		Ac
				Mulching (Temporary)		Ac

SOIL EROSION MIX

PLS RATE	NAME	QTY (lb)
	Total (lb)	

FERTILIZER: A ratio and application rate that equals or exceeds the required minimum rate per acre of N, P_2O_5, K_2O listed in Summary of Quantities will be acceptable.

- * - N = Nitrogen Rate of Application
- ** - P_2O_5 = Phosphorous Rate of Application
- *** - K_2O = Potassium Rate of Application

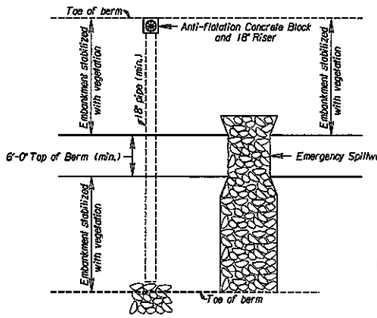
The Contractor will be required to finish areas of excavation, borrow and embankment in accordance with the specifications. Areas that require installation or construction of temporary water pollution control items will be finished in reasonable close conformity to the alignment, grade and cross section shown on the plans or as established by the Engineer.

NOTE: Projects of less than 1 acre shall be bid as "Seeding" by the lump sum. All disturbed areas shall be seeded, fertilized and mulched at the listed rate per acre. The acres are estimated.

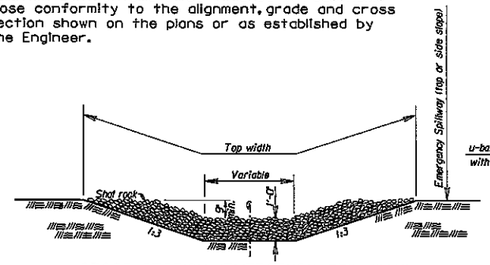
CLT = Construction Limit Tract. This area does not include the channel/slope area and shall be seeded in accordance with the chart to the right.

Channel = This is the area that shall be seeded prior to placement of the Erosion Control. Broadcasting seed is preferred, however, drilling is acceptable

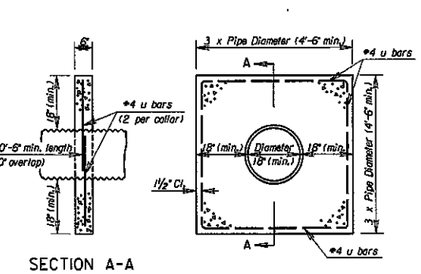
Slope = This is the area that shall be seeded prior to placement of the Erosion Control. Broadcasting seed is preferred, however, drilling is acceptable.



SEDIMENT STORAGE BASIN (PLAN)



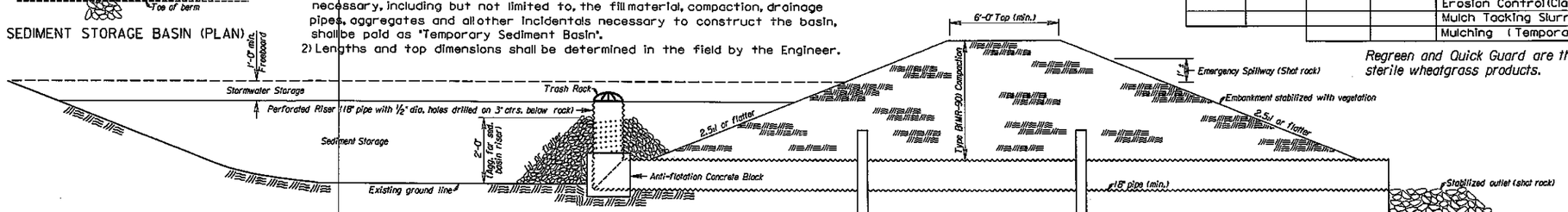
CROSS SECTION (EMERGENCY SPILLWAY)



SECTION A-A

CONCRETE ANTI-SEEP COLLAR

NOTES:
1) Temporary Sediment Basins shall be constructed at locations as directed by the Engineer or as approved in the SWPPP Schedule. All work and materials necessary, including but not limited to, the fill material, compaction, drainage pipes, aggregates and all other incidentals necessary to construct the basin, shall be paid as "Temporary Sediment Basin".
2) Lengths and top dimensions shall be determined in the field by the Engineer.



SEDIMENT STORAGE BASIN (ELEVATION)

SUMMARY OF SEEDING QUANTITIES: PROJECTS LESS THAN 1 ACRE

P.L.S. RATE/ ACRE		ACRES		BID ITEM	QUANTITY	UNIT
CLT	Slope	CLT	Slope			
				Temporary Fertilizer (* - ** - ***)		Lb
				Temporary Seed (Canada Wildrye Grass Seed)		Lb
				Temporary Seed (Sterile Wheatgrass)		Lb
				Soil Erosion Mix		Lb
				Agricultural Limestone		Tons
				Temporary Seeding	Lump Sum	LS
				Temporary Ditch Check (Rock) (Set Price)	1	Cu Yd
				Temporary Ditch Check		LF
				Temporary Slope Barrier (Set Price)	1	LF
				Sediment Removal (Set Price)	1	Cu Yd
				Mobilization, Emergency Erosion Control (Set Price)	1	Each
				Erosion Control (Class X, Type Y)		Sq Yd
				Mulch Tacking Slurry		Ac
				Mulching (Temporary)		Ac

Regreen and Quick Guard are the approved sterile wheatgrass products.

NO.	DATE	REVISIONS	BY	APP'D
3	3/01/10	Revised Standard	MRM	SHS
2	7/08/09	Revised Standard	MRM	SHS
1	5/29/07	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

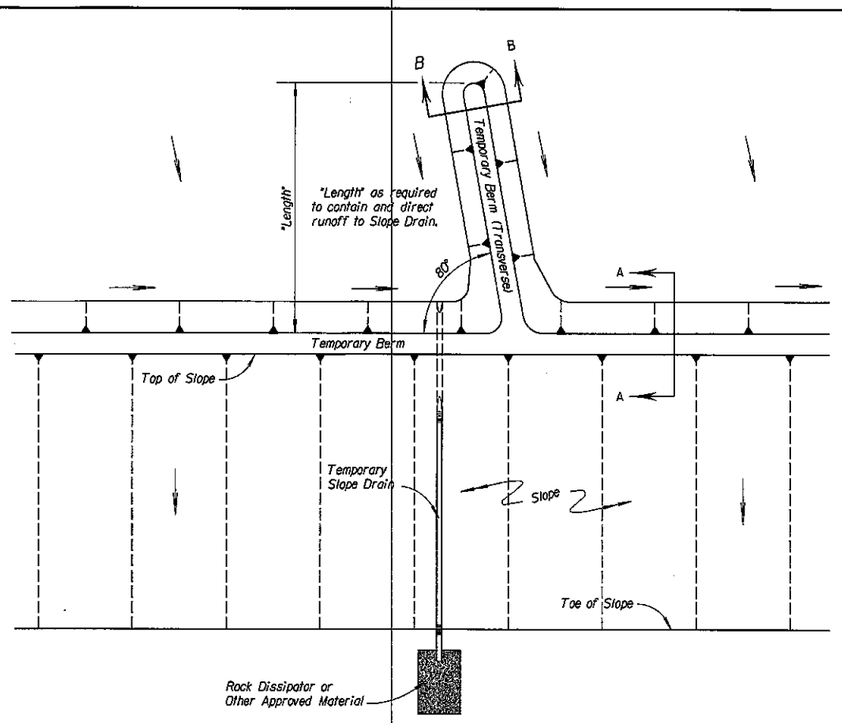
LA852A

F.H.W.A. APPROVAL	11/02/2009	APP'D	Scott H. Shields
DESIGNED BB	DETAILED BB	QUANTITIES	TRACED BB
DESIGN CK. SHS	DETAIL CK. SHS	QUAN.CK.	TRACE CK. SHS

Std. Base File: la852a.dgn
Plotted By: cluedke | Plot Locations: $UNITS
File: G:\1694\Micro\Highways\Client Standards\la852a.dgn
Plot Date: 3/26/2010

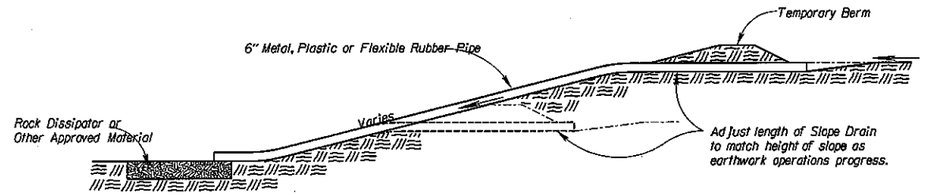
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	15	33

NOTES:
1) Temporary Slope Drain and Temporary Berm may be used on either project foreslopes or project backslopes.
2) Discharge of Slope Drains shall be into stabilized ditch or area, or into Sediment Basin.
3) Pipe shall be secured in place as approved by Engineer.



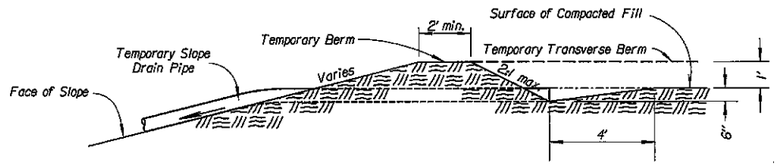
TYPICAL PLAN VIEW OF TEMPORARY BERM AND TEMPORARY SLOPE DRAIN

NO SCALE



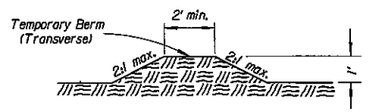
TYPICAL PROFILE OF TEMPORARY SLOPE DRAIN

NO SCALE



SECTION A-A

NO SCALE

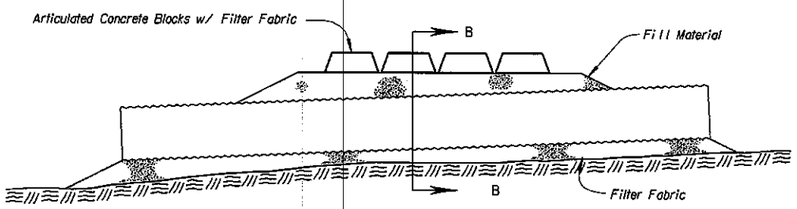


SECTION B-B

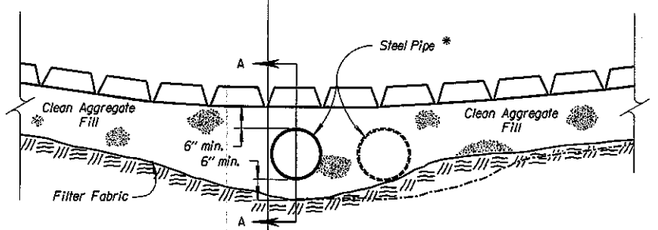
NO SCALE

TYPICAL PROFILE OF TEMPORARY BERM

NO SCALE



SECTION A-A

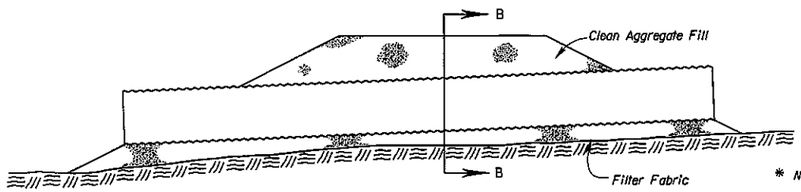


SECTION B-B

* NOTE:
Quantity, length and diameter of steel pipe to be determined by design flow calculations.

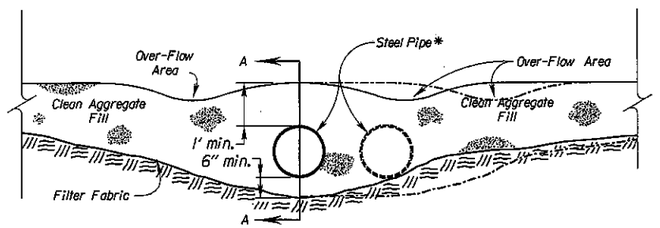
TEMPORARY STREAM CROSSING (ARTICULATED CONCRETE BLOCKS)

NO SCALE



SECTION A-A

* NOTE:
Quantity, length and diameter of steel pipe to be determined by design flow calculations.



SECTION B-B

TEMPORARY STREAM CROSSING (AGGREGATE)

NO SCALE

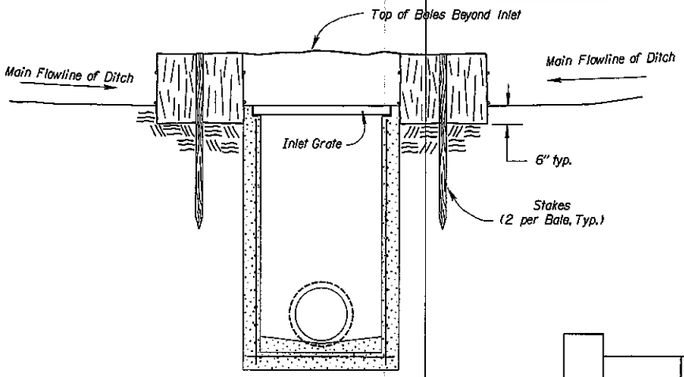
NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1	5/10/99	Revised Standard	WCL	RDR

KANSAS DEPARTMENT OF TRANSPORTATION
TEMPORARY EROSION AND POLLUTION CONTROL
TEMPORARY SLOPE DRAIN
TEMPORARY STREAM CROSSING (AGGREGATE)
TEMP. STREAM CROSS. (ARTC. CONC. BLOCKS)
LA852B

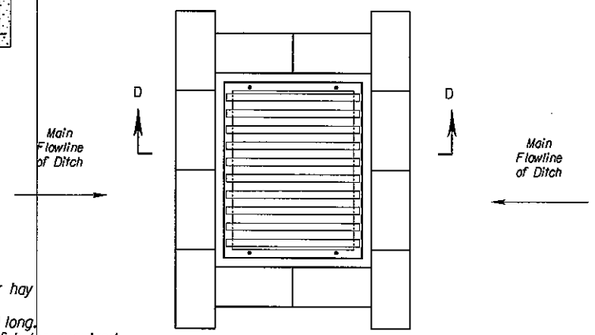
F.H.W.A. APPROVAL		5/20/99	APP'D	Richard D. Ross	
DESIGNED	WCL	DETAILED	WCL	QUANTITIES	TRACED WCL
DESIGN CK.	RDR	DETAIL CK.	RDR	QUAN.CK.	TRACE CK. RDR

Std. Base File:
Plotted By: cluedke
Plot Location: $UNIT$
File: G:\1694\Micro\Highways\Client Standards\la852b.dgn
Plot Date: 10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	16	33



SECTION D - D

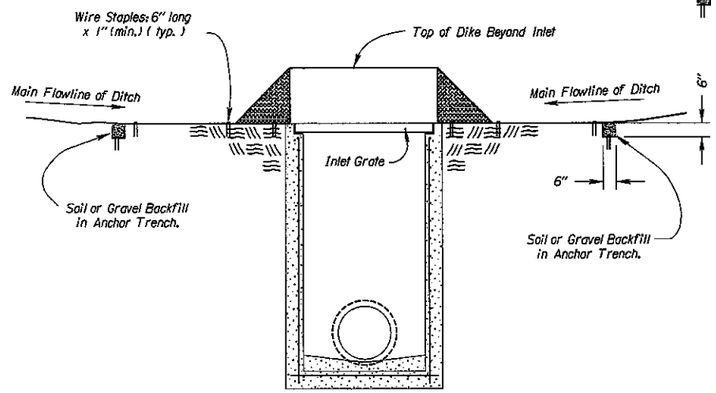


PLAN

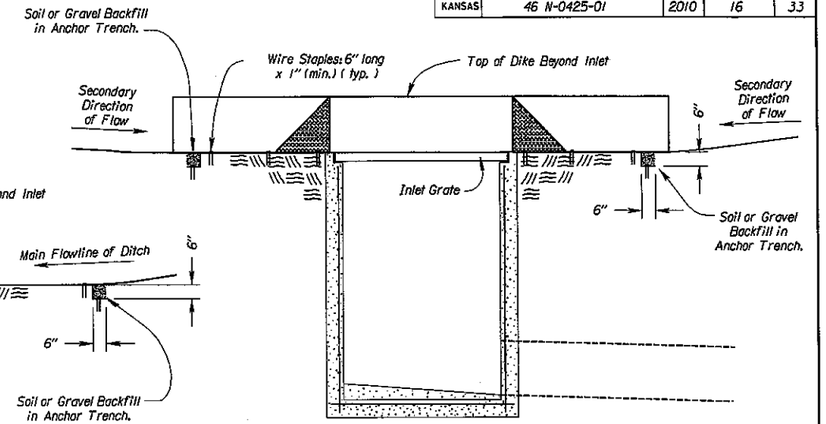
NOTES:

The use of Straw or Hay Bales, Silt Fence or Triangular Silt Dike for Temporary Inlet Sediment Barrier is at the option of the Contractor.

Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.



SECTION A - A



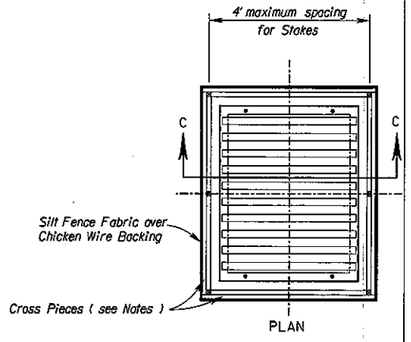
SECTION B - B

STRAW OR HAY BALES:

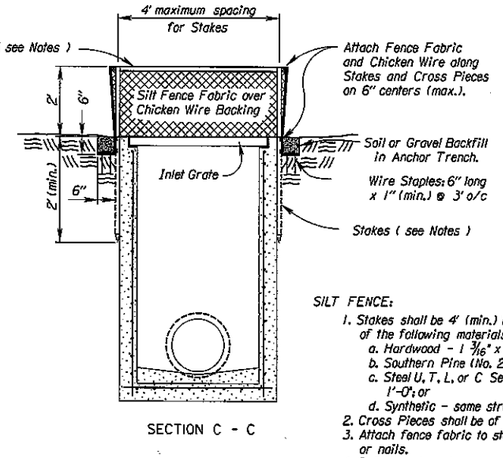
1. *Place bales tightly together, with loose straw or hay wedged between bales to close off openings.*
2. *Wood stakes shall be 2" x 2" (nom.) x 4' (min.) long.*
3. *Refer to plans sheets to estimate the number of bales required.*
4. *Use only twine to bind bales. The use of wire binding is prohibited because it does not readily biodegrade.*

TEMPORARY INLET SEDIMENT BARRIER
(STRAW OR HAY BALE METHOD)

NO SCALE



PLAN



SECTION C - C

SILT FENCE:

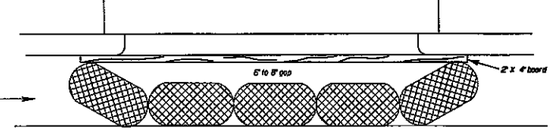
1. *Stakes shall be 4' (min.) long and of one of the following materials:*
   a. *Hardwood - 1 3/16" x 1 3/16";*
   b. *Southern Pine (No. 2) - 2 5/8" x 2 5/8";*
   c. *Steel U, T, L, or C Section - 1.33 lbs. per 1'-0"; or*
   d. *Synthetic - same strength as wood stakes.*
2. *Cross Pieces shall be of same material as Stakes.*
3. *Attach fence fabric to stakes with staples, wire or nails.*
4. *Refer to plan sheets to estimate the length of Silt Fence required.*

TEMPORARY INLET SEDIMENT BARRIER
(SILT FENCE METHOD)

NO SCALE

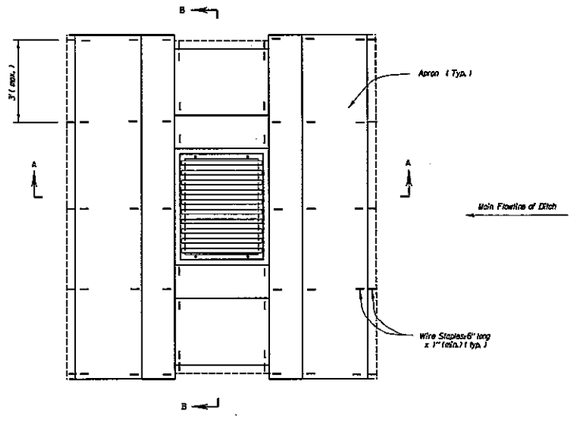
Bags = synthetic net (3mm mesh) or burlap bags (24" long by 12" wide)

Rock = approximately 1" to 2" diameter



CURB INLET PROTECTION

1. *Place gravel bags such that no gaps are evident.*
2. *Height of bags must not be above top of curb.*
3. *Alternative products may be used other than gravel bags such as the "Gutter Buddy". Products must be approved by the Engineer.*



PLAN

TEMPORARY INLET SEDIMENT BARRIER
(TRIANGULAR SILT DIKE METHOD)

NO SCALE

NO.	DATE	REVISIONS	BY	APP'D
3	8/01/08	Revised Standard	MRM	SHS
2	9/10/07	Revised Standard	MRM	SHS
1	6/16/05	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

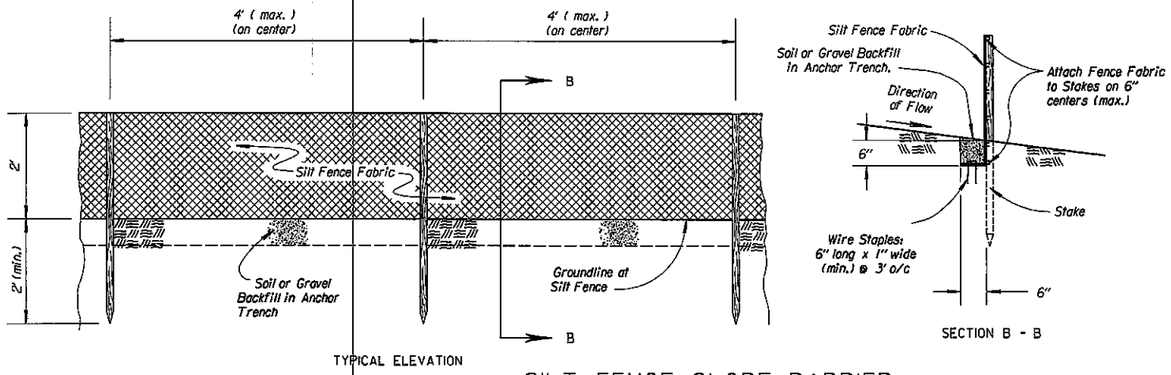
TEMPORARY EROSION AND POLLUTION CONTROL

TEMP. INLET SEDIMENT BARRIER (SILT FENCE)
TEMP. INLET SEDIMENT BARRIER (BALES)
TEMP. INLET SEDIMENT BARRIER (T.S.D.)
LA852C

F.H.W.A. APPROVAL	9/23/2008	APP'D	Scott H. Shields
DESIGNED	MRM	DETAILED	MRM	QUANTITIES	TRACED
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	TRACE CK.

Std. Base Files:	la852c.dgn
Plotted By:	dluedke
Plot Location:	$UNIT$
File:	GV1694\MicroVisions\Client Standards\la852c.dgn
Plot Date:	10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	17	33

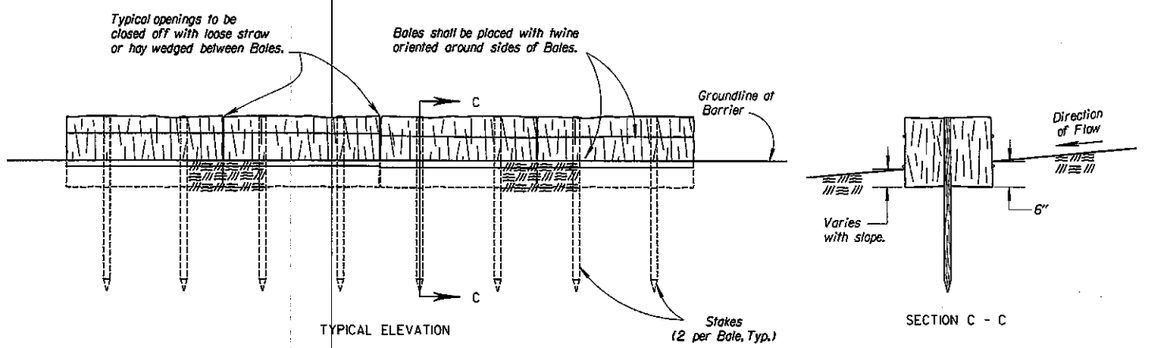


TYPICAL ELEVATION

SECTION B - B

SILT FENCE SLOPE BARRIER

NO SCALE

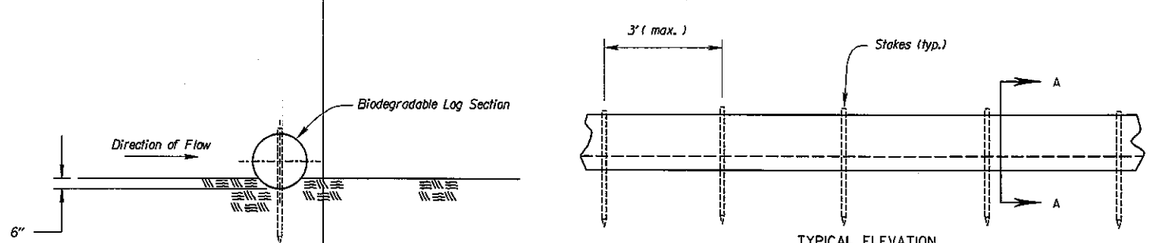


TYPICAL ELEVATION

SECTION C - C

STRAW BALE SLOPE BARRIER

NO SCALE



SECTION A - A

TYPICAL ELEVATION

BIODEGRADABLE LOG SLOPE BARRIER

NO SCALE

INSTALLATION NOTES

STRAW OR HAY BALES:

1. Place bales tightly together, with loose straw or hay wedged between bales to close off openings.
2. Wood stakes shall be 2" x 2" (nom.) x 4' (min.) long.
3. Refer to plans sheets to estimate the length of bales required.
4. Use only twine to bind bales. The use of wire binding is prohibited because it does not readily biodegrade.

SILT FENCE:

1. Stakes shall be 4' (min.) long and of one of the following materials:
   a. Hardwood - 1 3/16" x 1 3/16";
   b. Southern Pine (No. 2) - 2 5/8" x 2 5/8";
   c. Steel U, T, L, or C Section - 1.33 lbs. per 1'-0"; or
   d. Synthetic - same strength as wood stakes.
2. Attach fence fabric to stakes with staples, wire or nails.
3. Refer to plan sheets to estimate the length of Silt Fence required.

BIODEGRADABLE LOG BARRIERS

1. Place biodegradable logs tightly together.
2. Wood stakes shall be 2" x 2" (nom.) x 4' (min.) long.
3. Wire staples shall be 6" long x 1" wide (min.) and placed on 3' (max.) centers.
4. Refer to plan sheets to estimate length of biodegradable log barriers required.

Biodegradable Logs, Straw Wattles & Sediment Logs

		PRODUCT		
		9" Sediment Log & 9" Straw Wattle (ft)	12" Sediment Log & 12" Straw Wattle (ft)	20" Sediment Log & 20" Straw Wattle (ft)
Slope Gradient	≤4H:1V	40	60	80
	3H:1V	30	45	60
	2H:1V	20	30	40
	1H:1V	10	15	20

9" and 12" material should only be used in areas which have been seeded and mulched. 20" material should be used in all other areas. Deviations should be approved by the Field Engineer.

GENERAL NOTES

1) The use of Straw or Hay Bales, Silt Fence or Biodegradable Logs is at the option of the Contractor.
2) The slope barriers shall be placed along contour lines, with a short section turned upgrade at each end of the barrier. The maximum length of the slope barrier shall not exceed 250 feet, and the barrier ends need to be staggered.
3) At culverts, the Straw or Hay Bales or Silt Fence shall be placed over the culvert, not through the streambed flowline.
4) Barriers damaged by Contractor's negligence, including improper maintenance or lack of maintenance, shall be repaired immediately by Contractor at no additional cost to KDOT.
5) Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.

NO.	DATE	REVISIONS	BY	APP'D
3	8/26/09	Revised Standard	MRM	SHS
2	5/11/09	Revised Standard	MRM	SHS
1	9/10/07	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

STRAW OR HAY BALE SLOPE BARRIERS

SILT FENCE SLOPE BARRIERS

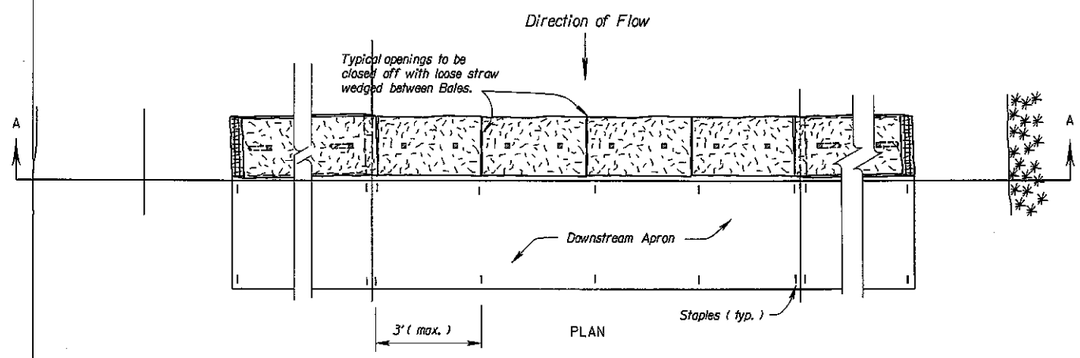
BIODEGRADABLE LOG SLOPE BARRIERS

LA852D

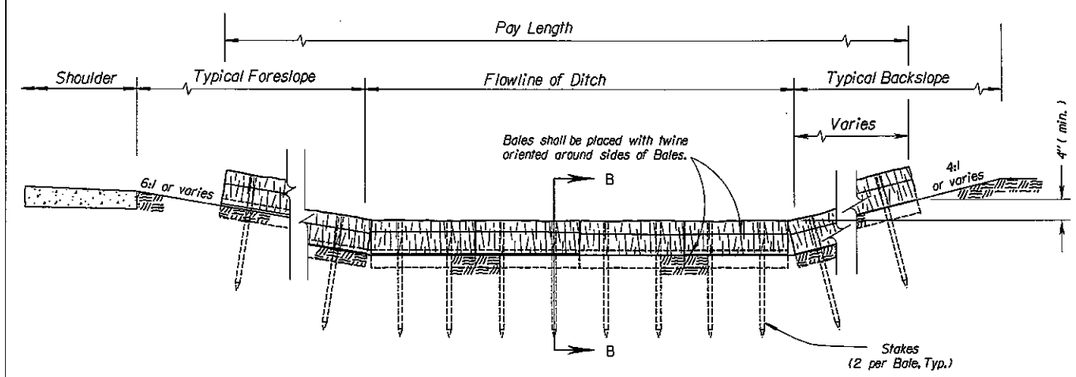
F.H.W.A. APPROVAL		11/02/2009	APP'D	Scott H. Shields	
DESIGNED	MRM	DETAILED	MRM	QUANTITIES	TRACED MRM
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	TRACE CK. SHS

Std. Base File: la852d.dgn
Plotted By: cluedke
Plot Location: $UNIT$
File: G:\1694\Micro\Highways\Client Standards\la852d.dgn
Plot Date: 3/26/2010

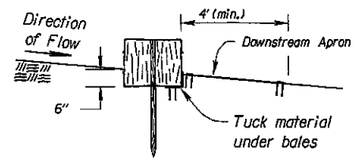
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	18	33



PLAN



SECTION A-A



SECTION B-B

STRAW OR HAY BALE DITCH CHECK

NO SCALE

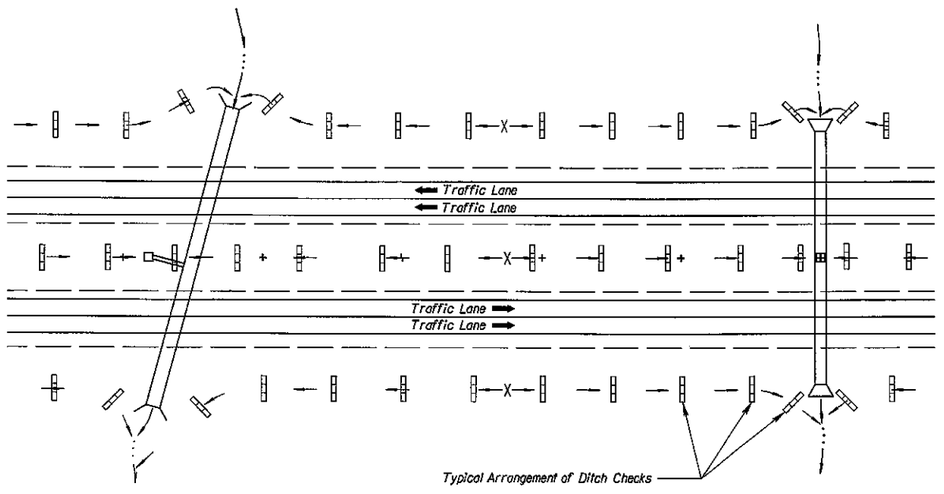
STRAW OR HAY BALE NOTES:

1. Place bales tightly together, with loose straw or hay wedged between bales to close off openings.
2. Wood stakes shall be 2" x 2" (nom.) x 4' (min.) long.
3. Use as many bales as necessary to completely block the ditch and to prevent water from flowing around the ends of the ditch check.
4. Use only twine to bind bales. The use of wire binding is prohibited because it does not readily biodegrade.
5. Use silt fence material as the downstream apron to prevent scour below the ditch check.
6. Wire staples shall be 6" long by 1" wide, minimum.

GENERAL NOTES:

1) The choice of ditch check methods is at the option of the Contractor.
2) Use only rock checks in situations where the ditch slope exceeds 6 percent.
3) Ditch checks damaged by Contractor's negligence, including improper maintenance or lack of maintenance, shall be repaired by Contractor at no extra cost to KDOT.
4) Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.

TEMPORARY DITCH CHECK SPACING	
DITCH ℄ SLOPE (%)	SPACING INTERVAL (FEET)
1.0	200
2.0	100
3.0	65
4.0	50
5.0	40
6.0	33
NOTE: Use this spacing for all except Rock Ditch Checks.	



TYPICAL DITCH CHECK LAYOUT PLAN

NO SCALE

Std. Base File: la852e.dgn
Plotted By: cluedke
Plot Location: $UNIT$
File: G:\1694\Micro\Highways\Client Standards\la852e.dgn
Plot Date: 10/28/2009

NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3	9/10/07	Revised Standard	MRM	SHS
2	6/16/05	Revised Standard	MRM	SHS
1	5/10/99	Revised Standard	WCL	RDR

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL
STRAW OR HAY BALE DITCH CHECKS

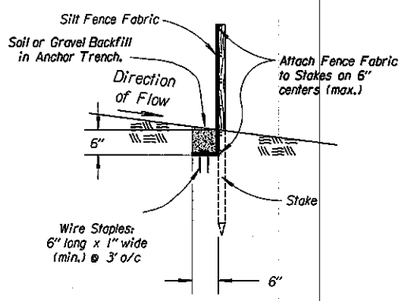
LA852E

F.H.W.A. APPROVAL	6/22/2005	APP'D	Scott H. Shields
DESIGNED MRM	DETAILED MRM	QUANTITIES	TRACED MRM
DESIGN CK. SHS	DETAIL CK. SHS	QUAN.CK.	TRACE CK. SHS

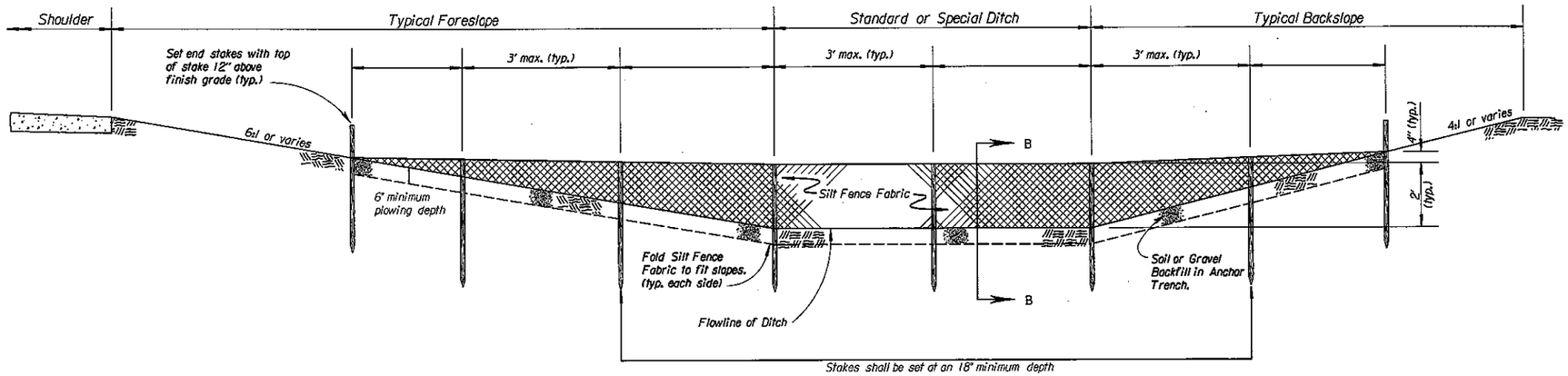
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	19	33

SILT FENCE:

1. Stakes shall be 4' (min.) long and of one of the following materials:
   a. Hardwood - 1 3/16" x 1 3/16";
   b. Southern Pine (No. 2) - 2 5/8" x 2 5/8";
   c. Steel U, T, L, or C Section - 1.33 lbs. per 1'-0"; or
   d. Synthetic - same strength as wood stakes.
2. Attach fence fabric to stakes with staples, wire or nails.
3. Refer to plan sheets to estimate the length of Silt Fence required.
4. Use support fencing when tributary area is greater than 2.4 acres or when ditch gradient is greater than 2 percent.
5. Silt fence plowing is acceptable at a 6" minimum depth.



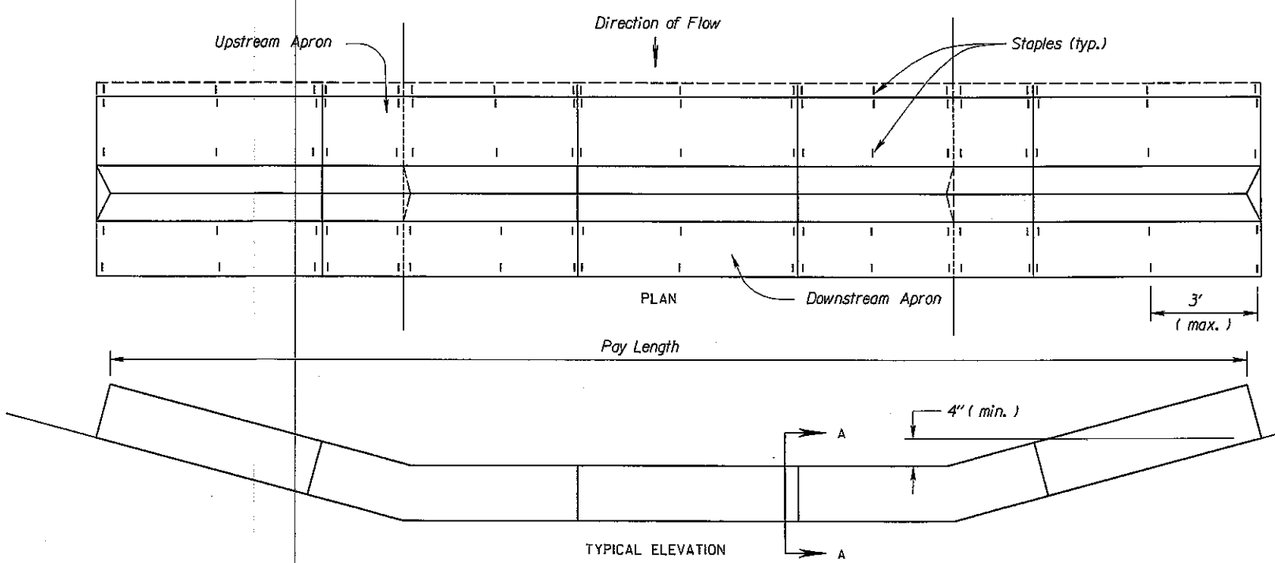
SECTION B - B

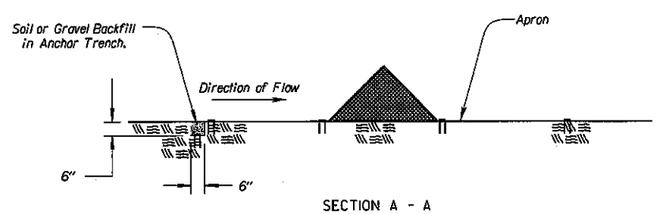


SILT FENCE DITCH CHECK

NO SCALE

TYPICAL ELEVATION





SECTION A - A

TRIANGULAR SILT DIKE NOTES

1. Place Triangular Silt Dike sections tightly together, with apron material overlapping end-to-end by 6".
2. Wire Staples shall be 6" long by 1" wide, minimum.
3. Use as many Triangular Silt Dike sections as necessary to insure water does not flow around end of ditch check.

TRIANGULAR SILT DIKE DITCH CHECK

NO SCALE

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1	07/02/07	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

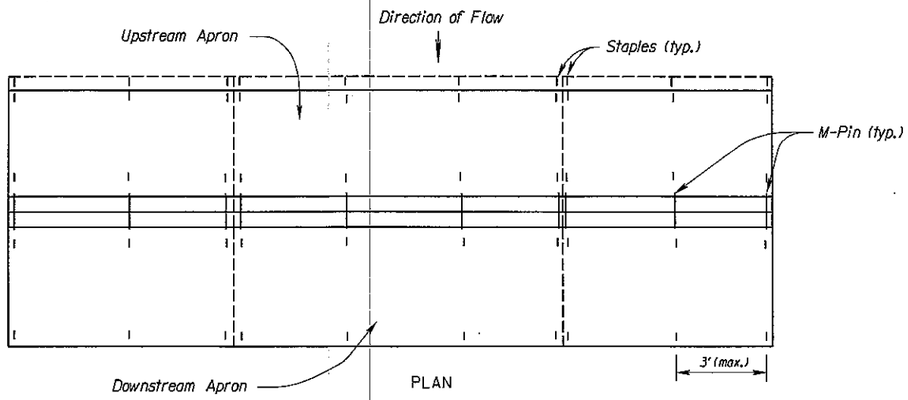
TEMPORARY EROSION AND POLLUTION CONTROL
SILT FENCE DITCH CHECKS
TRIANGULAR SILT DIKE DITCH CHECKS

LA852F

F.H.W.A. APPROVAL		7/23/07	APP'D	Scott H. Shields			
DESIGNED	MRM	DETAILED	MRM	QUANTITIES		TRACED	
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.		TRACE CK.	

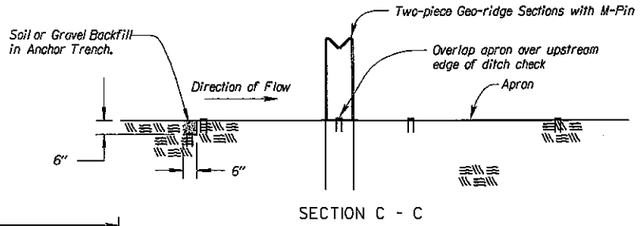
Std. Base Files:
Plotted By: dwedke | Plot Location: $UNIT$
Files: GV1694\MicroHighways\Client Standards\la852f.dgn
Plot Date: 10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	20	33

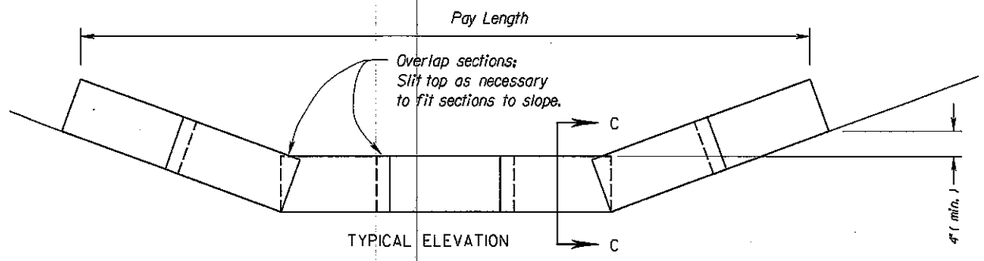


GEO-RIDGE PERMEABLE BERM NOTES

1. Overlap Geo-ridge Berm sections and apron material by 6".
2. Use M-Pins supplied by manufacturer to secure geo-ridge Berm sections.
3. Use as many Geo-ridge Berm sections as necessary to insure water does not flow around end of ditch check.
4. Use silt fence material as the apron to prevent scour above and below the ditch check.
5. Wire Staples shall be 6" long by 1" wide, minimum.



SECTION C - C



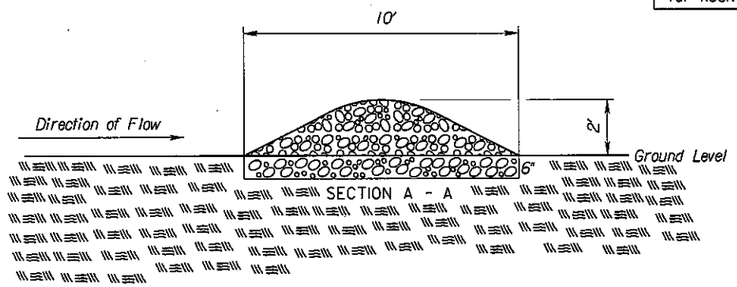
GEO-RIDGE PERMEABLE BERM DITCH CHECK

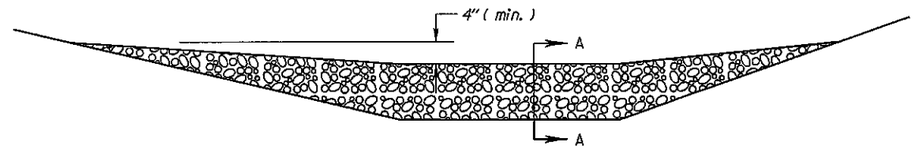
NO SCALE

ROCK DITCH CHECK NOTES

1. Rock shall be clean aggregate, D50 = 6".
2. Place rock in such manner that water will flow over, not around ditch check.
3. Do not use rock ditch checks in clear zone.
4. Excavation: The ditch area shall be reshaped to fill any eroded areas. Prior to placement of the rock, the ditch shall be excavated to the dimensions of the Rock Ditch Check and to a minimum depth of 6" (150mm). After placement of the rock, backfill and compact any over excavated soil to ditch grade. This work shall be subsidiary to the bid item Temporary Ditch Check (Rock) (Set Price).
5. Aggregate excavated on site may be used as an alternate to the 6" rock, if approved by the Engineer.

TEMPORARY ROCK DITCH CHECK SPACING	
DITCH ℄ SLOPE (%)	SPACING INTERVAL (FEET)
5.0	60
6.0	50
7.0	43
8.0	36
9.0	33
10.0	29
NOTE: Use this spacing only for Rock Ditch Checks.	

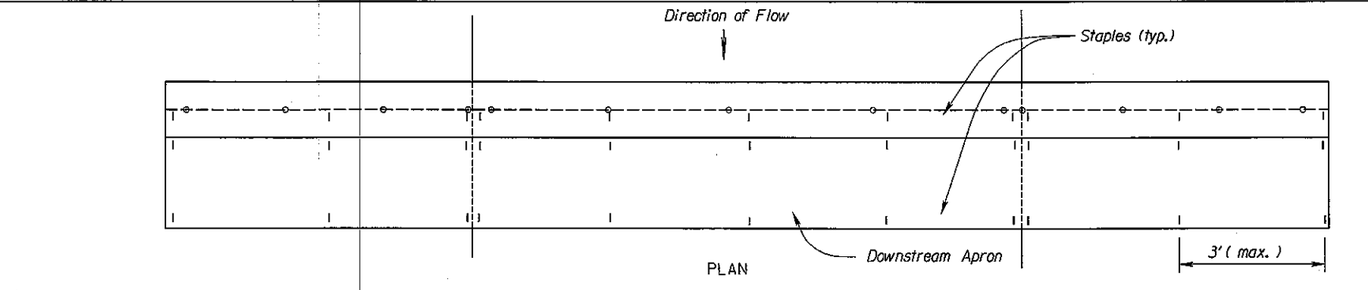


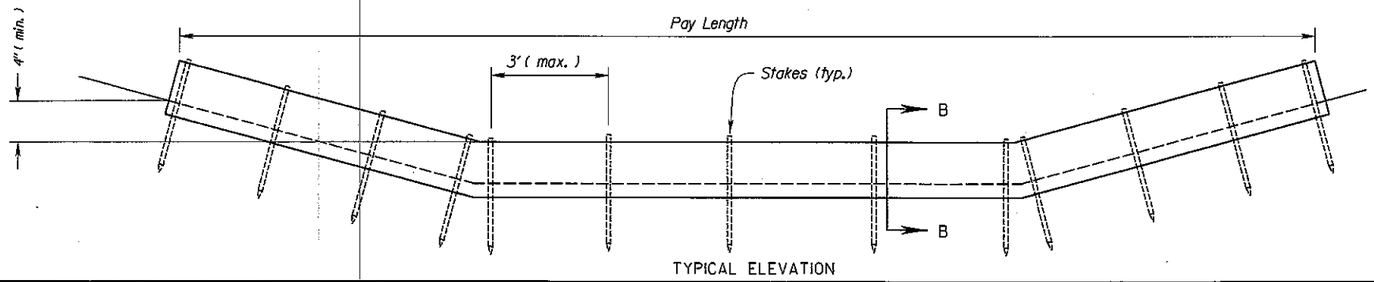


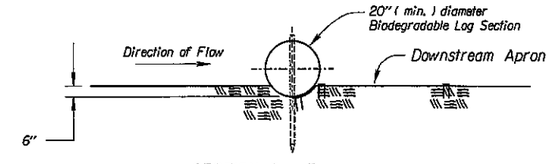
TYPICAL ELEVATION

ROCK DITCH CHECK

NO SCALE







SECTION B - B

BIODEGRADABLE LOG DIKE NOTES

1. Place biodegradable logs tightly together, with apron material overlapping end-to-end by 6".
2. Wire staples shall be 6" long by 1" wide, minimum.
3. Use as many biodegradable log sections as necessary to insure water does not flow around end of ditch check.
4. Wood stakes shall be 2" x 2" (nom.) x 4' (min.) long.
5. Use silt fence material as the downstream apron to prevent scour below the ditch check.

BIODEGRADABLE LOG DITCH CHECK

NO SCALE

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1	5/03/06	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

ROCK DITCH CHECKS

BIODEGRADABLE LOG DITCH CHECKS

GEO-RIDGE PERMEABLE BERM DITCH CHECKS

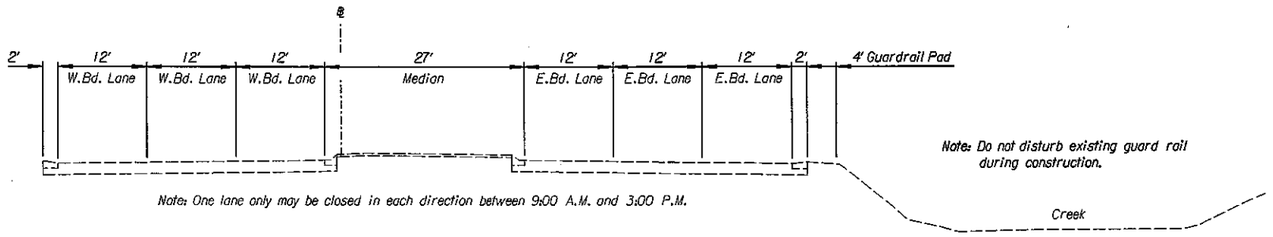
LA852G

F.H.W.A. APPROVAL		5/03/2006	APP'D	Scott M. Shields	
DESIGNED	MRM	DETAILED	MRM	QUANTITIES	TRACED MRM
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	TRACE CK. SHS

Std. Base File: ta852g.dgn
Plotted By: cluedke | Plot Location: $UNIT$
File: GV1694\MicroHighways\Client Standards\ta852g.dgn
Plot Date: 10/28/2009

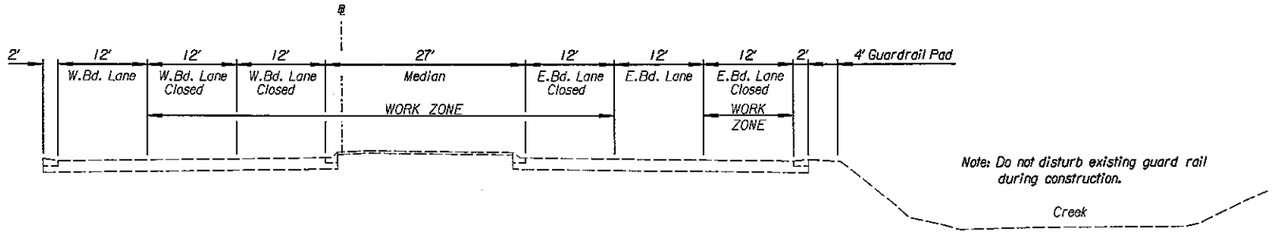
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	21	33

Guide to designer: Use this sheet when subgrading in rock or shale is not required.
See Soils and Geology Reports for additional plan notes.



Note: One lane only may be closed in each direction between 9:00 A.M. and 3:00 P.M.

TYPICAL SECTION
DAYTIME LANE CLOSURE



TYPICAL SECTION
NIGHT TIME LANE CLOSURES

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

CONSTRUCTION SEQUENCING
TYPICAL SECTIONS

FHWA APPROVAL		APP'D.	
DESIGNED	DETAILED	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK.

Drawn By : duedke Plotted : 10/28/2009
File : G:\11694\MicroHighways\116949501.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	22	33

LEGEND:

- Lane Open to Traffic
- Type III Barricade
- Road Closure Using Type III Barricades
- Channelizing Device
- Construction Sign - One Post
- Construction Sign - Two Posts

Traffic Control

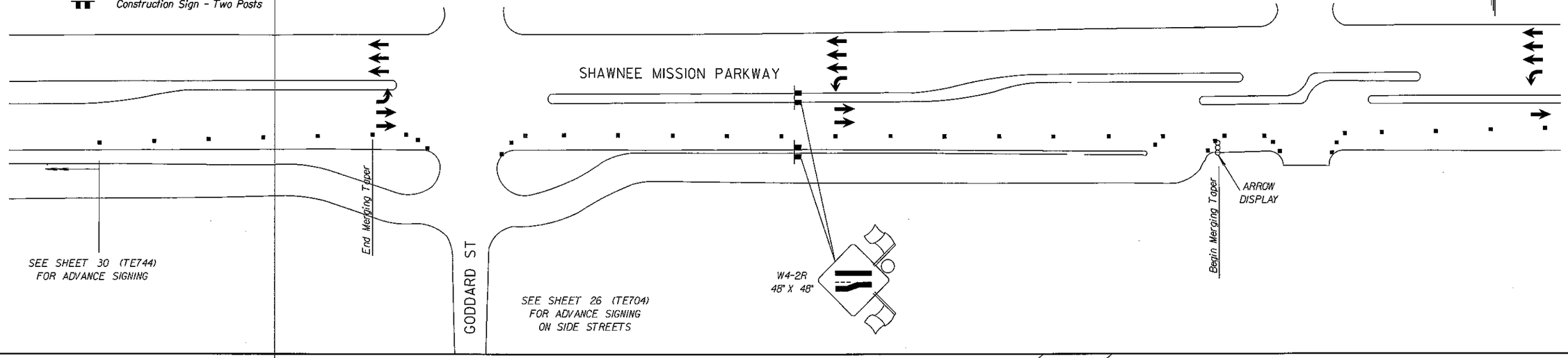
Single Lane Closure

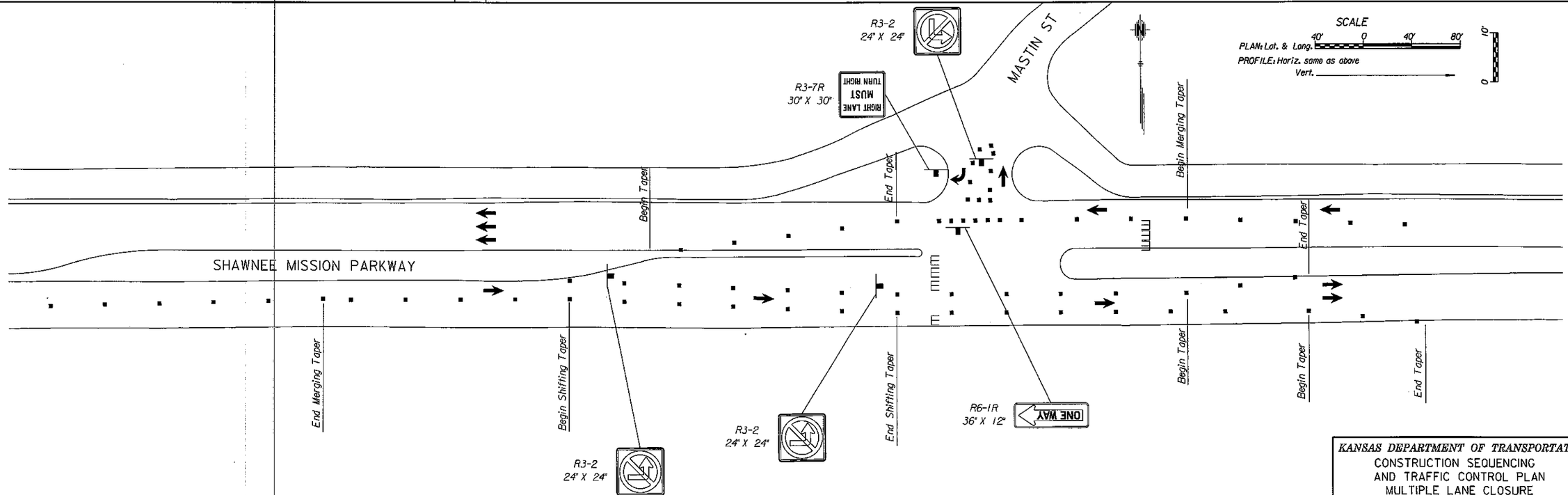
See Sheet 30 for Single Lane Closure details. Single lane may be closed between the hours of 9:00 a.m. to 3:00 p.m.

Multiple Lane Closure

Westbound traffic on Shawnee Mission Parkway will travel using the far right lane while eastbound traffic will be merged to the far left lane, then shifted to the center lane. Left turns not to be allowed on southbound Mastin at Shawnee Mission Parkway or on eastbound Shawnee Mission Parkway at Mastin. Lanes may be closed between the hours of 9:00 p.m. on Saturday night to 9:00 a.m. Sunday Morning. Lane closures must be coordinated with the Merriam Police Department.

SCALE
40' 0 40' 80'
PLAN: Lat. & Long.
PROFILE: Horiz. same as above
Vert.
10' 0





KANSAS DEPARTMENT OF TRANSPORTATION
CONSTRUCTION SEQUENCING
AND TRAFFIC CONTROL PLAN
MULTIPLE LANE CLOSURE
SHEET 1 OF 2

BY	DATE
REFERENCES NOTED	
REFERENCES CHECKED	

Drawn By : cluedke Plotted : 10/28/2009
File : G:\11694\Micro\Traffic\116949601.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	23	33

LEGEND:

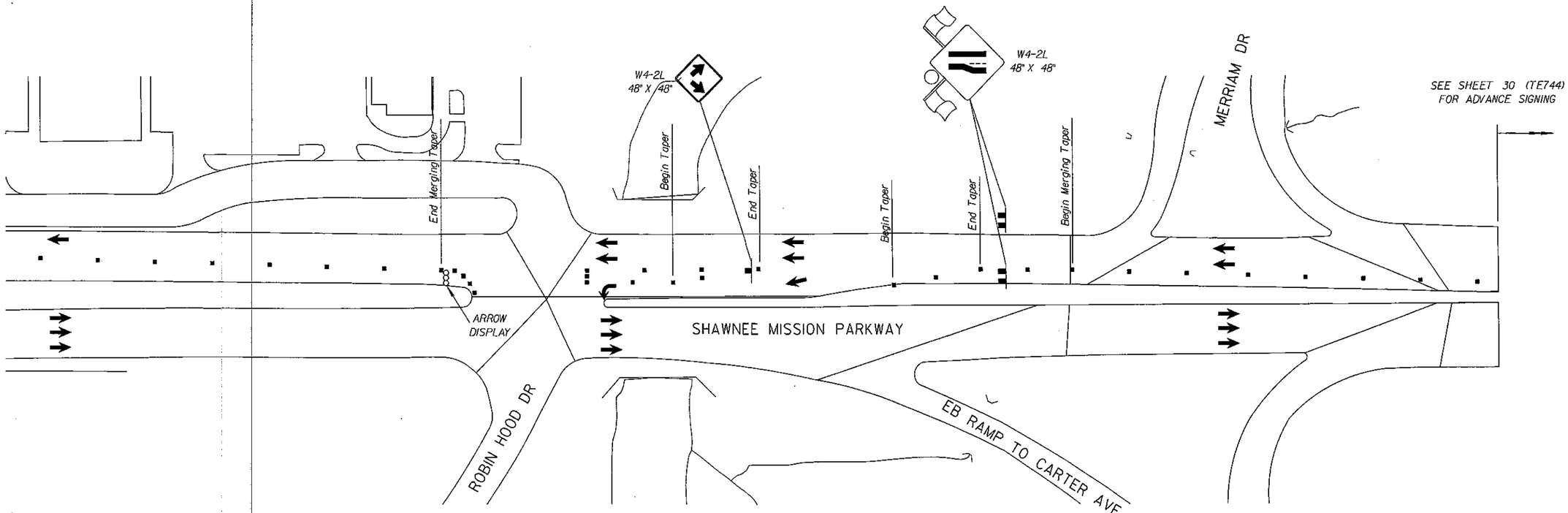
- Lane Open to Traffic
- Type III Barricade
- Road Closure Using Type III Barricades
- Channelizing Device
- Construction Sign - One Post
- Construction Sign - Two Posts

SCALE

PLAN: Lat. & Long. 40' 0 40' 80'

PROFILE: Horiz. same as above

Vert. 10' 0



DATE	BY	
		REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke Plotted : 10/28/2009
File : G:\11694\Micro\Traffic\116949602.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

CONSTRUCTION SEQUENCING
AND TRAFFIC CONTROL PLAN
MULTIPLE LANE CLOSURE
SHEET 2 OF 2

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	24	33

1. MUTCD COMPLIANCE:

ALL TEMPORARY TRAFFIC CONTROL DEVICES AND THEIR INSTALLATION AND MAINTENANCE SHALL COMPLY WITH THE LATEST EDITION OF THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (MUTCD) FOR STREETS AND HIGHWAYS WHICH HAS BEEN ADOPTED BY THE SECRETARY OF TRANSPORTATION. WHENEVER THE TEMPORARY TRAFFIC CONTROL STANDARDS CONFLICT WITH THE MUTCD, THE STANDARDS SHALL GOVERN.

2. DESIGN SPEED:

THOSE ITEMS DELEGATED TO TEMPORARY TRAFFIC CONTROL SHOULD BE DESIGNED AND INSTALLED USING THE POSTED/LEGAL SPEED OF THE ROADWAY PRIOR TO WORK STARTING.

3. CLEAR ZONE:

ALL CONSTRUCTION EQUIPMENT (INCLUDING VEHICLES), MATERIALS, AND DEBRIS SHALL BE STORED OUT OF THE CLEAR ZONE. WHERE THIS CANNOT BE ACHIEVED, THE CONTRACTOR SHALL PLACE APPROPRIATE SIGNS, OBJECT IDENTIFIERS, AND/OR BARRICADES AS DESIGNATED BY THE ENGINEER. TEMPORARY TRAFFIC CONTROL DEVICES NEEDED FOR THIS CONDITION SHALL BE CONSIDERED SUBSIDIARY TO OTHER BID ITEMS.

4. MINIMUM LANE WIDTHS:

LANE WIDTHS SHALL BE A MINIMUM OF 11' (MEASURED BETWEEN CENTERLINES OF PAVEMENT MARKINGS) OR AS SHOWN ON THE PLANS, OR AS DIRECTED BY THE ENGINEER. A LANE WIDTH LESS THAN 11' MAY REQUIRE RESTRICTED ROADWAY WIDTH SIGNING.

5. FLAGGER:

A MINIMUM OF ONE FLAGGER SHALL BE STATIONED WITHIN EACH MULTI-LANE ROADWAY ACTIVITY AREA WHERE WORK IS IN A CLOSED LANE ADJACENT TO TRAFFIC AND NOT SEPARATED BY A CONCRETE SAFETY BARRIER SYSTEM.

6. PAVEMENT MARKING:

WHEN THE WORK WILL OCCUPY A LOCATION MORE THAN THREE DAYS, ALL CONFLICTING PAVEMENT MARKINGS SHALL BE REMOVED OR MASKED AND ALL TRANSITION TAPERS, CROSSOVERS, AND EDGE LINES ALONG CHANNELIZING DEVICES SHALL BE MARKED WITH SOLID 4" WIDE PAVEMENT MARKING.

7. FIRST MODULE OF IBS:

THE FIRST MODULE OF EACH INERTIAL BARRIER SYSTEM (IBS) SHALL HAVE A MINIMUM OF 2 SQ. FT. OF FLUORESCENT ORANGE ASTM TYPE IV SHEETING FACING TRAFFIC. EITHER A VERTICAL RECTANGLE OR DIAMOND SHAPE MAY BE USED.

8. PEDESTRIAN / BICYCLE SAFETY:

WORK ZONE SIGNS SHALL NOT INHIBIT PEDESTRIAN AND BICYCLE TRAFFIC ON SIDEWALKS OR OTHER AREAS DESIGNATED FOR PEDESTRIAN OR BICYCLE USE.

CONSIDERATION SHOULD BE MADE TO SEPARATE PEDESTRIAN AND BICYCLE MOVEMENTS FROM BOTH WORK SITE ACTIVITY AND VEHICULAR TRAFFIC. UNLESS A REASONABLE SAFE ROUTE THAT DOES NOT INVOLVE CROSSING THE ROADWAY CAN BE PROVIDED, PEDESTRIANS AND BICYCLISTS SHOULD BE APPROPRIATELY DIRECTED WITH ADVANCE SIGNING THAT ENCOURAGES THEM TO CROSS TO THE OPPOSITE SIDE OF THE ROADWAY. IN URBAN AND SUBURBAN AREAS WITH HIGH VEHICULAR TRAFFIC VOLUMES, THESE SIGNS SHOULD BE PLACED AT INTERSECTIONS (RATHER THAN MIDBLOCK LOCATIONS) SO THAT PEDESTRIANS AND BICYCLISTS ARE NOT CONFRONTED WITH MIDBLOCK WORK SITES THAT WILL INDUCE THEM TO ATTEMPT SKIRTING THE WORK SITE OR MAKING A MIDBLOCK CROSSING.

WHEN EXISTING PEDESTRIAN FACILITIES ARE DISRUPTED, CLOSED, OR RELOCATED, THE TEMPORARY FACILITIES SHALL BE DETECTABLE AND INCLUDE ACCESSIBILITY FEATURES CONSISTENT WITH THE FEATURES PRESENT IN THE EXISTING PEDESTRIAN FACILITY.

9. CHANGED STOP CONDITIONS:

ATTACH TWO FLAGS AND A RED TYPE "B" HIGH INTENSITY WARNING LIGHT TO ANY STOP SIGN THAT CREATES A NEW STOP CONDITION OR MOVES THE STOP CONDITION TO A NEW LOCATION. LEAVE FLAGS AND LIGHTS IN PLACE FOR AT LEAST THE FIRST 30 DAYS. INSTALL W3-1 (SYMBOLIC STOP AHEAD) SIGN IN ADVANCE OF STOP SIGN IF STOP SIGN IS NOT VISIBLE FOR A MINIMUM OF DISTANCE 'A' (SEE CHART ON TE710) OR IF STOP CONDITION IS MOVED TO LESS THAN DISTANCE 'A' FROM AN EXISTING STOP AHEAD SIGN.

10. LUMP SUM BIDDING:

WHEN TRAFFIC CONTROL IS BID LUMP SUM, ADDITIONAL DEVICES WILL BE PAID FOR AS EXTRA WORK.

11. NIGHTTIME LIGHTING:

WHEN NIGHTTIME WORK IS REQUIRED, FLOODLIGHTS SHOULD BE USED TO ILLUMINATE FLAGGER STATIONS, EQUIPMENT CROSSINGS, AND OTHER AREAS WHERE EXISTING LIGHTING IS NOT ADEQUATE FOR THE WORK TO BE PERFORMED SAFELY.

IN NO CASE SHALL FLOODLIGHTS BE PERMITTED TO CREATE A DISABLING GLARE FOR THE DRIVER. THE ADEQUACY OF THE FLOODLIGHT PLACEMENT AND ELIMINATION OF POTENTIAL GLARE SHOULD BE CHECKED BY DRIVING THROUGH THE PROJECT.

12. NCHRP REPORT 350 CRASHWORTHY REQUIREMENTS:

TRAFFIC CONTROL DEVICES SHALL MEET THE EVALUATION CRITERIA IN NCHRP REPORT 350 AS SUPPLEMENTED BY FHWA MEMORANDUM "IDENTIFYING ACCEPTABLE HIGHWAY SAFETY FEATURES," DATED JULY 25, 1997. AVAILABLE ON THE INTERNET AT http://safety.fhwa.dot.gov/roadway_dept/road_hardware/nchrp_350.htm

ANY DEVICE NOT ADDRESSED BY THE TE STANDARDS MAY BE APPROVED ON A CASE BY CASE BASIS BY THE ENGINEER. THE DEVICE SHALL BE ACCOMPANIED BY AND INSTALLED ACCORDING TO NCHRP REPORT 350.

THE CONTRACTOR SHALL:

1) PROVIDE TO THE ENGINEER A COPY OF THE MANUFACTURER'S SELF-CERTIFICATION THAT ANY CATEGORY 1 (i.e. - PLASTIC CONICAL DELINEATORS, TUBULAR MARKERS, DRUMS WITHOUT ATTACHMENTS) AND CATEGORY 2 (i.e. - PORTABLE SIGN STANDS (WITH SIGNS), TYPE II AND III BARRICADES, AND VERTICAL PANELS) DEVICES USED ON THE PROJECT ARE NCHRP REPORT 350 COMPLIANT.

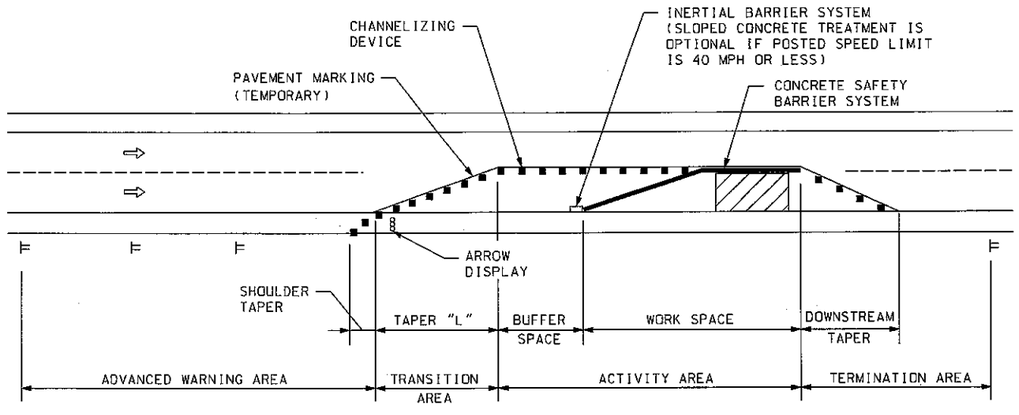
2) PROVIDE TO THE ENGINEER A COPY OF THE ENTIRE FHWA NCHRP REPORT 350 ACCEPTANCE LETTER (WZ-xxx) FOR ANY CATEGORY 2 DEVICE (i.e. - PORTABLE SIGN STANDS (WITH SIGNS), TYPE II AND III BARRICADES, AND VERTICAL PANELS) USED ON THE PROJECT. WORK ZONE FHWA NCHRP REPORT 350 ACCEPTANCE LETTERS (WZ-xxx) ARE AVAILABLE ON THE INTERNET AT:
http://safety.fhwa.dot.gov/roadway_dept/road_hardware/listing.cfm?code=workzone

3) CERTIFY THAT THE TRUCK MOUNTED ATTENUATORS (TMA'S) (WHICH ARE DEFINED AS CATEGORY 3 DEVICES BY THE FHWA MEMORANDUM) WERE PURCHASED PRIOR TO OCTOBER 1, 1998, AND INCLUDE A COPY OF THE ENTIRE FHWA ACCEPTANCE LETTER STATING THAT THE TMA'S ARE NCHRP REPORT 230 COMPLIANT; OR IF THE DEVICES WERE PURCHASED AFTER OCTOBER 1, 1998, INCLUDE A COPY OF THE ENTIRE FHWA'S ACCEPTANCE LETTER STATING THAT THE TMA'S ARE NCHRP REPORT 350 COMPLIANT.

ALL CATEGORY 1 & 2 DEVICES SHALL BE NCHRP REPORT 350 COMPLIANT. TMA'S, PURCHASED PRIOR TO OCTOBER 1, 1998, MAY BE USED UNTIL THE END OF THEIR SERVICEABLE LIVES.

13. TYPE "A" LOW INTENSITY WARNING LIGHTS:

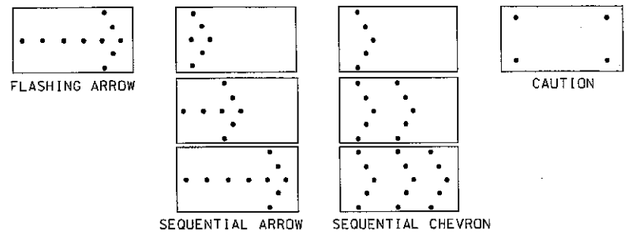
A TYPE "A" LOW INTENSITY WARNING LIGHT IS AN L.E.D. BI-DIRECTIONAL FLASHING WORK ZONE WARNING LIGHT.



TYPICAL WORK ZONE COMPONENTS

NOTE:
REFER TO STD. TE702 FOR TAPER "L" FORMULA.

ARROW DISPLAYS



ARROW DISPLAY ELEMENTS SHALL BE CAPABLE OF A MINIMUM 50 PERCENT DIMMING FROM THEIR FULL-RATED LAMP VOLTAGE. FULL LAMP VOLTAGE SHOULD BE USED DURING THE DAY AND DIMMED MODE SHALL BE USED AT NIGHT. FOR SHOULDER WORK, ROADSIDE WORK NEAR THE SHOULDER, BLOCKING THE SHOULDER, OR FOR TEMPORARY CLOSING ONE LANE ON A TWO-LANE, TWO-WAY ROADWAY, AN ARROW PANEL SHALL BE USED ONLY IN THE CAUTION MODE.

BUFFER SPACE

SPEED (MPH) *	20	25	30	35	40	45	50	55	60	65	70
LENGTH (ft)	115	155	200	250	305	360	425	495	570	645	730

NEITHER WORK ACTIVITY NOR STORAGE OF EQUIPMENT, VEHICLES, OR MATERIAL SHOULD OCCUR IN THE BUFFER SPACE. WHEN A PROTECTION VEHICLE IS PLACED IN ADVANCE OF THE WORK SPACE, ONLY THE SPACE UPSTREAM OF THE VEHICLE CONSTITUTES THE BUFFER SPACE.

* POSTED SPEED PRIOR TO WORK STARTING

IF TEMPORARY CONCRETE SAFETY BARRIER SYSTEM IS USED TO SEPARATE APPROACHING TRAFFIC FROM THE WORK SPACE, THE BARRIER SYSTEM SHALL BE CONSIDERED PART OF THE ACTIVITY AREA. A FULL LANE WIDTH SHOULD BE AVAILABLE THROUGHOUT THE LENGTH OF THE BUFFER SPACE. SEE TYPICAL WORK ZONE COMPONENTS.

NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	REVISED NOTE 7	J.M.	A.A.A.
2	8-8-07	ADD NOTE 13, REVISE NOTE 8 & WZ COMPONENTS	M.B.	A.A.A.
1	12-23-05	MODIFIED BUFFER SPACE TABLE	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

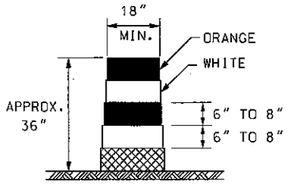
GENERAL TRAFFIC CONTROL

TE700 9/1/00

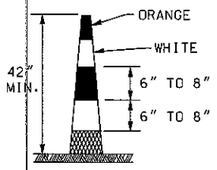
FHWA APPROVAL 4-20-09		APP'D Anthony A. Airgobire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : dluedke
Plot File : GAI1694\MicroTraffic\TE700.dgn
Plot Date : 10/28/2009
$$$KDOTGRP$$$.6th Floor

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	25	33



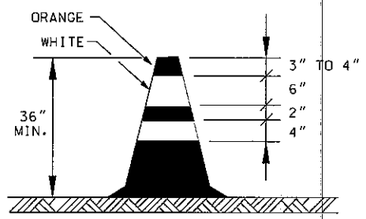
DRUM



CONICAL DELINEATOR

DRUMS AND CONICAL DELINEATORS SHALL HAVE AT LEAST TWO ORANGE AND TWO WHITE 6" TO 8" WIDE RETROREFLECTIVE STRIPES. ADDITIONAL STRIPES MAY BE NON-RETROREFLECTIVE. IF THERE ARE NON-RETROREFLECTIVE SPACES BETWEEN ADJACENT STRIPES, THEY SHALL BE NO MORE THAN 3" WIDE.

ALL RETROREFLECTIVE STRIPES ON DRUMS SHALL BE ASTM TYPE III SHEETING. THE WHITE STRIPES ON CONICAL DELINEATORS SHALL BE ASTM TYPE III SHEETING. ORANGE STRIPES ON ALL CONICAL DELINEATORS SHALL BE FLUORESCENT ORANGE ASTM TYPE IV SHEETING.



TRAFFIC CONE

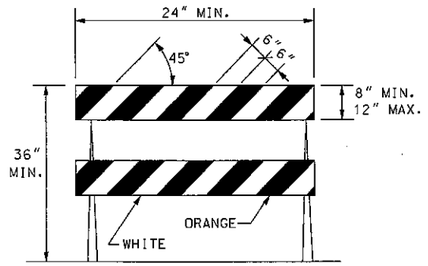
TRAFFIC CONES MAY BE USED AS CHANNELIZING DEVICES FOR DAYTIME OPERATIONS ONLY. THEY WILL NOT BE PAID FOR SEPARATELY, BUT WILL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS. THE ENGINEER MAY REQUIRE THAT TRAFFIC CONES BE SUPPLEMENTED BY OTHER TRAFFIC CONTROL DEVICES IN CERTAIN SITUATIONS.

ITEM \ LOCATION		CROSS-OVERS	SHOOFLY DIVERSIONS	TANGENTS	TAPERS	RAMPS	HEAD TO HEAD	OBJECT IDENTIFIER	
PORTABLE									
	DRUMS	YES	YES	YES	YES	YES	(1)	YES	
	CONICAL DELINEATORS	YES	YES	YES	YES	YES	(1)	YES	
	VERTICAL PANELS	(2)	(2)	(2)	(2)	(2)	(1,2)	YES	
	DIRECTION INDICATOR BARRICADE	NO	NO	NO	YES	NO	NO	NO	
	TYPE II BARRICADE	(2)	(2)	(2)	(2)	NO	NO	YES	
FIXED									
	TUBULAR MARKERS	(3)	(3)	(3)	NO	(3)	YES	NO	
	VERTICAL PANELS	(3)	(3)	(3)	(3)	(3)	(3)	YES	

(1) Not allowed on centerline delineation along freeways or expressways.

(2) The stripes shall slope downward to the traffic side for channelization.

(3) May be used upon the approval of the Engineer.

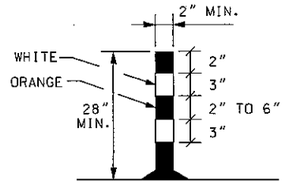


TYPE II BARRICADE

FOR RAILS LESS THAN 36" LONG, 4" WIDE STRIPES MAY BE USED.

THE ENTIRE AREA OF BARRICADE RAILS, BOTH FRONT AND BACK, SHALL BE ASTM TYPE III SHEETING.

THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.



TUBULAR MARKER

THE TWO WHITE RETROREFLECTIVE STRIPES SHALL BE ASTM TYPE III SHEETING.

TAPER FORMULAS:

L = WS FOR SPEEDS OF 45 MPH OR MORE

$L = WS^2/60$ FOR SPEEDS OF 40 MPH OR LESS

WHERE: L = MINIMUM LENGTH OF TAPER IN FEET
S = NUMERICAL VALUE OF POSTED SPEED PRIOR TO WORK STARTING IN MPH
W = WIDTH OF OFFSET IN FEET

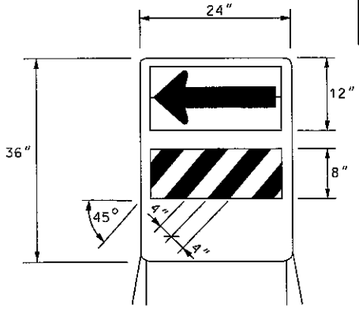
CHANNELIZER PLACEMENT:

(A) THE SPACING BETWEEN DEVICES IN TRANSITION AREA (TAPER) SHOULD NOT EXCEED A DISTANCE IN FEET EQUAL TO 1/2 THE POSTED SPEED LIMIT IN MPH PRIOR TO WORK STARTING.

(B) THE SPACING BETWEEN DEVICES IN THE ADVANCED WARNING AREA AND THE ACTIVITY AREA SHOULD NOT EXCEED A DISTANCE IN FEET EQUAL TO TWO TIMES THE POSTED SPEED LIMIT IN MPH PRIOR TO WORK STARTING.

(C) CHANNELIZING DEVICES SHALL BE PLACED FOR OPTIMUM VISABILITY, NORMALLY AT RIGHT ANGLES TO THE TRAFFIC FLOW.

(D) CHANNELIZING DEVICES PLACED ALONG SHOULDER EDGES OR IN DROPOFFS SHALL HAVE A MINIMUM OF 24" FROM THE TOP OF THE CHANNELIZING DEVICE TO THE TOP OF THE PAVEMENT.

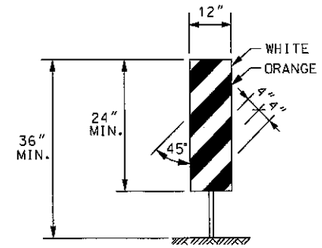


DIRECTION INDICATOR BARRICADE

THE ARROW PANEL SHALL BE BLACK ON FLUORESCENT ORANGE ASTM TYPE IV SHEETING. THE STRIPES SHALL BE ORANGE AND WHITE ASTM TYPE III SHEETING SLOPING DOWNWARD IN THE DIRECTION TRAFFIC IS TO PASS.

THE DIRECTION INDICATOR BARRICADE SHALL BE USED IN SERIES TO DIRECT THE MOTORIST INTO THE INTENDED LANE OF TRAVEL.

THE ARROW PANEL SHOULD NOT BE VISIBLE TO OPPOSING TRAFFIC.



VERTICAL PANEL

THE ENTIRE AREA OF VERTICAL PANELS, BOTH FRONT AND BACK, SHALL HAVE ASTM TYPE III SHEETING. THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.

NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	CHANNELIZER PLACEMENT & TRAFFIC CONE DETAIL	J.M.	A.A.A.
2	8-8-07	TRIMLINE CHANGED TO CONICAL DELINEATOR	M.B.	A.A.A.
1	11-15-03	CHANGED BORDER	B.H.	S.A.B.

KANSAS DEPARTMENT OF TRANSPORTATION

CHANNELIZING DEVICES

TE702 9/1/00

FHWA APPROVAL 4-20-09		APP'D Anthony A. Alrobaire	
DESIGNED L.E.R.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : cluedke
Plot File : 6x16x9\MicroStrat\Traffic\te702.dgn
Plot Date : 10/28/2009
$$$$KDOTGRP$$$$, 6th Floor

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	26	33

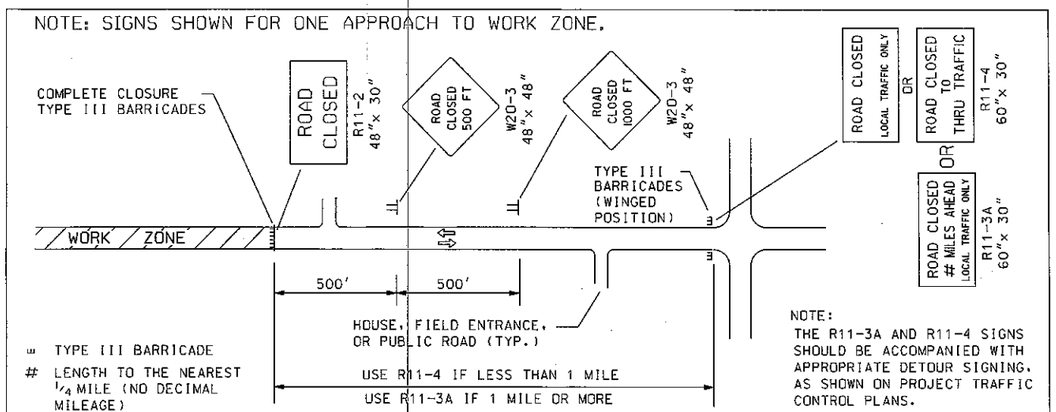


FIGURE 1: TYPICAL SIGNING FOR ROAD CLOSURE

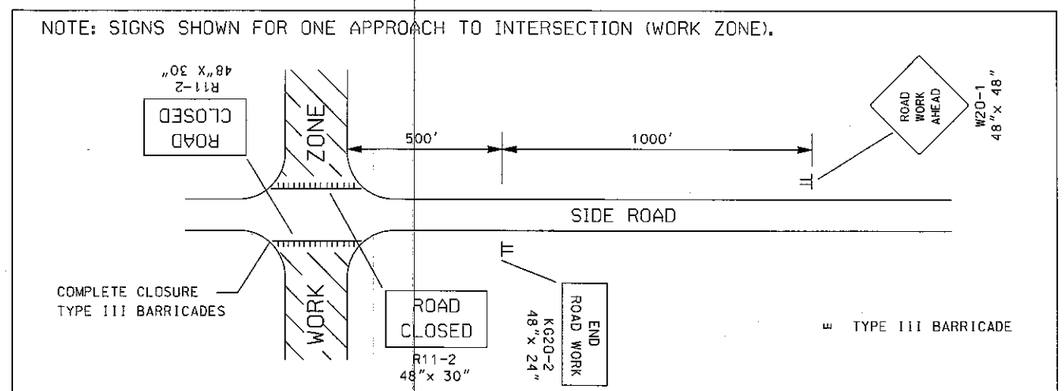


FIGURE 2: TYPICAL SIGNING FOR SIDE ROAD OPEN

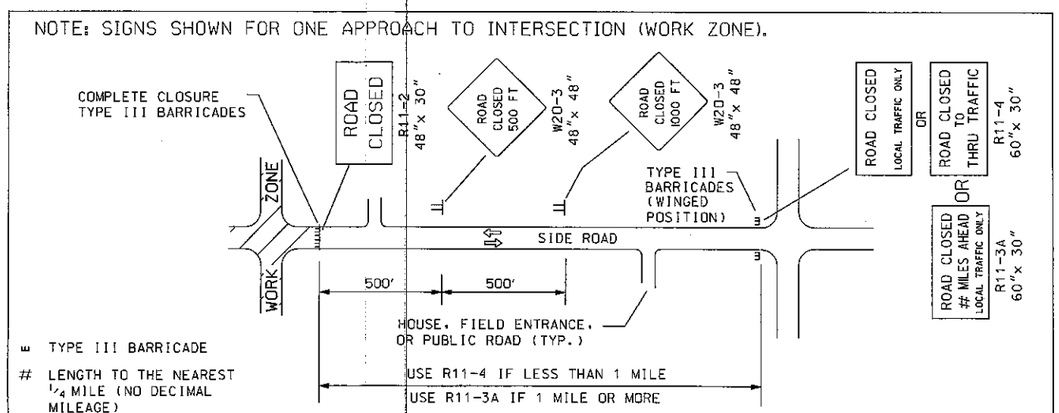


FIGURE 3: TYPICAL SIGNING FOR SIDE ROAD CLOSED

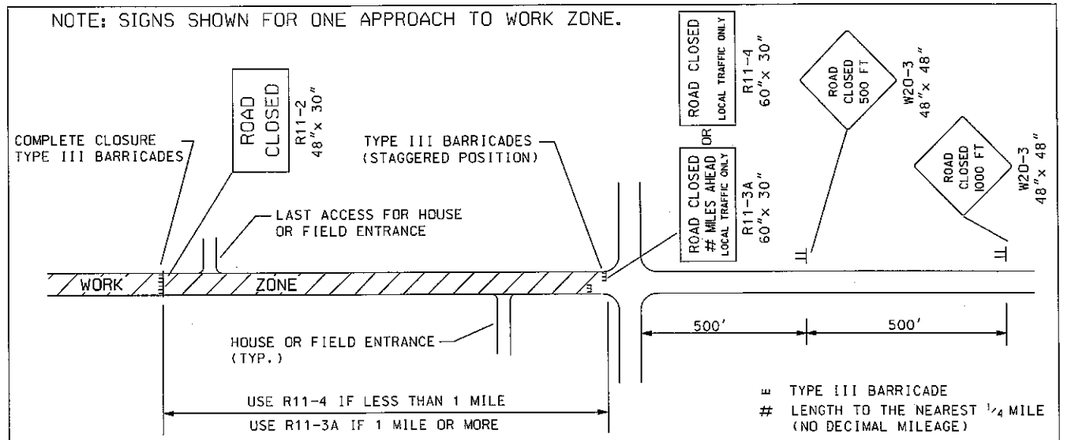


FIGURE 4: TYPICAL SIGNING FOR ROAD CLOSURE - LOCAL TRAFFIC ACCESS

NOTES:

1. SIGNS:

   THE R11-4 (ROAD CLOSED TO THRU TRAFFIC OR ROAD CLOSED LOCAL TRAFFIC ONLY) SIGN SHALL BE USED WHEN THE DISTANCE TO THE POINT OF COMPLETE CLOSURE OF THE ROADWAY IS LESS THAN 1 MILE.

   THE R11-3A (ROAD CLOSED # MILES AHEAD LOCAL TRAFFIC ONLY) SIGN SHALL BE USED WHEN THE DISTANCE TO THE POINT OF COMPLETE CLOSURE OF THE ROADWAY IS 1 MILE OR GREATER.

   THE WORDS "BRIDGE OUT" (OR BRIDGE CLOSED) MAY BE SUBSTITUTED FOR THE WORDS "ROAD CLOSED" ON THE R11-3A OR R11-4 SIGN WHERE APPLICABLE.

2. BARRICADE PLACEMENT:

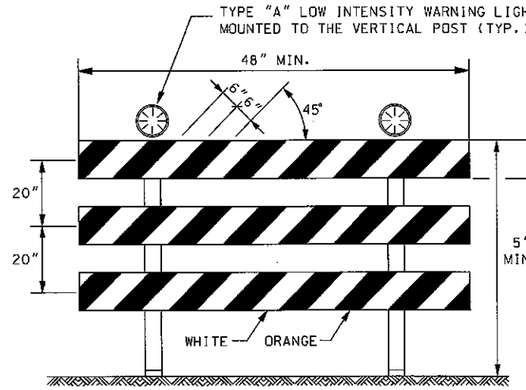
   A) COMPLETE ROAD CLOSURE

   WHEN A ROADWAY IS CLOSED, TYPE III BARRICADES SHALL BE PLACED END-TO-END TO COMPLETELY COVER THE ROADWAY AND SHOULDERS. WHEN ACCESS MUST BE ALLOWED FOR CONSTRUCTION OR OTHER OFFICIAL/GOVERNMENT VEHICLES, TYPE III BARRICADES SHALL BE LONGITUDINALLY STAGGERED FAR ENOUGH APART FROM ONE ANOTHER TO ALLOW SAFE PASSAGE OF VEHICLES AND MAINTAIN THE APPEARANCE OF A CLOSED ROADWAY. TYPE III BARRICADES SHALL BE REALIGNED AND PLACED END-TO-END TO DENY ANY ACCESS WHEN THE CONSTRUCTION ACTIVITY HAS CEASED FOR THE DAY.

   B) ROAD CLOSED - LOCAL TRAFFIC

   AS SHOWN IN FIGURE 4, WHEN LOCAL TRAFFIC MUST BE ALLOWED ACCESS INTO THE WORK ZONE, TYPE III BARRICADES SHALL BE LONGITUDINALLY STAGGERED TO MAINTAIN THE APPEARANCE OF A CLOSED ROADWAY. A SECOND LINE OF END-TO-END TYPE III BARRICADES SHALL BE PLACED JUST BEYOND THE LAST ACCESS POINT IN THE WORK ZONE, TO COMPLETELY CLOSE THE ROADWAY AS DESCRIBED IN NOTE 2-A.

   AS SHOWN IN FIGURE 1 AND FIGURE 3, AT THE POINT WHERE THRU TRAFFIC MUST DETOUR AND LOCAL TRAFFIC CAN PROCEED TO THE LOCATION WHERE THE ROADWAY IS COMPLETELY CLOSED, THE R11-3A (ROAD CLOSED # MILES AHEAD LOCAL TRAFFIC ONLY) OR R11-4 (ROAD CLOSED LOCAL TRAFFIC ONLY OR ROAD CLOSED TO THRU TRAFFIC) SIGN SHALL BE USED WITH TYPE III BARRICADES (WINGED POSITION), PLACED ON THE SHOULDERS OF ROADWAY.



TYPE III BARRICADE WITH LIGHTS

THE ENTIRE AREA OF BARRICADE RAILS, BOTH FRONT AND BACK, SHALL HAVE ASTM TYPE III SHEETING.

THE STRIPES SHALL SLOPE DOWNWARD TO THE SIDE TRAFFIC IS TO PROCEED OR TOWARD THE CENTER OF THE ROADWAY AT ROAD CLOSURES.

APPROVED SIGNS MOUNTED ON TYPE III BARRICADES SHOULD NOT COVER MORE THAN 50% OF THE TOP TWO RAILS OR 33% OF THE TOTAL AREA OF THE THREE RAILS.

WHEN BARRICADES ARE PLACED END-TO-END OR STAGGERED, A TYPE "A" LOW INTENSITY WARNING LIGHT SHALL BE MOUNTED TO THE VERTICAL POST NEAR EACH OUTSIDE CORNER OF THE END BARRICADES.

TYPE "A" LOW INTENSITY WARNING LIGHTS SHALL BE MAINTAINED SO AS TO BE VISIBLE ON A CLEAR NIGHT FROM A DISTANCE OF 3000'.

NO.	DATE	REVISIONS	BY	APP'D
3	8-8-07	ADDED POSITION TO TYPE III BARRICADES	M.B.	A.A.A.
2	12-29-05	MODIFIED NOTE #1	M.B.	A.A.A.
1	2-1-05	MODIFIED NOTES TYPE III BARRICADES WITH LIGHTS	B.M.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL TRAFFIC CONTROL
ROAD CLOSURES

TE704 9/26/02

FHWA APPROVAL 8-8-07		APP'D Anthony A. Alrobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : aluedke
Plot File : GV1169\MicroXTraffic\te704.dgn
Plot Date : 10/28/2009
$$$$KDOTGRP$$$$, 6th Floor

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	27	33

GENERAL NOTES

1. MAINTENANCE:

THE CONTRACTOR SHALL MAINTAIN ALL SIGNS AND DEVICES IN AN UPRIGHT POSITION. THE CONTRACTOR SHALL CLEAN OR REPLACE ANY DAMAGED OR ILLEGIBLE SIGN OR DEVICE AS DIRECTED BY THE ENGINEER.

2. EXISTING SIGNS:

IF EXISTING SIGNS THAT ARE TO REMAIN (WHETHER DENOTED ON THE PLANS OR NOT) INTERFERE WITH CONSTRUCTION WORK, THE CONTRACTOR SHALL REMOVE, STORE, AND RESET THE SIGNS. THIS SHALL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS. SIGNING DAMAGED BY THE CONTRACTOR SHALL BE REPLACED AT THE CONTRACTOR'S EXPENSE.

3. CONFLICTING SIGNS, SIGNS NOT IN USE, AND TRAFFIC SIGNALS:

SIGNS AND TRAFFIC SIGNALS THAT ARE IN CONFLICT WITH THE TRAFFIC CONTROL PLAN OR DO NOT APPLY TO THE TRAFFIC OPERATIONS SHALL BE IMMEDIATELY REMOVED, TURNED SO NOT VISIBLE TO TRAFFIC FROM ANY DIRECTION, OR COMPLETELY COVERED WITH ADEQUATE OPAQUE WATERPROOF MATERIAL. TAPE SHALL NOT BE APPLIED TO THE FACE OF THE SIGN.

4. PORTABLE AND POST MOUNTED SIGNS:

TEMPORARY TRAFFIC CONTROL SIGNS THAT ARE ANTICIPATED TO REMAIN IN PLACE FOR 3 DAYS OR LESS ARE CONSIDERED "PORTABLE." PORTABLE SIGNS SHALL BE MOUNTED ON AN APPROVED SUPPORT AT A MINIMUM HEIGHT OF 12" ABOVE THE TRAVELED WAY. TRAFFIC CONTROL SIGNS IN PLACE FOR OVER 3 DAYS ARE REQUIRED TO BE MOUNTED ON APPROVED POSTS. A MINIMUM OF 42" OF THE APPROVED POST MUST BE BELOW THE GROUND SURFACE WITH ADEQUATE BACKFILL AND COMPACTION. ALL POSTS AT MINIMUM SHALL EXTEND TO THE TOP EDGE OF THE SIGN AND NO GREATER THAN 6" ABOVE THE SIGN.

WHEN THE SIGN WIDTH IS EQUAL TO OR GREATER THAN 9', THREE OR MORE WOOD POSTS MAY BE USED WITH A MINIMUM OF 4' BETWEEN THE CENTERLINE OF EACH POST. ALL SIGNS LESS THAN 9' IN WIDTH SHALL USE A MAXIMUM OF TWO WOOD POSTS.

"ROLL-UP" SIGNS MAY BE USED FOR PORTABLE WARNING SIGNS. THEY MUST BE FLUORESCENT ORANGE ASTM TYPE IV SIGNS OF OPAQUE MATERIAL. MESH SIGNS ARE NOT ALLOWED.

5. SHEETING:

ALL ORANGE SIGNS SHALL HAVE FLUORESCENT ORANGE ASTM TYPE IV SHEETING. ALL OTHER SIGNS SHALL HAVE ASTM TYPE III SHEETING OF STANDARD COLORS.

6. SIGNS INVOLVING SPEEDS:

THE W3-5 (SPEED REDUCTION) SHOULD BE USED ONLY IF THE ENGINEER DETERMINES THAT A REDUCED SPEED IS REQUIRED ON THE PROJECT.

THE KM4-20 (WORK ZONE) PLAQUE SHALL BE PLACED ABOVE ALL SPEED LIMIT SIGNS, (R2-1), EXISTING AND TEMPORARY. MOUNT THE WORK ZONE PLAQUES TO THE POST. DO NOT OVERLAP THE R2-1 AND KM4-20 SIGNS.

FOR SPEEDS OF 30 MPH OR LESS, THE W1-1(TURN) OR W1-3(REVERSE TURN) SHOULD BE USED. FOR SPEEDS OF 35 MPH OR MORE, THE W1-2(CURVE) OR W1-4(REVERSE CURVE) SHOULD BE USED. THE W13-1(MPH) IS TO BE ELIMINATED IF THE ADVISORY SPEED IS WITHIN 5 MPH OF THE SPEED LIMIT.

7. SIGNS CONTROLLING WORK ZONE:

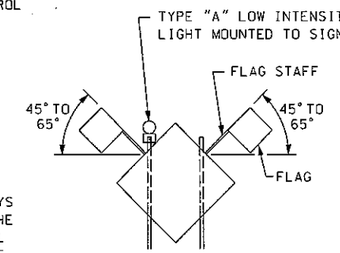
THE KG20-2(END ROAD WORK) SHOULD BE PLACED 500' FROM THE END OF THE ACTUAL WORK SPACE, NOT NECESSARILY AT THE EXTREME LIMITS OF THE PROJECT. THE KG20-2 SHOULD BE MOUNTED ON TWO POSTS. THE KG20-2 MAY BE MOUNTED ON ONE POST IF IN URBAN AREAS WHERE UTILITIES ARE A PROBLEM AND WIND LOADS ARE NOT AN ISSUE.

WHERE TWO WORK ZONES ARE LESS THAN 1 MILE APART IN RURAL AREAS OR 1/4 MILE APART IN URBAN AREAS, THE KG20-2(END ROAD WORK) FOR THE FIRST WORK ZONE AND THE W20-1(ROAD WORK) FOR THE SECOND WORK ZONE SHOULD BE ELIMINATED.

8. WARNING LIGHTS ON SIGNS:

TYPE "A" LOW INTENSITY WARNING LIGHTS SHOULD BE USED WITH ALL CONSTRUCTION ACTION WARNING SIGNS AND SHALL NOT BE USED ON SIGNS MOUNTED LESS THAN 5' HIGH ON TEMPORARY SUPPORTS. ON ALL OTHER CONSTRUCTION WARNING SIGNS, TYPE "A" LOW INTENSITY WARNING LIGHTS ARE TO BE USED AS DIRECTED BY THE ENGINEER.

TYPE "A" LOW INTENSITY WARNING LIGHTS SHALL BE MAINTAINED SO AS TO BE CAPABLE OF BEING VISIBLE ON A CLEAR NIGHT FROM A DISTANCE OF 3000 FT. IF A TYPE "A" LOW INTENSITY WARNING LIGHT HAS A SEPARATE BATTERY CASE, THE BATTERY CASE SHALL BE MOUNTED NO HIGHER THAN 12" ABOVE THE GROUND AND MOUNTED BEHIND THE SIGN POST. A TYPE "A" LOW INTENSITY WARNING LIGHT WHERE THE LENS AND BATTERY ARE ONE UNIT SHALL BE MOUNTED ON THE TEMPORARY SIGN POST NEAREST TO THE TRAVELED WAY. FLAGS SHALL NOT INTERFERE WITH THE VISABILITY OF THE TYPE "A" LOW INTENSITY WARNING LIGHT.



TWO (2) 18" x 18" FLUORESCENT RED-ORANGE FLAGS SHALL BE ATTACHED (IN THE POSITION SHOWN) ON THE W20-2(DETOUR), W1-1(TURN), W1-2(CURVE), W1-3(REVERSE TURN), W1-4(REVERSE CURVE), W3-3(SIGNAL AHEAD), W4-2(LANE REDUCTION), W20-4(ONE LANE ROAD), W20-5(LANE CLOSED), W20-7A(FLAGGER), AND W3-4 (BE PREPARED TO STOP) SIGNS AND ANY OTHER ACTION SIGNS AS SHOWN ON THE PLANS OR DIRECTED BY THE ENGINEER. THE FLAGS AND STAFFS ARE TO BE ATTACHED IN SUCH A MANNER THAT THE SIGN WILL NOT BE OBSCURED. THE FLAGS MAY BE EITHER A CLOTH OR VINYL MATERIAL. THE FLAGS SHALL BE SUBSIDIARY TO THE CONSTRUCTION SIGN BID ITEMS.

9. AFAD'S

FLAGGER SHALL BE A PERSON WHO PROVIDES TEMPORARY TRAFFIC CONTROL.

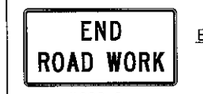
FLAGGER STATIONS SHALL BE LOCATED SUCH THAT APPROACHING ROAD USERS WILL HAVE SUFFICIENT DISTANCE TO STOP AT AN INTENDED STOPPING POINT.

THE FIVE CATEGORIES OF WORK DURATION AND THEIR TIME AT A LOCATION SHALL BE:

A. LONG-TERM STATIONARY IS WORK THAT OCCUPIES A LOCATION FOR MORE THAN 3 DAYS.
B. INTERMEDIATE-TERM STATIONARY IS WORK THAT OCCUPIES A LOCATION MORE THAN ONE DAYLIGHT PERIOD UP TO 3 DAYS, OR NIGHTTIME WORK LASTING LONGER THAN 1HOUR.
C. SHORT-TERM STATIONARY IS DAYTIME WORK THAT OCCUPIES A LOCATION FOR MORE THAN 1HOUR WITHIN A SINGLE DAYLIGHT PERIOD.
D. SHORT DURATION IS WORK THAT OCCUPIES A LOCATION UP TO 1HOUR.
E. MOBILE IS WORK THAT MOVES INTERMITTENTLY OR CONTINUOUSLY.

ONE LANE, TWO-WAY VEHICULAR TRAFFIC FLOW REQUIRES AN ALL-RED INTERVAL OF SUFFICIENT DURATION FOR ROAD USERS TO CLEAR THE PORTION OF THE TEMPORARY TRAFFIC CONTROL ZONE CONTROLLED BY THE TRAFFIC CONTROL SIGNALS. SAFEGUARDS SHALL BE INCORPORATED TO AVOID THE POSSIBILITY OF CONFLICTING SIGNAL INDICATIONS AT EACH END OF THE TEMPORARY TRAFFIC CONTROL ZONE.

SIGN LAYOUT INFORMATION



KG20-2
(BLACK ON ORANGE)

STD. SIZE EXPWY/FREEWAY
6" C
48"x 24"



KM4-20
(BLACK ON ORANGE)

STD. SIZE	EXPWY/FREEWAY
3" C	6" C
24"x 6"	48"x 12"

NOTE:
TEXT DIMENSIONS ARE IN INCHES.
BORDER IS BLACK NON-REFLECTIVE.
SEE STD. TE730 OR TE731 FOR KG20-5 SIGN LAYOUT.

LETTER SIZES FOR BLACK ON ORANGE "DESTINATION" SIGNS

STD. SIZE	EXPWY/FREEWAY
6" C	10" D



COLORS: W7-3a

LEGAND - BLACK (NON-REFLECTIVE)
BACKGROUND - ORANGE (REFLECTIVE)

MILEAGE TO BE DETERMINED BY THE ENGINEER.



W8-11

COLORS:
LEGAND - BLACK (NON-REFLECTIVE)
BACKGROUND - ORANGE (REFLECTIVE)



W8-9a

COLORS:
LEGAND - BLACK (NON-REFLECTIVE)
BACKGROUND - ORANGE (REFLECTIVE)

THIS SIGN IS TO BE INSTALLED FOR CONDITIONS OF HEIGHT DIFFERENTIAL BETWEEN ADJACENT LANES OVER ONE (1) INCH. IT IS TO BE PLACED AT EACH INTERSECTING CROSSROAD, MAJOR TRAFFIC GENERATOR, OR AT APPROXIMATELY ONE-HALF (1/2) MILE INTERVALS. THE SIGNS ARE TO BE REMOVED OR COVERED WHEN NOT APPLICABLE.

MINIMUM ADVANCE WARNING SIGN SPACING (IN FEET):

	A	B	C
URBAN (40 MPH OR LOWER)	*100*	*100*	*100*
URBAN (45 MPH OR HIGHER)	*350*	*350*	*350*
RURAL (55 MPH OR LOWER)	*500*	*500*	*500*
RURAL (60 MPH OR HIGHER)	*750*	*750*	*750*
EXPRESSWAY/FREEWAY	*1000*	*1500*	*2640*

THE MINIMUM SPACING BETWEEN SIGNS SHALL BE NO LESS THAN 100', UNLESS DIRECTED BY THE ENGINEER.

THE SPACING BETWEEN ANY SIGNS MAY BE INCREASED BEYOND THE MINIMUM VALUES IN THE TABLE ABOVE AS APPROVED BY THE ENGINEER IN ORDER TO MAXIMIZE VISIBILITY.

NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	ADDED AFAD NOTE & NEW SIGN LAYOUT	J.M.	A.A.A.
2	8-8-07	MODIFIED NOTES 4, 5, 6, 7 & 8, KG20-2 LAYOUT	M.B.	A.A.A.
1	12-29-05	MODIFIED FLAGS, M4-20 & SIGN LAYOUT INFO	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

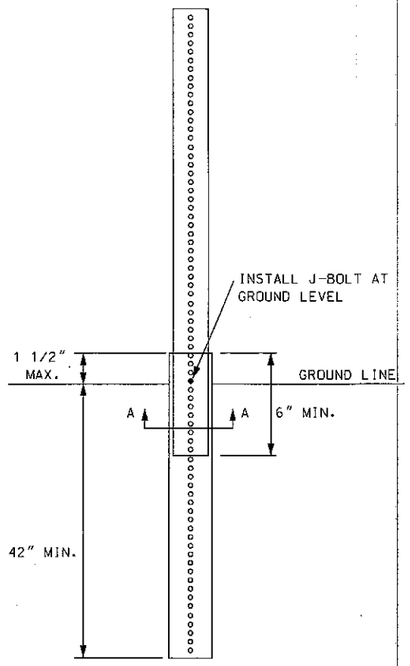
TRAFFIC CONTROL SIGNS

TE710 9/1/00

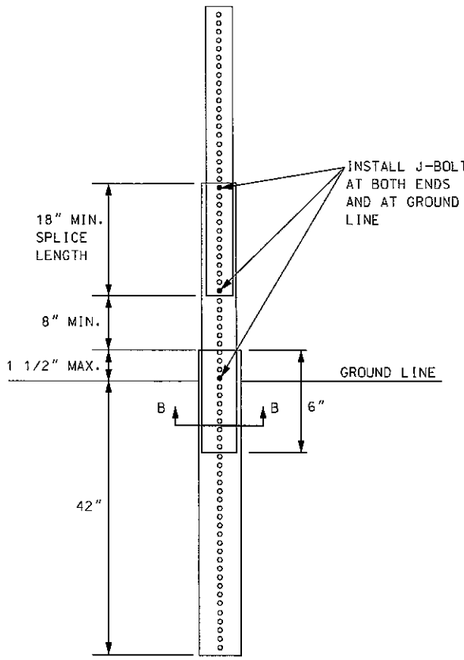
FHWA APPROVAL 4-20-09		APP'D Anthony A. Alrabaiy	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : *duedke* $$$KDOTGRP$$$$_6th Floor
Plot File : *GV1694\MicroNTraffic\te710.dgn*
Plot Date : *10/28/2009*

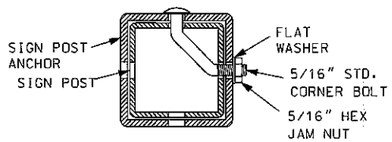
PERFORATED SQUARE STEEL TUBE (P.S.S.T.) POST SETUP



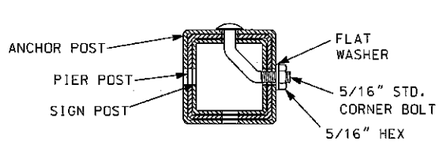
P.S.S.T. DETAIL



TELESCOPING P.S.S.T. DETAIL



SECTION A-A



SECTION B-B

DETAILS FOR 2", 2 1/4", OR 2 1/2" SIGN POST

NOTE:

SIGN POSTS, POST ANCHORS, AND POST ANCHOR SLEEVES:

THE SIGN POSTS, POST ANCHORS, AND POST ANCHOR SLEEVES SHALL CONFORM TO (1) HOT ROLLED CARBON STEEL ASTM A-570 GRADE 33 OR ASTM STRUCTURAL QUALITY C 1021 WITH A HOT-DIPPED GALVANIZING CONFORMING TO ASHTO M-120 WITH AN INLINE SYSTEM FOLLOWED BY A CHROMATE CONVERSION AND A CROSS-LINKED POLYURETHANE ACRYLIC EXTERIOR COATING OR (2) A GALVANIZED FINISH COLD ROLLED CARBON STEEL ASTM A-446 GRADE A WITH A GALVANIZING CONFORMING TO ASTM A 525, DESIGNATION G-90.

THE SIGN POST SHALL HAVE 7 /16" DIA. HOLES OR 7 /16" DIA. EMBOSSED RING (DIE-CUT KNOCKOUTS) AT 1" CENTERS ON ALL FOUR SIDES FOR THE TOTAL LENGTH OF THE POST. THE CORNER BOLTS, NUTS, AND WASHERS SHALL BE BID SUBSIDIARY.

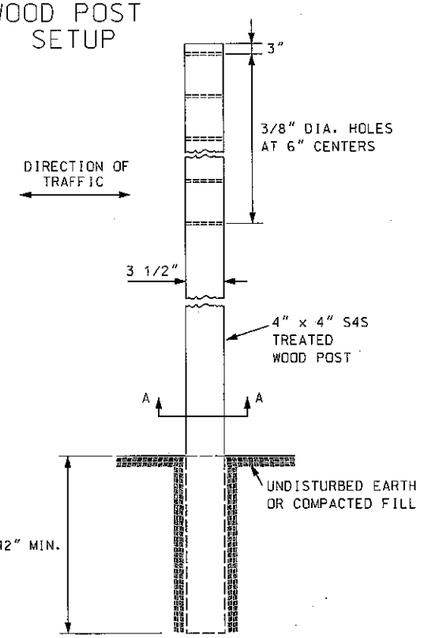
ALL HOLES SHALL BE DRILLED OR PUNCHED AND ALL WELDS, CUTS, BURRS, AND SHARP EDGES ARE TO BE SMOOTHED OFF BEFORE APPLICATION OF FINISH.

BASIS OF ACCEPTANCE:

ACCEPTING OF THE STEEL POSTS, ANCHORS, SLEEVES, AND FASTENERS FURNISHED TO COMPLY WITH THE ABOVE REQUIREMENTS WILL BE BASED ON RECEIPT AND APPROVAL OF A TYPE D CERTIFICATION AS DESCRIBED IN SECTION 2600 OF THE STANDARD SPECIFICATIONS FOR STATE ROAD AND BRIDGE CONSTRUCTION (2007) AMD VISUAL INSPECTION FOR CONDITION AND DIMENSIONAL REQUIREMENTS.

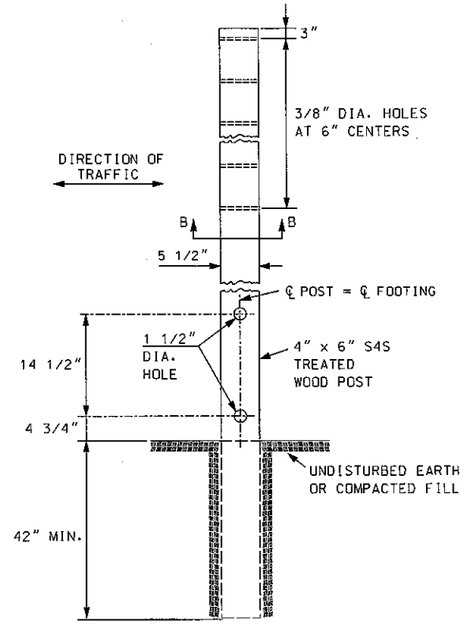
Plotted By : claelke
Plot File : 6V1694\MicroV Traffic\Va712.dgn
Plot Date : 10/28/2009
$$$$DOTGRP$$$$, 6th Floor

WOOD POST SETUP



SIDE ELEVATION

4" X 4" WOOD POST IN SOIL



SIDE ELEVATION

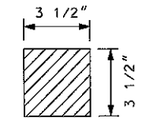
4" X 6" WOOD POST IN SOIL

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	28	33

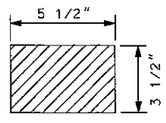
NOTE:

ALL SIGN MOUNTING HOLES IN THE WOOD POSTS SHALL BE DRILLED PRIOR TO TREATING.

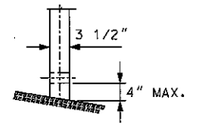
BREAKAWAY HOLES AND FIELD CUTS SHALL BE TREATED IN ACCORDANCE WITH THE PRESERVATIVE TREATMENT SPECIFICATIONS.



SECTION A-A

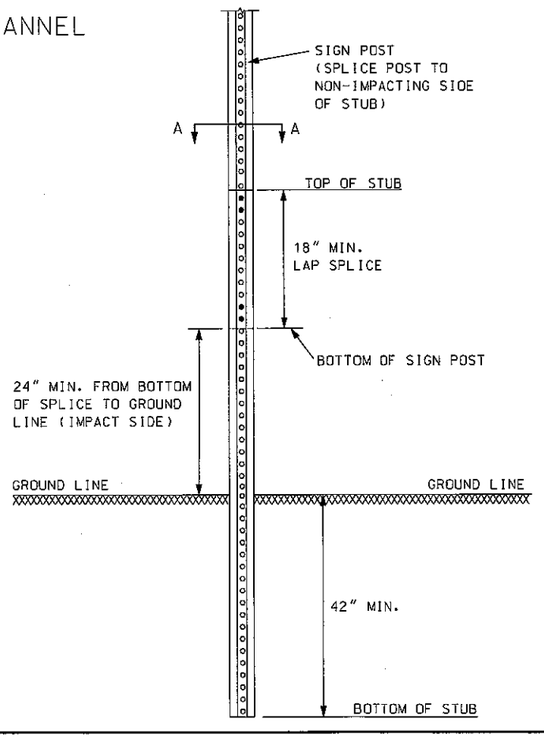


SECTION B-B



FRONT ELEVATION

3 LB/F U-CHANNEL SETUP



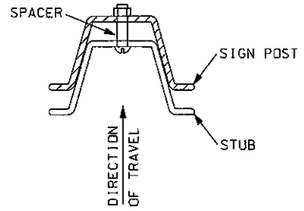
NOTE:

THE LAP SPLICE IS 18" MIN. AND IS FASTENED BY FOUR 5 /16" DIA. GALVANZIED A449 BOLTS (SAE J429 GRADE 5) OR GALVANIZED A325 BOLTS.

TWO BOLTS AT BOTH ENDS OF THE SPLICE THROUGH THE HOLES NEAREST THE ENDS OF THE SPLICE.

SPACERS WILL BE USED OVER THE BOLTS BETWEEN THE SPLICED PIECES OF U-CHANNEL. THREADED OR UNTHREADED SPACERS MAY BE USED. DO NOT SUBSTITUTE PIPE OR OTHER "THINGS" FOR THE SPACERS.

SECTION A-A



NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1	4-20-09	REVISED J-BOLT DETAIL	J.M.	A.A.A.

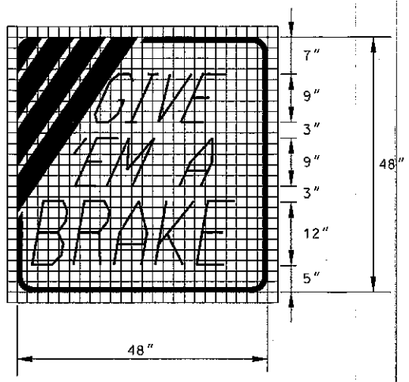
KANSAS DEPARTMENT OF TRANSPORTATION

APPROVED TEMPORARY POST SETUPS

TE712 7/3/07

FHWA APPROVAL	4-20-09	APP'D Anthony A. Alrobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	29	33

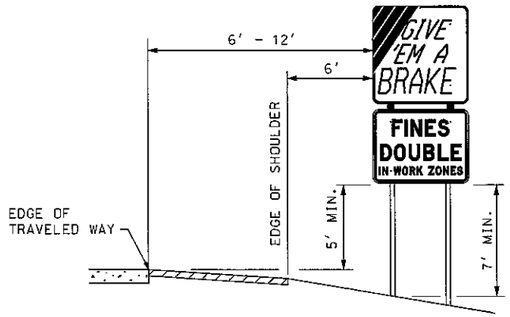


KI-104a

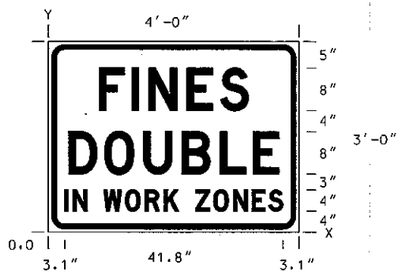
COLORS:
- BACKGROUND - BLACK
- BORDER - WHITE
- LEGEND - WHITE
- STRIPES - ORANGE

BORDER WIDTH - 1"
STRIPE WIDTH - 4"
CORNER RADIUS - 3"

LEGEND:
" GIVE " - DUTCH 801 ROMAN SWC - 25 DEGREE SLANT
" 'EM A " - DUTCH 801 ROMAN SWC - 25 DEGREE SLANT
" BRAKE " - DUTCH 801 ROMAN SWC - 10 DEGREE SLANT



HEIGHT AND LATERAL PLACEMENT



KI-105a

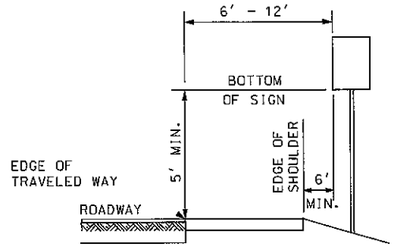
SIGN NUMBER	FINES DOUBLE
WIDTH x HEIGHT	4'-0" x 3'-0"
BORDER WIDTH	0.9"
CORNER RADIUS	3.0"
MOUNTING	GROUND
BACKGROUND	TYPE: REFLECTIVE
	COLOR: WHITE
LEGEND/BORDER	TYPE: NON-REFLECTIVE
	COLOR: BLACK

DIMENSIONS IN INCHES

SPACINGS ARE TO START OF NEXT LETTER

Y FONT	LETTER SPACINGS																								HT LEN
23.0 D		F	I	N	E	S																			8.0
	9.7	6.4	3.2	7.3	6.4	5.4	9.7																		28.6
11.0 D		D	O	U	B	L	E																		8.0
	3.9	6.9	7.5	7.3	7.3	6.4	4.9	3.9																	40.3
4.0 D		I	N		W	O	R	K		Z	O	N	E	S											4.0
	3.1	1.6	2.7	3.2	4.3	3.8	3.6	2.8	3.2	3.4	3.8	3.6	3.2	2.7	3.1										41.8

HEIGHT AND LATERAL DIMENSIONS FOR GROUND MOUNTED SIGNS
(SIGNS LEFT IN PLACE OVER 3 DAYS)



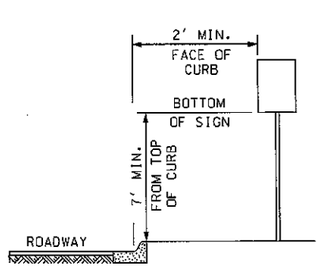
RURAL DISTRICT

RURAL:
1) GROUND-MOUNTED SIGNS SHALL BE MOUNTED AT A MINIMUM HEIGHT OF 5' MEASURED FROM THE BOTTOM OF SIGN TO THE NEAR EDGE OF THE PAVEMENT.

2) LARGE SIGNS HAVING AN AREA EXCEEDING 50 SQUARE FEET INSTALLED ON MULTIPLE BREAKAWAY POSTS SHALL BE MOUNTED A MINIMUM OF 7' ABOVE THE GROUND.

3) THE HEIGHT FROM THE BOTTOM OF THE SECONDARY SIGN MOUNTED BELOW ANOTHER SIGN MAY BE 4' MEASURED FROM THE BOTTOM OF THE SIGN TO THE NEAR EDGE OF THE PAVEMENT. SIGNS SHALL NOT OVERLAP EACH OTHER.

4) SIGN SUPPORTS SHALL BE CRASHWORTHY.



URBAN DISTRICT

URBAN:
1) SIGNS SHALL BE MOUNTED AT A MINIMUM HEIGHT OF 7' MEASURED FROM THE BOTTOM OF SIGN TO THE NEAR EDGE OF THE PAVEMENT.

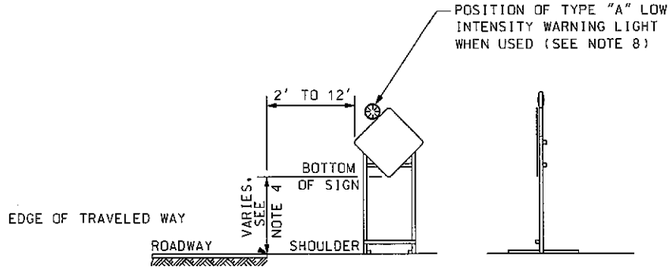
2) NEITHER PORTABLE NOR PERMANENT SIGN SUPPORTS SHOULD BE LOCATED ON SIDEWALKS OR AREAS DESIGNATED FOR PEDESTSRIAN OR BICYCLE TRAFFIC.

3) SIGNS MOUNTED LOWER THAN 7' SHOULD NOT PROJECT MORE THAN 4" INTO PEDESTRIAN FACILITIES.

4) THE HEIGHT FROM BOTTOM OF THE SECONDARY SIGN MOUNTED BELOW ANOTHER SIGN MAY BE 6' MEASURED FROM THE BOTTOM OF SIGN TO THE NEAR EDGE OF THE PAVEMENT. SIGNS SHALL NOT OVERLAP EACH OTHER.

5) LARGE SIGNS HAVING AN AREA EXCEEDING 50 SQUARE FEET INSTALLED ON MULTIPLE BREAKAWAY POSTS SHALL BE MOUNTED A MINIMUM OF 7' ABOVE THE GROUND.

6) SIGN SUPPORTS SHALL BE CRASHWORTHY.



NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	ADDED DETAIL FOR SKID MOUNTED SIGNS	J.M.	A.A.A.
2	8-8-07	KI04 & KI05 CHANGE, ADDED GROUND MOUNT	M.B.	A.A.A.
1	12-29-05	ADD K TO KI04 & KI05	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

GROUND AND SKID MOUNTED DETAIL DETAILS FOR "GIVE 'EM A BRAKE" AND "FINES DOUBLE" SIGNS

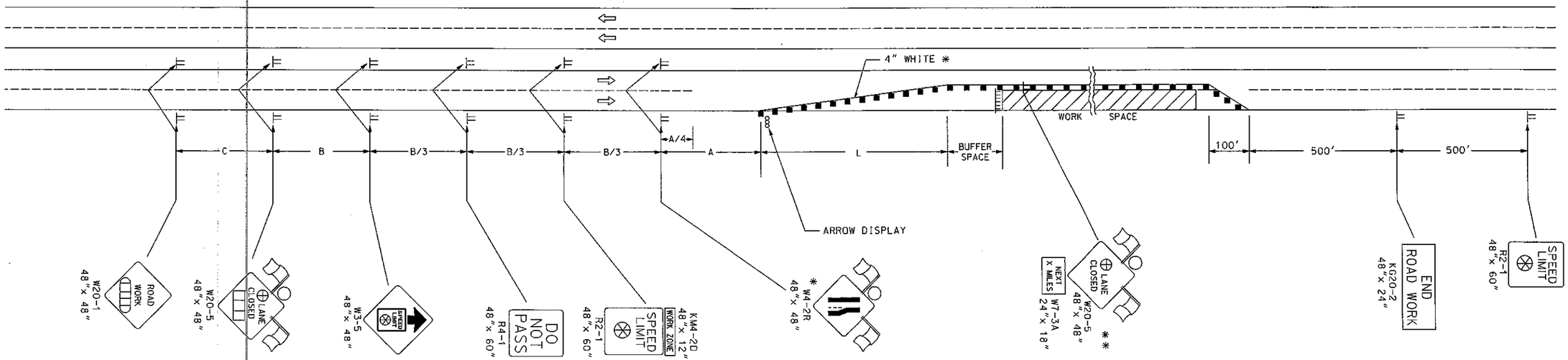
TE714 9/1/00

FHWA APPROVAL 8-8-07		APP'D Anthony A. Airobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : cluedke
Plot File : GN1694MicroNTraffic\Te714.dgn
Plot Date : 10/28/2009
$$$$KDOTGRP$$$$_6th Floor

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	30	33

REFER TO STD. TE710 FOR ADDITIONAL INFORMATION ON TEMPORARY TRAFFIC CONTROL SIGNS AND SIGN SPACING.
REFER TO STD. TE704 FOR TYPE III BARRICADES.
REFER TO STD. TE702 FOR INFORMATION ON TAPERS AND CHANNELIZING DEVICES.
REFER TO STD. TE700 FOR LENGTH OF BUFFER SPACE.



LEFT-SIDE SIGNS SHALL BE OMITTED FOR A FOUR-LANE UNDIVIDED HIGHWAY.

* FOR LEFT LANE CLOSURES USE W4-2L AND YELLOW EDGE LINE ALONG CHANNELIZING DEVICES.

** THE W20-5 (⊕ LANE CLOSED) AND W7-3A (NEXT X MILES) SIGNS SHOULD BE PLACED AT 2 MILE INCREMENTS ON A PROJECT OF 4 MILES OR LONGER.

- Type III Barricades
- X Length To The Nearest Whole Mile
- ■ Channelizing Device
- AHEAD, 1500 FT. OR 1 MILE
- AHEAD, 1000 FT. 1500 FT. OR 1/2 MILE
- ⊕ RIGHT OR LEFT
- ⊗ Speed To Be Determined By The Engineer
- ○ Type "A" Low Intensity Warning Light

NO.	DATE	REVISIONS	BY	APP'D
3	8-8-07	SIGN SPACING CHANGE	M.B.	A.A.A.
2	12-29-05	M4-20 CHANGED TO KM4-20	M.B.	A.A.A.
1	2-1-05	CLARIFIED NOTES, UPDATED WARNING SIGNS	B.H.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION
TYPICAL TRAFFIC CONTROL
FOUR-LANE HIGHWAY
ONE LANE CLOSED

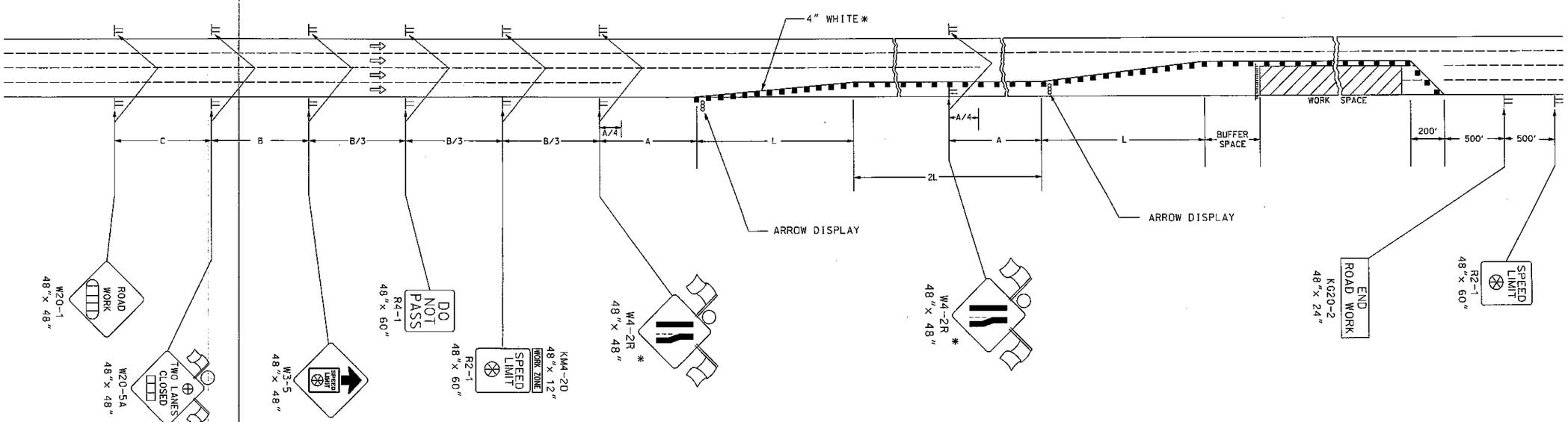
TE744 9/1/00

FHWA APPROVAL 8-8-07		APP'D Anthony A. Alrobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : cluedke $$$KDOTGRP$$$$, 6th Floor
Plot File : G:\V1694\Micro\Traffic\te744.dgn
Plot Date : 10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	31	33

REFER TO STD. TE710 FOR ADDITIONAL INFORMATION ON TEMPORARY TRAFFIC CONTROL SIGNS AND SIGN SPACING.
REFER TO STD. TE704 FOR TYPE III BARRICADES.
REFER TO STD. TE702 FOR INFORMATION ON TAPERS AND CHANNELIZING DEVICES.
REFER TO STD. TE700 FOR LENGTH OF BUFFER SPACE.



* FOR LEFT LANE CLOSURES USE W4-2L AND YELLOW EDGE LINE ALONG CHANNELIZING DEVICES.

Type III Barricades
X Length To The Nearest Whole Mile
■ Channelizing Device
AHEAD, 1500 FT, OR 1 MILE
AHEAD, 1000 FT, 1500 FT, OR 1/2 MILE
RIGHT OR LEFT
Speed To Be Determined By The Engineer
Type "A" Low Intensity Warning Light

NO.	DATE	REVISIONS	BY	APP'D
3	8-8-07	SIGN SPACING CHANGE	M.B.	A.A.A.
2	12-29-05	M4-20 CHANGED TO KM4-20	M.B.	A.A.A.
1	2-1-05	CLARIFIED NOTES, UPDATED WARNING SIGNS	B.H.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION
TYPICAL TRAFFIC CONTROL
MULTI-LANE HIGHWAY
TWO LANES CLOSED

TE746 9/1/00

FHWA APPROVAL 8-8-07		APP'D Anthony A. Alrobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : cluedke $$$$KDOTGRP$$$$, 6th Floor
Plot File : GV1694MicroVTrafficVer46.dgn
Plot Date : 10/28/2009

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	32	33

SUMMARY OF TRAFFIC CONTROL DEVICES (EACH)

WORK ZONE SIGN (SPECIAL)		
SIGN NO.	16.25 SQ.FT. & LESS	16.26 SQ.FT. & OVER

SUMMARY OF TRAFFIC CONTROL DEVICES (EACH PER DAY)

* QUANTITY MOST USED ON THE PROJECT AT ANY ONE TIME

WORK ZONE SIGNS *			
SIGN NO.	SIZE - SQ.FT.		
	0-9.25	9.26-16.25	16.26 & OVER
W20-7A		2	
KG20-2	8		
KM4-20	4		
R2-1 (35)			4
R2-1 (45)			2
R3-2	3		
R4-1			4
W3-5		4	
W4-2		8	
W12-1	1		
W20-1		9	
W20-5a		4	

BARRICADES *	CHANNELIZING DEVICES *	
TYPE III (4' TO 12')	FIXED	PORTABLE
6		150

LIGHTED DEVICES *	
WORK ZONE WARNING LIGHT (TYPE "A" LOW INTENSITY)	16
WORK ZONE WARNING LIGHT (RED TYPE "B" HIGH INTENSITY)	
ARROW DISPLAY	4
PORTABLE CHANGEABLE MESSAGE SIGN	2

RECAPITULATION OF QUANTITIES

ITEM	QUANTITY	UNIT
WORK ZONE SIGNS (0 TO 9.25 SQ.FT.)	1200	EACH PER DAY
WORK ZONE SIGNS (9.26 TO 16.25 SQ.FT.)	1725	EACH PER DAY
WORK ZONE SIGNS (16.26 SQ.FT. & OVER)	750	EACH PER DAY
WORK ZONE BARRICADES (TYPE III - 4' TO 12')	450	EACH PER DAY
CHANNELIZER (FIXED)		EACH PER DAY
CHANNELIZER (PORTABLE)	11250	EACH PER DAY
WORK ZONE WARNING LIGHT (TYPE "A" LOW INTENSITY)	1200	EACH PER DAY
WORK ZONE WARNING LIGHT (RED TYPE "B" HIGH INTENSITY)		EACH PER DAY
ARROW DISPLAY	300	EACH PER DAY
PORTABLE CHANGEABLE MESSAGE SIGN	150	EACH PER DAY
PAVEMENT MARKING (TEMPORARY)		
4" SOLID (TYPE I TAPE)		STA./LINE
4" SOLID (TYPE II TAPE)		STA./LINE
4" BROKEN (8.2') (TYPE I TAPE)		STA./LINE
4" BROKEN (8.2') (TYPE II TAPE)		STA./LINE
4" BROKEN (3.3') (TYPE I TAPE)		STA./LINE
4" BROKEN (3.3') (TYPE II OR IIA TAPE)		STA./LINE
4" BROKEN(3.3')(FLEXIBLE RAISED PAVEMENT MARKER)		STA./LINE
4" DOTTED EXTENSION (TYPE I TAPE)		STA./LINE
4" DOTTED EXTENSION (TYPE II TAPE)		STA./LINE
SOLID (LINE MASKING TAPE)		STA./LINE
BROKEN (LINE MASKING TAPE)		STA./LINE
PAVEMENT MARKING REMOVAL		LIN. FT.
GLARE SCREEN		LIN. FT.
CONCRETE SAFETY BARRIER (TYPE F3)(TEMPORARY)		LIN. FT.
CONCRETE SAFETY BARRIER (TYPE F3)(TEMP.-INSTALL ONLY)		LIN. FT.
CONCRETE SAFETY BARRIER (TYPE F3)(TEMP.-RELOCATE)		LIN. FT.
INERTIAL BARRIER SYSTEM		EACH
REPLACEMENT MODULES (F200)		EACH
REPLACEMENT MODULES (F400)		EACH
REPLACEMENT MODULES (F700)		EACH
REPLACEMENT MODULES (F1400)		EACH
REPLACEMENT MODULES (F2100)		EACH
WORK ZONE SIGN (SPECIAL) (16.25 SQ. FT. & LESS)		EACH
WORK ZONE SIGN (SPECIAL) (16.26 SQ. FT. & MORE)		EACH
TEMPORARY RAISED PAVEMENT MARKER (TYPE I)		EACH
TEMPORARY RAISED PAVEMENT MARKER (TYPE II)		EACH
TRAFFIC SIGNAL INSTALLATION (TEMPORARY)		LUMP SUM
TRAFFIC CONTROL (INITIAL SET UP)	LUMP SUM	LUMP SUM
TRAFFIC CONTROL		LUMP SUM
FLAGGER (SET PRICE)	1	HOUR

NO.	DATE	REVISIONS	BY	APP'D
3	8-8-07	2007 SPEC REVISION	M.B.	A.A.A.
2	12-29-05	ADDED TYPE A & RED TYPE B LIGHTS	B.H.	A.A.A.
1	2-1-05	CONST. SIGN SPECIAL CHART	B.H.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

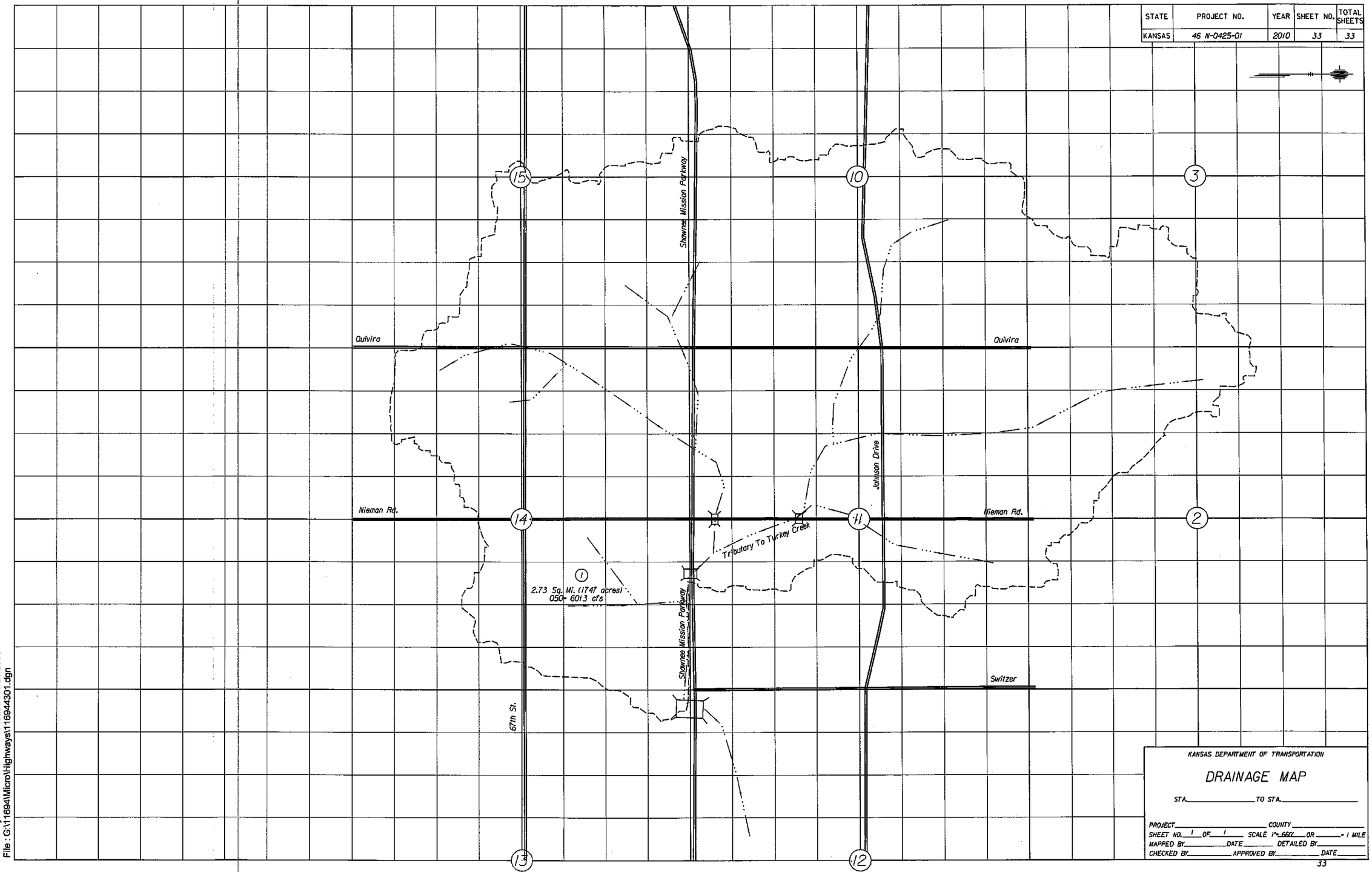
SUMMARY OF DEVICES AND RECAPITULATION OF QUANTITIES

TE795 9/1/00

FHWA APPROVAL 8-8-07		APP'D Anthony A. Alrobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted By : dtaeke $$$$KDOTGRP$$$$.6th Floor
Plot File : GV1694\MicroSTraffic\te795.dgn
Plot Date : 3/26/2010

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	46 N-0425-01	2010	33	33



KANSAS DEPARTMENT OF TRANSPORTATION

DRAINAGE MAP

STA._____ TO STA._____

PROJECT_____ COUNTY_____

SHEET NO. 1 OF 1 SCALE 1"= 660' OR _____= 1 MILE

MAPPED BY_____ DATE_____ DETAILED BY_____

CHECKED BY_____ APPROVED BY_____ DATE_____

33

Drawn By : cluedke Plotted : 10/28/2009
File : G:\11694\Micro\Highways\116944301.dgn

	BY	DATE
SURVEYED	Andrew Riddle-GBA	2012
PLOTTED	Chad Luedke-GBA	2012
INKED		2013
DESIGNED	Chad Luedke, Derek Harth-GBA	2013
SQUAD	Tod Salfrank, Michael Ingalls-KDOT	

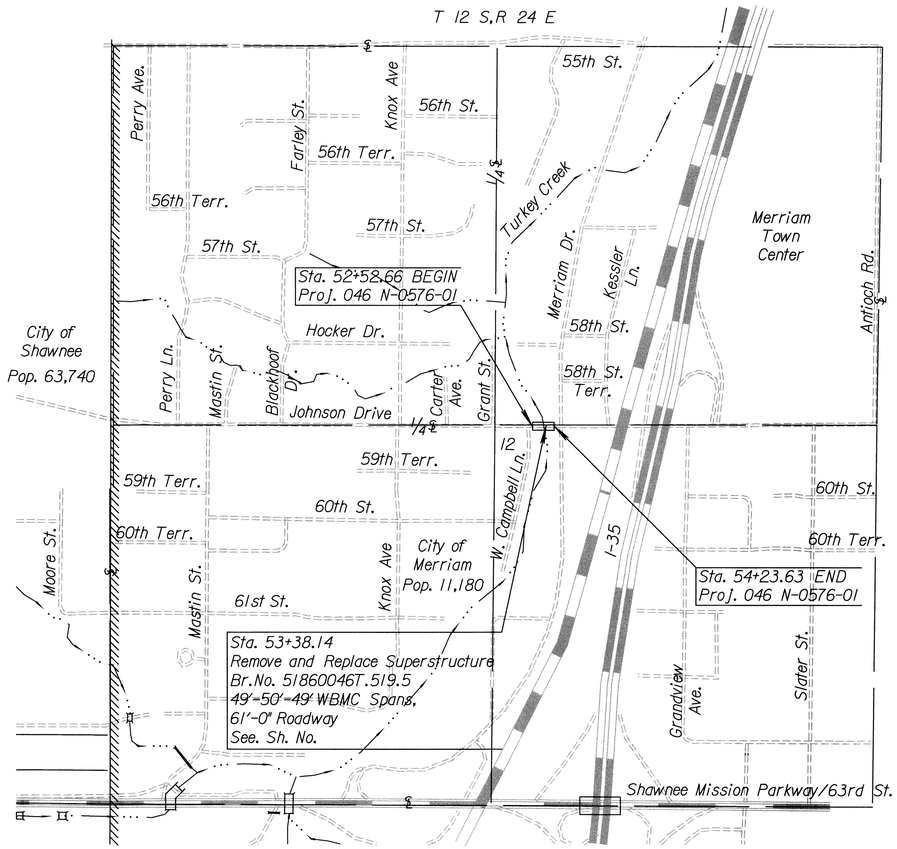
INDEX OF SHEETS

1	Title Sheet
2	General Notes and Details
3	Plan-Profile Sheet
4	Intersection Details
5 - 9	Pavement Details
10 - 12	Retaining Walls
13 - 37	Bridge No. 51860046T.519.5
38	Bridge Excavation
39	Supports and Spacers for Reinforcing Steel
40	Summary of Quantity Sheets
41 - 48	Temporary Erosion and Pollution Control (Soil Erosion)
49	Seeding
50 - 55	Lighting
56 - 71	Signing and Pavement Marking
72	Project Surfacing
73	Construction Sequence
74 - 86	Traffic Control
87	Concrete Safety Barrier (TEMPORARY)
88	Inertial Barrier- General Configuration
89 - 99	Cross Sections

STATE OF KANSAS
DEPARTMENT OF TRANSPORTATION
PLAN AND PROFILE OF PROPOSED
46 N-0576-01
FEDERAL AID PROJECT
JOHNSON COUNTY
CITY OF MERRIAM

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	1	99
F.A. NO.	STP-N057(601)			

Grading and Surfacing
Bridge
Fencing
Seeding
Sodding
Lighting
Signing and Pavement Marking



SCALE 1"= 500'

Note: Traffic to be detoured as shown in the construction sequence.

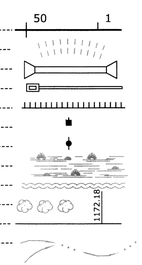
DESIGN DESIGNATION

AADT (2013)	17,000
AADT (2033)	20,000
DHV	8%
D	65/35
T	2%
V	35 mph
C of A	None

CONVENTIONAL SIGNS

COUNTY LINE
CITY LIMITS
STATE OR NATIONAL LINE
TOWNSHIP, SECTION or GRANT LINE
PROPERTY LINE
HIGHWAY FENCE
EXISTING FENCE
GUARDRAIL
CONSTRUCTION LIMITS
RIGHT OF WAY LINE
TRAVELED WAY
RAILROADS

CENTER LINE OF PROJECT
TERRACE
CULVERTS
DROP INLET & STORM SEWER
ACCESS CONTROL
POWER POLE
TELEPHONE POLE
MARSH
HEDGE
TREES
PROFILE ELEVATION
STREAM or CREEK



GROSS LENGTH OF PROJECT	170.97 FT.	
EXCEPTIONS	FT.	
ADDITIONS	FT.	
NET LENGTH OF PROJECT	170.97 FT.	0.032 MILES
NET LENGTH OF BRIDGES	150.97 FT.	0.029 MILES
NET LENGTH OF ROAD	20.00 FT.	0.004 MILES

DEREK W. HARTH LICENSED 17477 KANSAS PROFESSIONAL ENGINEER 9/13/13

RECOM. FOR APPROVAL-DATE 9-13-13

LOCAL PUBLIC OFFICIAL

KANSAS DEPARTMENT OF TRANSPORTATION

GENERAL NOTE

THE GEOLOGICAL INFORMATION SHOWN ON THESE PLANS IS FROM STUDIES MADE IN THE FIELD AND REPRESENTS THE BEST INFORMATION AVAILABLE TO THE KANSAS DEPARTMENT OF TRANSPORTATION.

AT BORROW AREA LOCATIONS ADJACENT TO THE RIGHT OF WAY, UTILITY POLES MAY BE SET AT THE PERMANENT LOCATIONS PRIOR TO CONSTRUCTION AS APPROVED BY THE ENGINEER PROVIDED A MINIMUM VERTICAL CLEARANCE, IN ACCORDANCE WITH THE NATIONAL ELECTRICAL SAFETY CODE, IS OBTAINED. THE CONTRACTOR WILL BE REQUIRED TO WORK AROUND THESE POLES TO COMPLETE THE WORK.

ALL BORROW TO BE OBTAINED FROM AREAS PROVIDED BY THE CONTRACTOR SHALL BE APPROVED BY THE ENGINEER, BOTH AS TO SUITABILITY OF MATERIAL AND SITE LOCATION. LOCATIONS WHICH, IN THE OPINION OF THE ENGINEER, CONTAIN UNSUITABLE MATERIAL OR WILL LEAVE AN UNSIGHTLY APPEARANCE ON THE PROJECT WILL NOT BE APPROVED.

CHANNELS SHALL BE CUT AT BOX CULVERTS (UNLESS OTHERWISE NOTED) TO FLOW LINE ELEVATIONS AND TO A WIDTH OF ONE FOOT OUTSIDE OF EACH OUTSIDE WALL AND WITH SLOPES 2 TO 1 PRIOR TO CONSTRUCTION OF THE CULVERT.

EMBANKMENT QUANTITIES FOR INITIAL CONSOLIDATION AND SETTLEMENT SHOWN IN THE EARTHWORK QUANTITIES ARE SUBSIDIARY TO OTHER EARTHWORK ITEMS. MATERIAL FOR THE EMBANKMENT IS INCLUDED IN THE EXCAVATION QUANTITIES.

EXCAVATION REQUIRED FOR PLACING SELECT SOIL IS INCLUDED IN THE COMMON EXCAVATION QUANTITIES.

WHERE EASEMENTS ARE SHOWN ON RAILROAD RIGHT OF WAY, THE CONTRACTOR SHALL BE REQUIRED TO WORK AROUND AND NOT DISTURB THE RAILROAD COMMUNICATION OR SIGNAL POLES OR LINES.

EXCAVATION SHOWN TO BE WASTED SHALL BE WASTED ON SITES PROVIDED BY THE CONTRACTOR. THESE SITES SHALL BE APPROVED BY THE ENGINEER AS TO SUITABILITY, APPEARANCE, AND SITE LOCATION. LOCATIONS THAT, IN THE OPINION OF THE ENGINEER, WILL LEAVE AN UNSIGHTLY APPEARANCE WILL NOT BE APPROVED.

ALL TREES, HEDGE ROWS, SHELTERBELTS, AND WOODY SHRUBS NOT SHOWN TO BE REMOVED AND LOCATED BETWEEN THE CONSTRUCTION LIMITS AND THE RIGHT-OF-WAY LINE OR EASEMENT LINE SHALL BE SPARED UNLESS DIRECTED BY THE ENGINEER TO BE REMOVED. ALL TREES WINTHIN THE APPROPRIATE CLEAR ZONE SHALL BE REMOVED.

ALL SAW CUTS SHALL BE FULL DEPTH AND SHALL NOT BE PAID FOR DIRECTLY, BUT SHALL BE CONSIDERED SUBSIDIARY TO OTHER ITEMS IN THE CONTRACT.

CONSTRUCTION ACTIVITIES WILL NOT IMPACT THE STREAM BED OF TURKEY CREEK. FORMWORK, EQUIPMENT, TEMPORARY SHORING, COFFERDAMS, ARE NOT ALLOWED WITHIN THE STREAM BED.

THE CITY OF MERRIAM WILL REMOVE THE DECORATIVE STREET LIGHTS LOCATED ON EACH CORNER OF THE EXISTING BRIDGE PRIOR TO CONSTRUCTION. NEW STREET LIGHT POLES AND LUMINARES WILL BE PROVIDED AND INSTALLED BY THE CITY OF MERRIAM AFTER COMPLETION OF THE PROJECT. THE CONTRACTOR WILL INSTALL ALL CONDUITS, WIRE, AND FOOTINGS. SEE BRIDGE AND AESTHETIC TREATMENT PLANS.

TEMPORARY EROSION CONTROL SHALL BE PLACED ACCORDING TO KDOT STANDARD SPECIFICATIONS. CURB INLET PROTECTION SHALL BE PLACED AT THE FIRST CURB INLET LOCATION UPSTREAM AND DOWNSTREAM OF THE PROJECT LIMITS. CURB INLET PROTECTION WILL BE PLACED ALONG THE PEDESTRIAN TRAIL UNDERNEATH THE BRIDGE, ALONG EXCAVATION AREAS ON ABUTMENT 2, AND RETAINING WALL LOCATIONS.

THE CONTRACTOR SHALL CONTACT DAN MILLER AT THE CITY OF OVERLAND PARK KANSAS AND PROVIDE A MINIMUM OF SEVEN (7) DAYS NOTICE SO THAT THE CITY CAN REMOVE THE STORM WATER MONITORING SYSTEM LOCATED AT THE SOUTHEAST CORNER OF THE BRIDGE.

THE CONTRACTOR SHALL RE-ESTABLISH THE JOHNSON COUNTY BENCHMARK #825 LOCATED ON THE SOUTHEAST CORNER OF THE EXISTING BRIDGE OVER TURKEY CREEK ACCORDING TO THE REQUIREMENTS AND SPECIFICATIONS OF JOHNSON COUNTY PUBLIC WORKS. ALL WORK AND MATERIALS REQUIRED TO RE-ESTABLISH THE BENCHMARK SHALL BE INCLUDED UNDER THE BID ITEM "CONTRACTOR CONSTRUCTION STAKING (LUMP SUM)". SEE THE KDOT STANDARD SPECIFICATIONS FOR DETAILS.

THE CONTRACTOR SHALL CONTACT KCP&L A MINIMUM OF 3 DAYS PRIOR TO ANY OVERHEAD WORK BEING PERFORMED (I.E. STEEL GIRDER PLACMENT) TO ALLOW KCP&L TO PROVIDE AN ON-SITE REPRESENTATIVE WHILE THE WORK IS BEING PERFORMED.

ALL CONCRETE CONSTRUCTION SHALL BE "KCMMB 4K".

THE CONTRACTOR SHALL CONTACT THE CITY OF MERRIAM TO FINALIZE THE SOD PLACEMENT LIMITS FOR THE PROJECT. LANDSCAPING PLANTINGS AT THE CORNERS OF THE BRIDGE WILL BE PROVIDED AND INSTALLED BY THE CITY OF MERRIAM.

Utility Owners

Power	Kansas City Power and Light Gary Price 16215 W. 108th Street Lenexa, KS 66219-1347 913-681-7420
Telephone	AT&T Russ Claybaker 5400 Foxridge Room 500 Mission, KS 66202 913-676-1846
Telephone	Quest Communications Brian Cornish 5799 Broadmoor Street Suite 700 Mission, KS 66202
Gas —G—	Kansas Gas Service Tony Cellitti 11401 West 89th Street Overland Park, KS 66214 913-599-8981
San. Sewer	Johnson County Wastewater Charles McAllister 4800 Nall Avenue Mission, KS 66062 913-715-8537
Water	Water District One of Johnson County Gerry Sherlock 10747 Renner Boulevard Lenexa, KS 66219 913-895-5774
Stream Monitoring System	City of Overland Park, Kansas Dan Miller 8500 Santa Fe Drive Overland Park, KS 66212 913-895-6032

Project Survey Control:

Horiz. Proj. Datum:
North American Datum of 1983 (NAD83)
Projection: Kansas State Plan North

Proj. Coord. x 0.99992120607 = NGS State Plane Coord. NAD83

Vertical Datum:
Assumed Datum.
Units: US Survey Feet

Datum Benchmark:
Johnson County Public Works BM #825
2" Aluminum JCPW monument top of 15" Conc. wingwall at S.E. corner Johnson Drive Bridge spanning Turkey Creek.
Elevation= 1006.06 ft.

NOTE: An assumed elevation of 1006.06 ft was used on the Project Benchmark "JOCO BM 825". The published NAVD88 elevation for the "JOCO BM 825" is 915.13 ft. To get to the North American Vertical Datum of 1988 subtract 90.93 ft from all plan elevations called out in the plan set.

Project Assummed Elevations - 90.93 ft. = NAVD88 Datum Elevation.

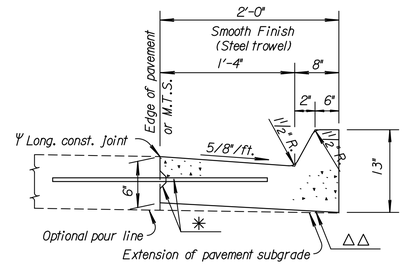
Drawn By : cluedke
Plotted : 9/13/2013
File : G:\12664\MicroHighways\N057601rgn-01_generalnote.dgn

GENERAL NOTE

Combined curb and gutter or gutter adjoining concrete pavement may, at the contractor's option, be constructed either monolithically or separately, using KCMMB 4K. The combined curb and gutter or gutter shall have the same section as shown on the plans. If constructed monolithically, the longitudinal joint and tie bars shall be omitted from the combined curb and gutter or gutter. Pavement joints shall be continued through curb or gutter and no other planes of weakness will be required. Joints in the combined curb and gutter or gutter are to be filled with the same material as used for the pavement joints.
Expansion joints in the combined curb and gutter are to be placed opposite expansion joints in the pavement.
Where combined curb and gutter or gutter does not abut concrete pavement or concrete base course, omit tie bars and place a 1" Preformed Expansion Joint Filler (Type B) cut to the dimensions of the combined curb and gutter or gutter, at a spacing not to exceed 250' and at the ends of curb returns. Planes of weakness shall be constructed at 10'-0" intervals.
A 4' length of transition from normal gutter section to the tapered gutter section shall be used at the ends of each run of gutter except where the gutter abuts a curb, such as at the end of a bridge. Inlets shall be located so as not to fall within this transition section.
Where pressure relief joint is placed across the pavement, and gutter or curb and gutter is continued on for more than 10', use 4"x4" membrane sealant installed with bonding adhesive through gutter section, shaped to fit gutter or curb and gutter. See Std. Drawing RD712.
For expansion joint treatment where combined curb and gutter or gutter abuts a bridge wing on a U-type abutment see bridge drawings.
Longitudinal joints shall be sawed and sealed with joint sealant, see Standard Specifications.

Y If constructed monolithically, the longitudinal joint is not required.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	2	99

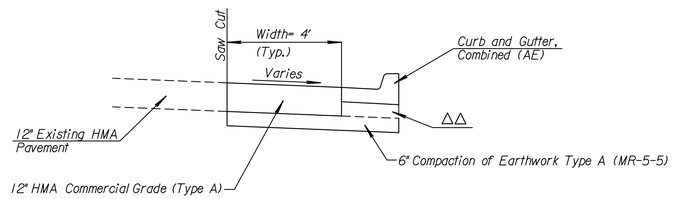


TYPICAL SECTION

CURB AND GUTTER, COMBINED (AE)

△△ *Contractor has the option of thickening the curb and gutter as shown.*

✳ *Longitudinal construction joint and #4 x 3'-0" bars @ 2'-6" ctrs., where concrete pavement is constructed.*



△△ *Contractor has the option of thickening the curb and gutter as shown.*

PAVEMENT WIDENING DETAIL

KANSAS DEPARTMENT OF TRANSPORTATION

GENERAL NOTES AND DETAILS

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	3	99

Control Point 100
55.51' Lt. of ℄ Sta. 52+00.25
N= 268,090.11 E= 2,251,231.51
1. Set 1/2" bar with GBA cap on property at 9404 Johnson Drive at NW quadrant of bridge.
2. Back asphalt curb east side of parking lot. 1.85' W.
3. Near face of steel bird house pole. 14.10' N.E.
4. ℄ manhole. 7.55' E.N.E.

Control Point 101
232.90' Rt. of ℄ Sta. 55+01.31 Elev.=1002.25'
N= 267,815.85 E= 2,251,545.52
1. Set 1/2" bar with GBA cap on property at 5911 Merriam Drive.
2. Near Face North Sign post Merriam Feed and Garden Center. 8.38' E.
3. Northeast corner curb inlet east side Merriam Drive. 13.00' N.N.W.
4. ℄ Water meter. 3.20' W.

℄ Sta. 48+00
N 268,016.26
E 2,250,834.24

℄ P.O.T. Sta. 52+00.00
N 268,034.65 E 2,251,233.82
1. Not Set.

Sta. 52+52.66 BEGIN
Proj. 046 N-0576-01

℄ P.O.T. Sta. 55+00.00
N 268,048.44 E 2,251,533.50
1. Not Set.

Control Point 102
71.59' Lt. of ℄ Sta. 54+98.33 Elev.=1003.51'
N= 268,119.88 E= 2,251,528.54
1. Chiselled "+" cut top back of curb at corner handicap parking stalls on property 9320 Johnson Drive.
2. Near face 6" maple tree in island. 8.45' N.E.
3. Near face at top of 48" diameter concrete planter. 4.45' W.
4. North edge concrete handicap walk. 7.76' S.

① Merriam Drainage District of Johnson County Kansas TOL NE 1/4 Sec. 12, T12S, R24E

② The Merriam Christian Church Lots 4,5,6 Merriam Addition to Campbellton

NOTE: THE CITY OF MERRIAM WILL REMOVE THE DECORATIVE STREET LIGHTS LOCATED ON EACH CORNER OF THE EXISTING BRIDGE PRIOR TO CONSTRUCTION. NEW STREET LIGHT POLES AND LUMINARES WILL BE PROVIDED AND INSTALLED BY THE CITY OF MERRIAM AFTER COMPLETION OF THE PROJECT. THE CONTRACTOR WILL INSTALL ALL CONDUITS, WIRE, AND FOOTINGS.

Note: The Contractor shall not disturb the existing pocket park planting area located on the N.E. corner of the the bridge.

℄ Sta. 59+00
N 268,066.83
E 2,251,933.08

Center Cor. Sec. 12, T12S, R23E
0.00' Lt. of ℄ Sta. 50+00.00
N 268,025.45 E 2,251,034.03
1. Found 1/2" bar 0.15' below centerline asphalt travelway Johnson Dr.
2. Chiseled plus cut in top of curb. 29.24' NNE
3. PK nail and plastic survey washer west face power pole. 94.53' ENE
4. Cross on northeast flange bolt fire hydrant. 51.90' ESE

N 1/4 Cor. Sec. 12, T12S, R23E
2661.57' Lt. of ℄ Sta. 50+00.00 NE ∡ = 89°31'13"
N 270,685.14 E 2,250,933.91
1. Found ⅝" bar with 2.5" aluminum cap stamped Affinis Corp KS CLS#133 in concrete 0.6' below surface.
2. Chiseled plus cut in gutter of curb. 5.22' NNE
3. Axle at property corner. 25.02' SSE
4. PK nail and JCPW washer north face power pole/street light pole. 10.53' SSW

CURVE DATA- MERRIAM LANE
P.I. Sta. 11+10.26 Bk. = 11+09.87 Ahd.
Δ= 11°46' 44" (RT) L= 113.07'
R= 550.00' E= 2.92'
T= 56.73'

BM #825 2" Aluminum Johnson County Public Works monument at S.E. corner Johnson Drive Bridge over Turkey Creek. 41.26' Rt. ℄ Sta. 54+12.80 Elevation= 1006.06'

BM #103 Chiselled "□" top N.E. corner curb inlet west side Merriam Drive. 1st inlet south of Johnson Dr. 223.25' Rt. ℄ Sta. 54+34.39 Elevation= 1001.74'

East 1/4 Cor. Sec. 12, T12S, R23E
0.00' Lt. of ℄ Sta. 76+74.86
N 268,148.44 E 2,253,706.06
1. Found bar with 2" Aluminum cap 0.10' below asphalt travelway.
2. Chiseled plus cut top north end nose of island. 51.89' SSW
3. Chiseled plus cut top west end nose of island. 50.20' ESE
4. Chiseled plus cut top south end nose of island. 53.43' NNE

Sta. 53+38.14
Remove and Replace Superstructure
Br.No. 51860046T.519.5
49'-50'-49' WBMC Spans,
61'-0" Roadway
See. Sh. No. 13-37

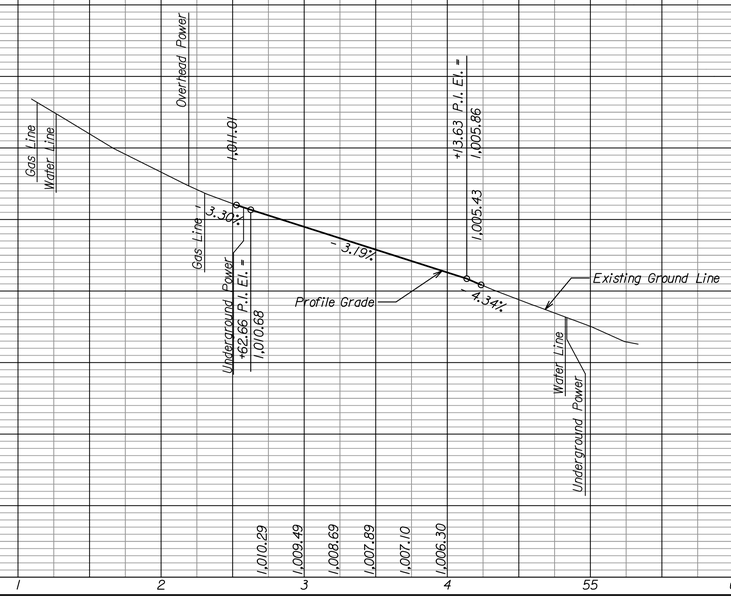
Sta. 54+23.63 END
Proj. 046 N-0576-01

SCALE
50' 0 50' 100'
PLAN: Lat. & Long.
PROFILE: Horiz. same as above
Vert.
5' 0

DATE	BY	REFERENCES NOTED	REFERENCES CHECKED
3-2013	Chad Luedke-GBA	Chad Luedke-GBA	
3-2013			

Drawn By : cluedke
File : G:\12664\MicroHighways\N057601rpl-01.dgn
Plotted : 9/13/2013

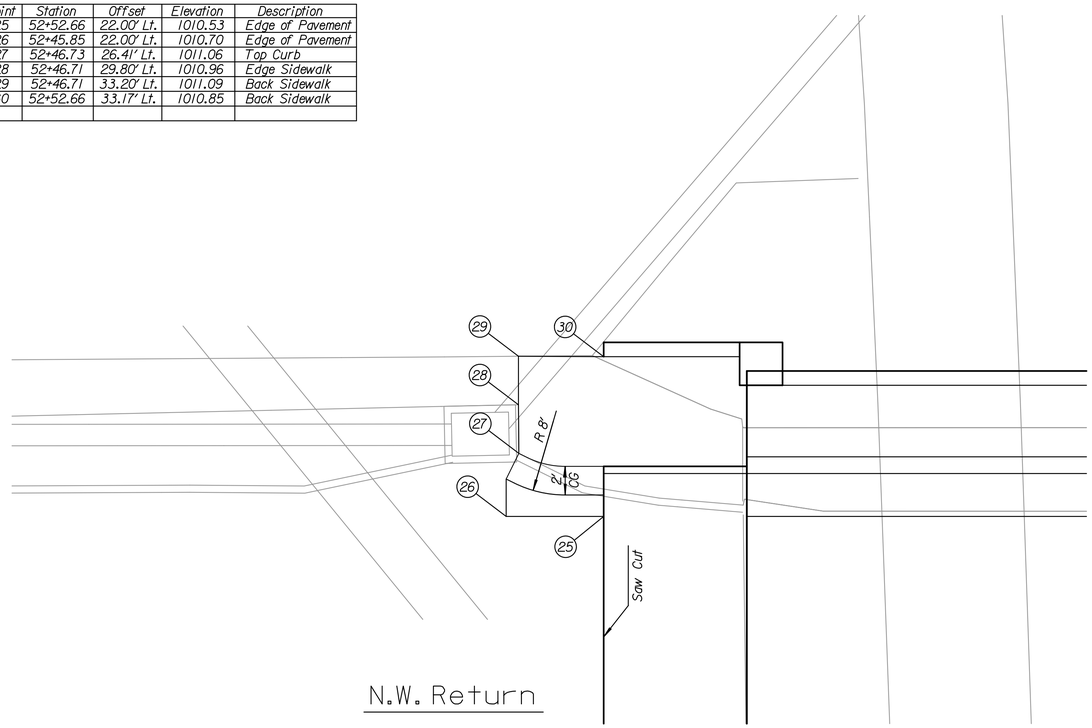
17 Cu. Yds. Common Excavation (VMF=0.80)
22 Cu. Yds. Common Excavation (Contractor Furnished)(VMF=0.80)
99 Cu. Yds. Rock Excavation (VMF=1.00)(To be Wasted)
31 Cu. Yds. Embankment



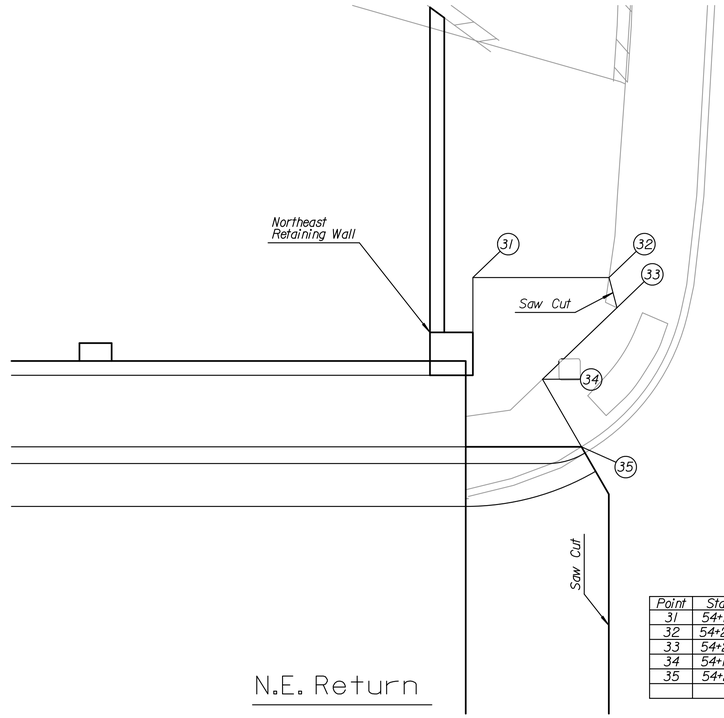
KANSAS DEPARTMENT OF TRANSPORTATION
PLAN AND PROFILE
STA. 46+00 TO STA. 61+00

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	4	99

Point	Station	Offset	Elevation	Description
25	52+52.66	22.00' Lt.	1010.53	Edge of Pavement
26	52+45.85	22.00' Lt.	1010.70	Edge of Pavement
27	52+46.73	26.41' Lt.	1011.06	Top Curb
28	52+46.71	29.80' Lt.	1010.96	Edge Sidewalk
29	52+46.71	33.20' Lt.	1011.09	Back Sidewalk
30	52+52.66	33.17' Lt.	1010.85	Back Sidewalk



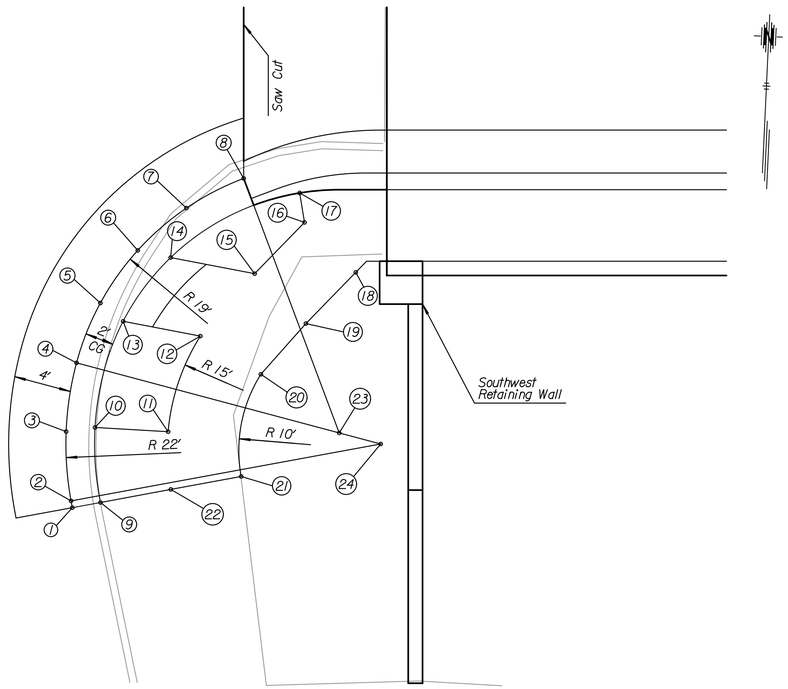
N.W. Return



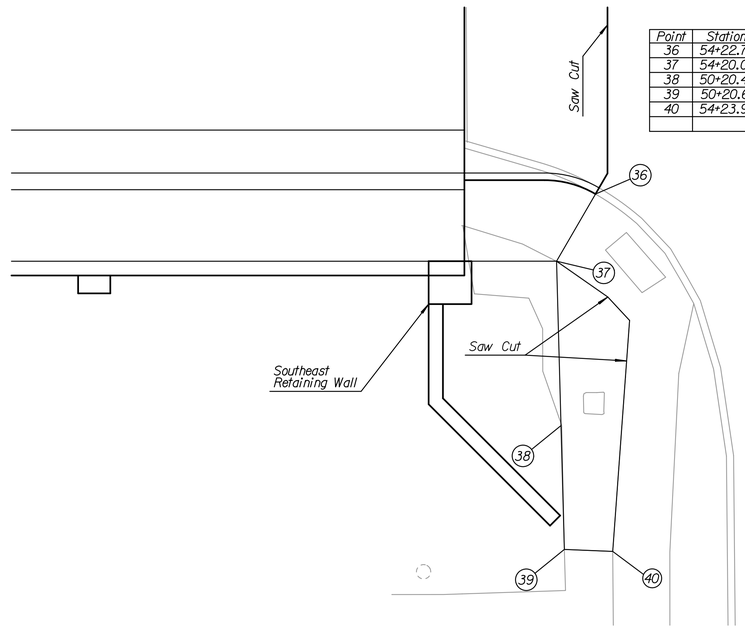
Point	Station	Offset	Elevation	Description
31	54+14.13	38.00' Lt.	1005.22	Back Conc. Slab
32	54+23.64	38.00' Lt.	1004.36	Back Sidewalk
33	54+24.19	35.89' Lt.	1004.51	Match Exist. Sidewalk
34	54+18.99	30.89' Lt.	1005.32	Back Sidewalk
35	54+21.71	26.17' Lt.	1005.10	Top Curb

N.E. Return

Point	Station	Offset	Elevation	Description
1	52+40.69	59.39' Rt.	1009.23	POT
2	52+40.60	58.92' Rt.	1009.26	PC
3	52+40.25	54.07' Rt.	1009.56	1/2 Pt
4	52+40.98	49.27' Rt.	1009.79	PCC
5	52+42.65	45.09' Rt.	1009.96	1/4 Pt
6	52+45.26	41.41 Rt.	1010.08	1/2 Pt
7	52+48.66	38.45' Rt.	1009.96	3/4 Pt
8	52+52.66	36.38 Rt.	1010.16	PT
9	52+42.65	59.03' Rt.	1009.70	Top Curb
10	52+42.27	53.79' Rt.	1010.06	Top Curb
11	52+47.38	54.08' Rt.	1009.70	Sidewalk Ramp
12	52+49.64	47.40' Rt.	1010.05	Sidewalk Ramp
13	52+44.24	46.36' Rt	1009.95	Top Curb
14	52+47.56	41.91' Rt.	1010.10	Top Curb
15	52+53.43	43.04' Rt.	1010.05	Sidewalk Ramp
16	52+56.91	39.45' Rt.	1010.08	Sidewalk Ramp
17	52+56.57	37.39' Rt.	1010.12	Top Curb
18	52+60.50	42.94' Rt.	1010.00	Back Sidewalk
19	52+57.01	46.52' Rt.	1010.05	Back Sidewalk
20	52+53.86	50.07' Rt.	1010.05	Back Sidewalk
21	52+52.49	57.21' Rt.	1009.70	Back Sidewalk
22	52+47.57	58.11' Rt.	1009.70	Edge Sidewalk
23	52+59.34	54.17' Rt.	-	Radius Pt.
24	52+62.24	54.94' Rt.	-	Radius Pt.



S.W. Return



Point	Station	Offset	Elevation	Description
36	54+22.79	34.47' Rt.	1004.88	Top Curb
37	54+20.07	42.16' Rt.	1004.97	Back Sidewalk
38	50+20.43	53.66' Rt.	1003.82	Back Conc. Slab
39	50+20.61	62.31' Rt.	1003.42	Back Conc. Slab
40	54+23.99	62.45' Rt.	1003.42	Match Exist. Sidewalk

SCALE
PLAN: Lat. & Long. 5' 0 5' 10'

S.E. Return

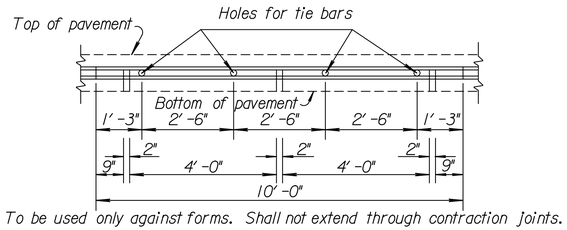
KANSAS DEPARTMENT OF TRANSPORTATION

Intersection Details

DATE | BY | REFERENCES NOTED | REFERENCES CHECKED

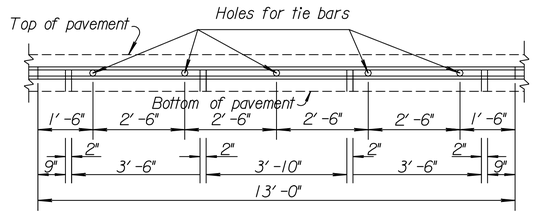
Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\MicroHighways\N057601rpd-01_plandetail.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	5	99



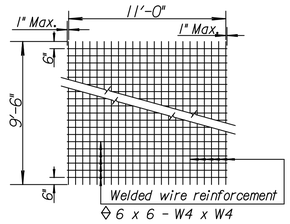
To be used only against forms. Shall not extend through contraction joints.

METAL STRIP FOR
LONGITUDINAL CONSTRUCTION JOINT (10'-0")



To be used only against forms. Shall not extend through contraction joints.

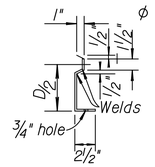
METAL STRIP FOR
LONGITUDINAL CONSTRUCTION JOINT (13'-0")



◊ 6 x 6 - W4 x W4

TYPICAL SHEET OF WELDED WIRE REINFORCEMENT
FOR SPECIAL BRIDGE APPROACH PAVEMENT

◊ Note: Epoxy coated #3 bars longitudinally @ 12" ctrs. & #3 bars transversely @ 18" ctrs. may be substituted for each layer of epoxy coated welded wire reinforcement.



ϕ Snap-in leg or other approved designs may be used in lieu of welded leg.

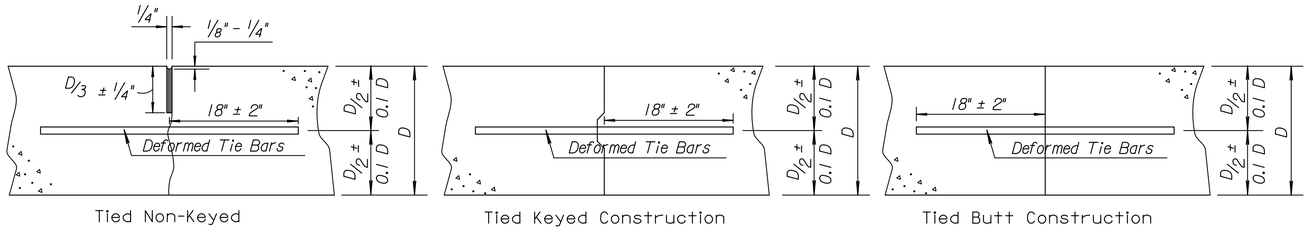
SECTION OF RECESSED
FORM LEG



DETAIL OF LAP FOR WELDED WIRE REINFORCEMENT

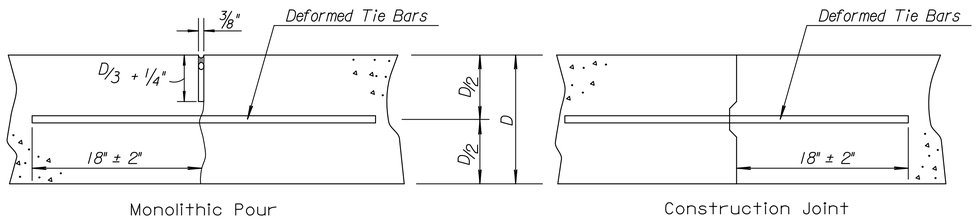
The lap shall extend beyond the first transverse or bag wire of each sheet.

The sheet shall be wired securely at the edges and at intervals not to exceed 2'-6" for the full width of the sheet. Approximate weight of welded wire reinforcement = 58 lbs. per 100 sq. ft. Other methods for fastening the sheets of welded wire reinforcement at the laps may be used with the approval of the Engineer.



LONGITUDINAL JOINTS

Note: For longitudinal construction joints the contractor has the option of using either the keyed or butt type.



TRANSVERSE JOINTS

Note: A construction joint is required when the concrete placement has been interrupted for a substantial length of time or at the end of a day's placement.

GENERAL NOTES

All work shall be done in conformity with the Standard Specifications applicable to the project.

The cost of all bars and joint material shown on this sheet is to be included in the bid price for Concrete Pavement.

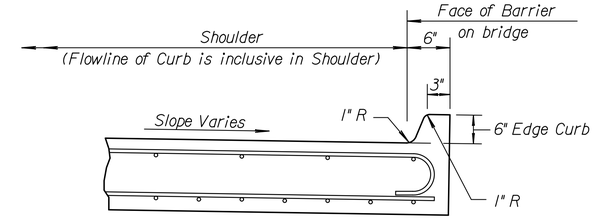
At each planned transverse joint location, a 4 to 6 inch wide strip of the pavement surface shall be protected from the texturing operation to provide a transverse textureless surface centered over the joint sawcut.

All sawed joints on this project shall be filled with sealant in accordance with Standard Specifications.

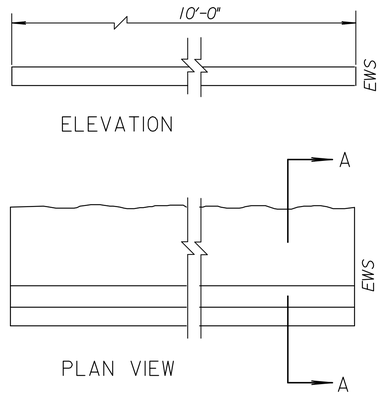
The 6 inch edge curb shall be constructed integral with the approach slab shoulder.

All materials and work required for this construction shall be Subsidiary to the concrete approach slab.

Tie bars shall be evenly spaced along the length of the slab and no tie bars shall be within 12" of contraction joint.



SECTION A-A



ELEVATION

PLAN VIEW

6" EDGE CURB DETAIL

NO.	DATE	REVISIONS	BY	APP'D
12	5-14-09	Pres. Relief Jt. to RD712/tie bar lab.	S.W.K.	J.O.B.
11	10-23-08	Revised Sec. A-A and Sec. B-B	S.W.K.	J.O.B.
10	10-3-07	Add. manufacturer jt. size recom'd.	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

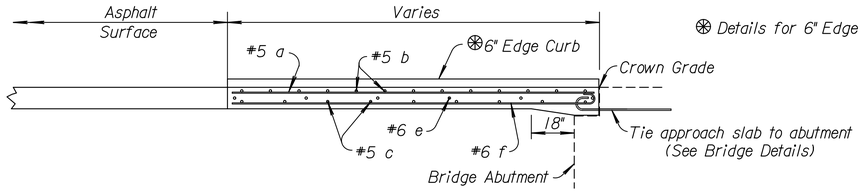
MISCELLANEOUS DETAILS FOR CONCRETE BRIDGE APPROACH PAVEMENT

~~RD711~~

FHWA APPROVAL	6-9-09	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. King

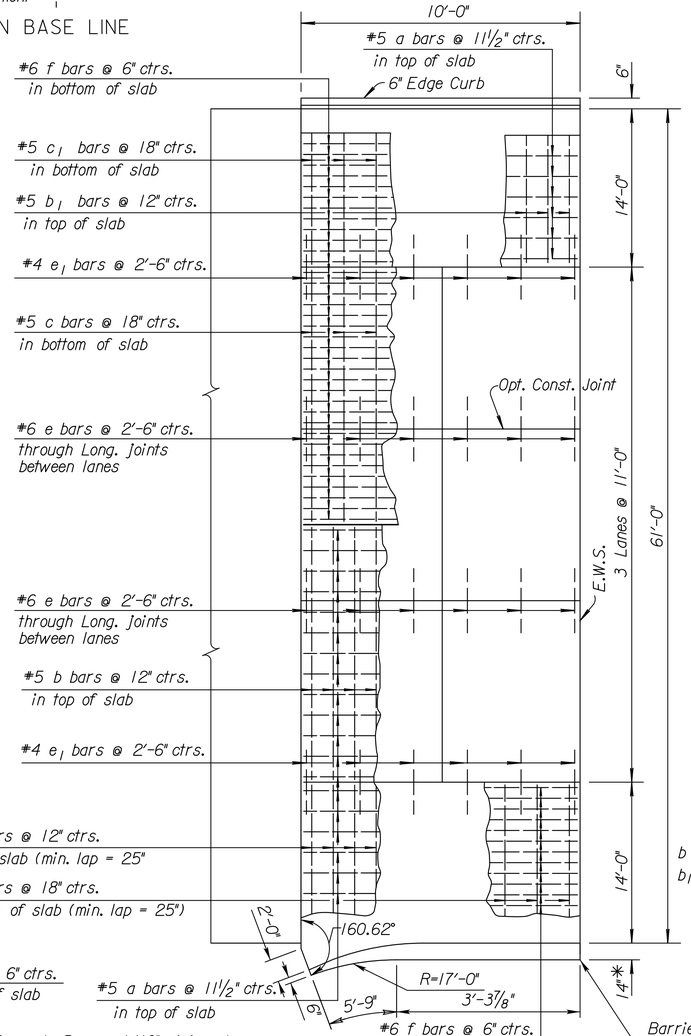
Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\MicroHighways\Client Standards\rd711.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	6	99



⊛ Details for 6" Edge Curb, See Standard Drawing RD711.

LONGITUDINAL SECTION ON BASE LINE



PLAN FOR NORMAL APPROACH
(No Scale)

PLAN FOR SKEWED APPROACH (SKEW _ 5)
(No Scale)

"D" Thickness = Thickness of Project Concrete Pavement (10" minimum).

BILL OF MATERIALS

BAR SCHEDULE

NORMAL APPROACH											
Bar	a	b	b_1	b_2	c	c_1	c_2	e	e_1	f	
No.	74	33	11	22	24	8	16	10	10	133	
Size	#5	#5	#5	#5	#5	#5	#5	#6	#4	#6	
Length	9'-8"	11'-10"	15'-4"	10'-1"	10'-8"	14'-2"	9'-6"	3'-0"	3'-0"	10'-4"	
Reinforcing Steel (Grade 60) (Epoxy Coated)											4,234 lbs.
Concrete Pavement (10" Unif.)(AE)											69.8 Sq. Yds.

__°SKEW											
a	b	b_1	b_2	c	c_1	c_2	e	e_1	f	
#5	#5	#5	#5	#5	#5	#5	#6	#4	#6	
							3'-0"	3'-0"		
Reinforcing Steel (Grade 60) (Epoxy Coated)										lbs.
Concrete Pavement (__" Unif.)(AE)										Sq. Yds.

__°SKEW											
a	b	b_1	b_2	c	c_1	c_2	e	e_1	f	
#5	#5	#5	#5	#5	#5	#5	#6	#4	#6	
							3'-0"	3'-0"		
Reinforcing Steel (Grade 60) (Epoxy Coated)										lbs.
Concrete Pavement (__" Unif.)(AE)										Sq. Yds.

GENERAL NOTE

Special Concrete Bridge Approach shall be paid for as Sq. Yds. of Concrete Pavement (10" Unif.)(AE) and includes all work and materials required to construct the approach slab as shown on this sheet, except the barrier transition. Barrier transition concrete and reinforcing steel is included in the bridge quantities.

All work and materials required for installation of joint material shall be subsidiary to this bid item.

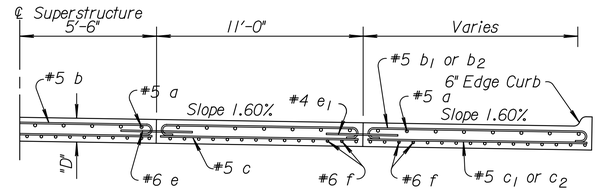
At the Contractor's option #4x3'-0" tie bars @ 15" centers may be substituted for the #6 e bars at 2'-6" centers.

All reinforcing steel shall be epoxy coated.

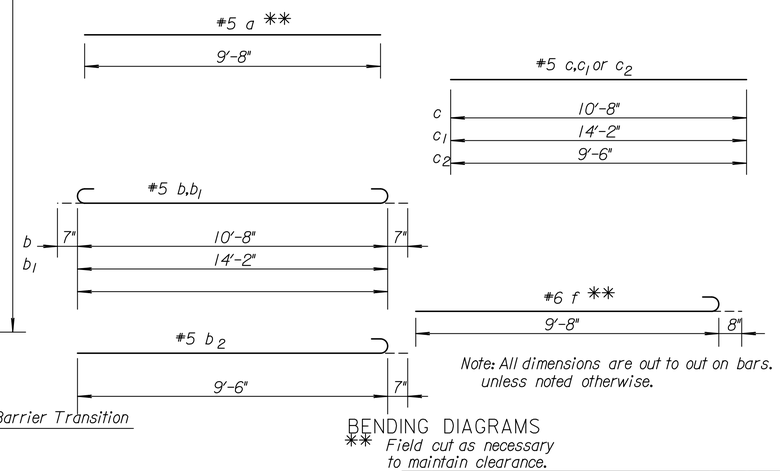
See Standard Drawing RD711 for details of joints and edge curb.

Clearance from the face of concrete for all reinforcing steel shall be 2 inches.

Standard reinforcing bar hooks in accordance with the latest ACI specifications shall be used throughout.



TYPICAL HALF SECTION
(No Scale)



Note: All dimensions are out to out on bars unless noted otherwise.

BENDING DIAGRAMS

** Field cut as necessary to maintain clearance.

* See barrier details for barrier on approach slab.

Note: Reinforcing steel and joint lengths shown for information only.

NO.	DATE	REVISIONS	BY	APP'D
9	9-09-09	Revised Reinforcing Steel listing	S.W.K.	J.O.B.
8	5-14-09	Revised General Note	S.W.K.	J.O.B.
7	10-30-08	Added guardrail post detail at curb	S.W.K.	J.O.B.
6	11-07-07	Revised pavement slope to percent	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

CONCRETE BRIDGE APPROACH PAVEMENT ADJACENT TO ASPHALT SURFACE ABUTMENT NO. 1

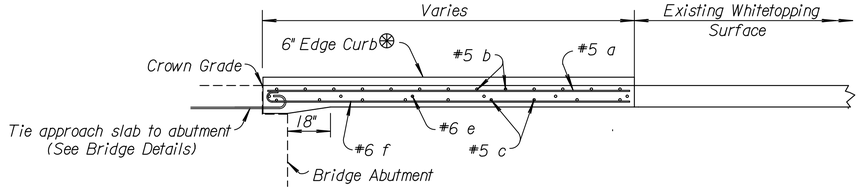
~~RD715~~

FHWA APPROVAL	6-9-09	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. King

Note to Designer: The designer shall be responsible for designating pavement thickness and computing reinforcing steel and concrete quantities and dimensions necessary to complete this sheet

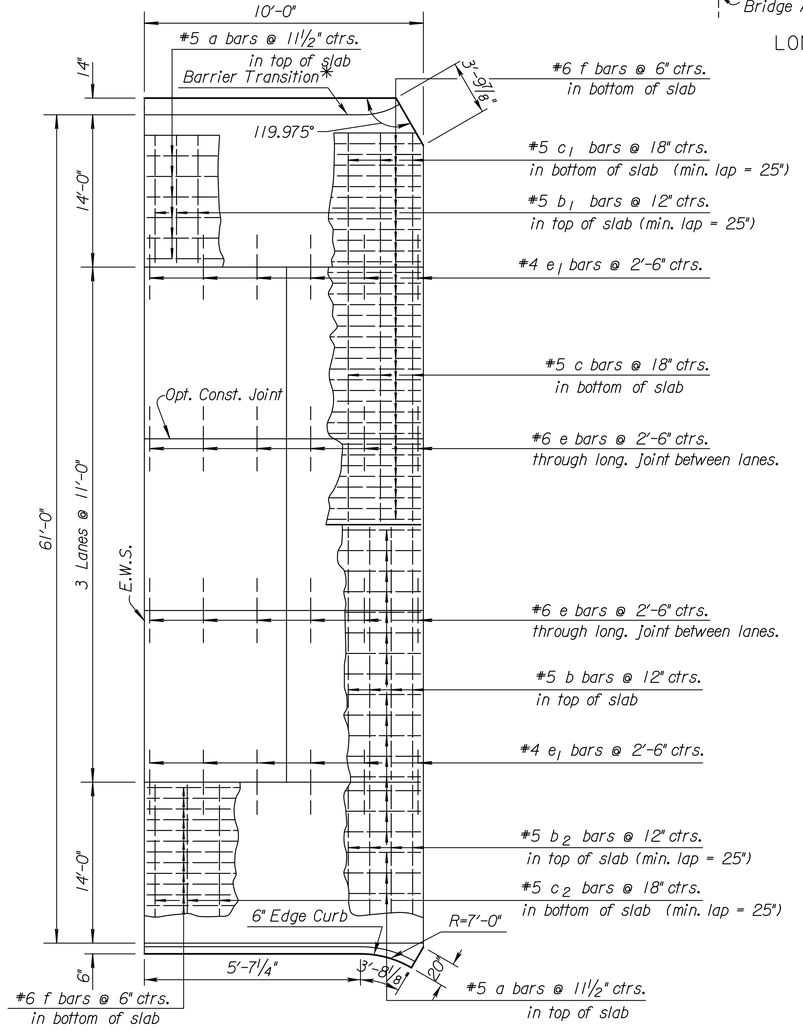
Drawn By : cluedke Plotted : 9/13/2013
G:\12664\Micro\Highways\Client Standards\rd715-abut1.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	7	99



⊛ Details for 6" Edge Curb, See Standard Drawing RD711.

LONGITUDINAL SECTION ON BASE LINE



PLAN FOR NORMAL APPROACH
(No Scale)

"D" Thickness = Thickness of Project Concrete Pavement (10" minimum).

GENERAL NOTE

Special Concrete Bridge Approach shall be paid for as Sq. Yds. of Concrete Pavement (10" Unif.)(AE) and includes all work and materials required to construct the approach slab as shown on this sheet, except the barrier transition. Barrier transition concrete and reinforcing steel is included in the bridge quantities.

All work and materials required for installation of joint material shall be subsidiary to this bid item.

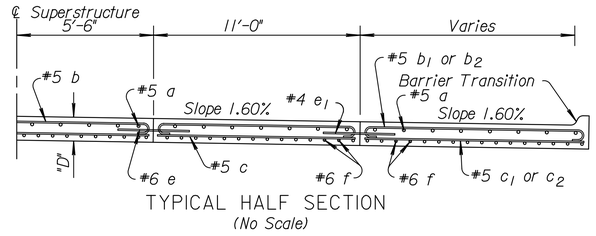
At the Contractor's option #4x3'-0" tie bars @ 15" centers may be substituted for the #6 e bars at 2'-6" centers.

All reinforcing steel shall be epoxy coated.

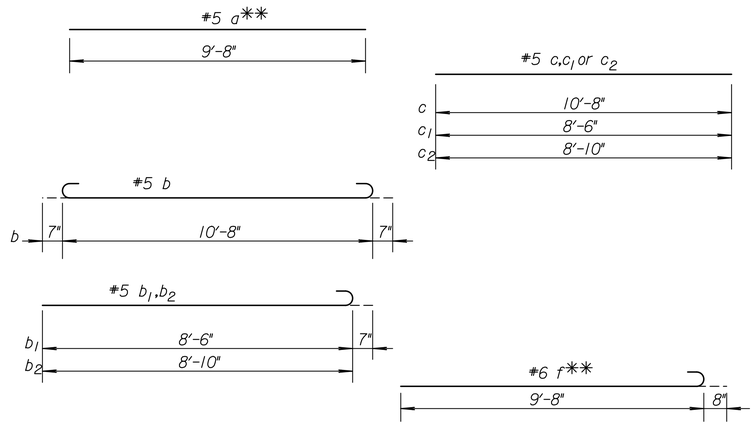
See Standard Drawing RD711 for details of joints and edge curb.

Clearance from the face of concrete for all reinforcing steel shall be 2 inches.

Standard reinforcing bar hooks in accordance with the latest ACI specifications shall be used throughout.



TYPICAL HALF SECTION
(No Scale)



Note: All dimensions are out to out on bars. unless noted otherwise.

BENDING DIAGRAMS

** Field cut as necessary to maintain clearance.

BILL OF MATERIALS

BAR SCHEDULE

NORMAL APPROACH

Bar	a	b	b_1	b_2	c	c_1	c_2	e	e_1	f
No.	74	33	22	22	16	16	10	10	133	
Size	#5	#5	#5	#5	#5	#5	#5	#6	#4	#6
Length	9'-8"	11'-10"	9'-1"	9'-5"	10'-8"	8'-6"	8'-10"	3'-0"	3'-0"	9'-8"

Reinforcing Steel (Grade 60) (Epoxy Coated) 4,130 lbs.

Concrete Pavement (10" Unif.)(AE) 69.6 Sq. Yds.

__°SKEW

a	b	b_1	b_2	c	c_1	c_2	e	e_1	f
#5	#5	#5	#5	#5	#5	#5	#6	#4	#6
							3'-0"	3'-0"	

Reinforcing Steel (Grade 60) (Epoxy Coated) lbs.

Concrete Pavement (__" Unif.)(AE) Sq. Yds.

__°SKEW

a	b	b_1	b_2	c	c_1	c_2	e	e_1	f
#5	#5	#5	#5	#5	#5	#5	#6	#4	#6
							3'-0"	3'-0"	

Reinforcing Steel (Grade 60) (Epoxy Coated) lbs.

Concrete Pavement (__" Unif.)(AE) Sq. Yds.

* See barrier details for barrier on approach slab.

Note: Reinforcing steel and joint lengths shown for information only.

NO.	DATE	REVISIONS	BY	APP'D
9	9-09-09	Revised Reinforcing Steel listing	S.W.K.	J.O.B.
8	5-14-09	Revised General Note	S.W.K.	J.O.B.
7	10-30-08	Added guardrail post detail at curb	S.W.K.	J.O.B.
6	11-07-07	Revised pavement slope to percent	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

CONCRETE BRIDGE APPROACH PAVEMENT ADJACENT TO CONCRETE SURFACE ABUTMENT NO. 2

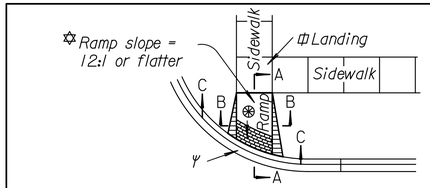
~~RD715~~

FHWA APPROVAL		APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. King

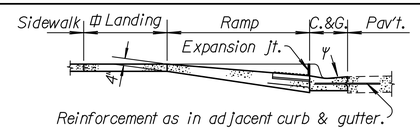
Note to Designer: The designer shall be responsible for designating pavement thickness and computing reinforcing steel and concrete quantities and dimensions necessary to complete this sheet

Drawn By : cluedke Plotted : 9/13/2013
G:\12664\Micro\Highways\Client Standards\rd715-abut2.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	8	99

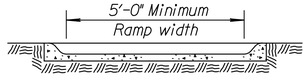


PLAN VIEW

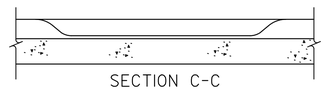


Reinforcement as in adjacent curb & gutter.

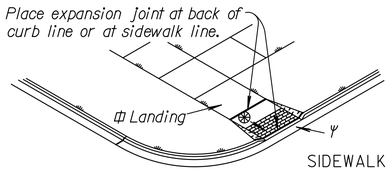
SECTION A-A



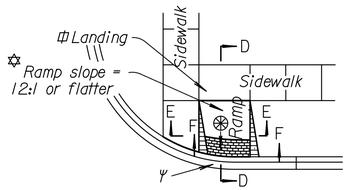
SECTION B-B



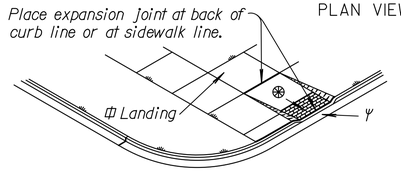
SECTION C-C



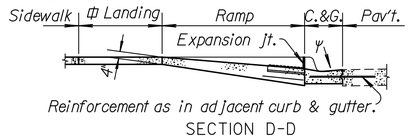
SIDEWALK RAMP TYPE 1



PLAN VIEW

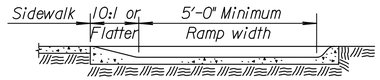


SIDEWALK RAMP TYPE 2

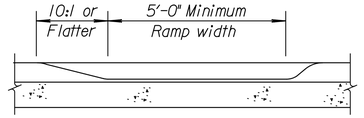


Reinforcement as in adjacent curb & gutter.

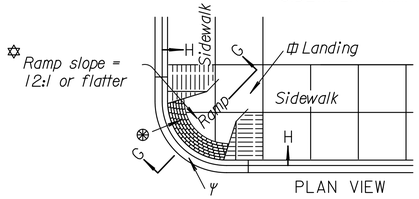
SECTION D-D



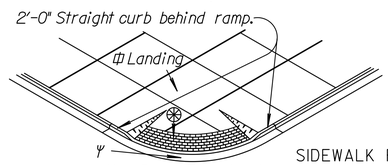
SECTION E-E



SECTION F-F

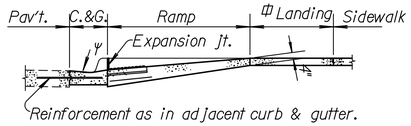


PLAN VIEW



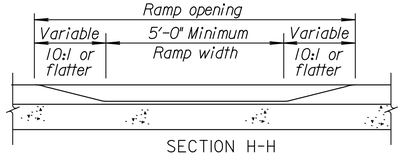
SIDEWALK RAMP TYPE 3

Use Ramp Type 3 where sidewalk occupies entire area between curb and property line.



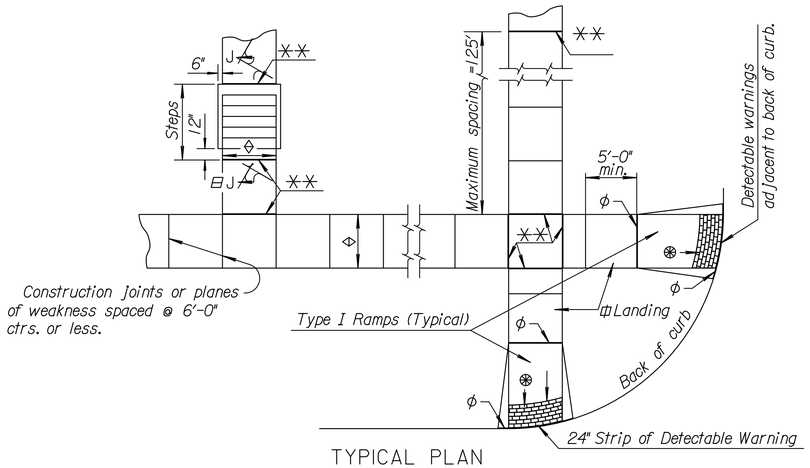
Reinforcement as in adjacent curb & gutter.

SECTION G-G

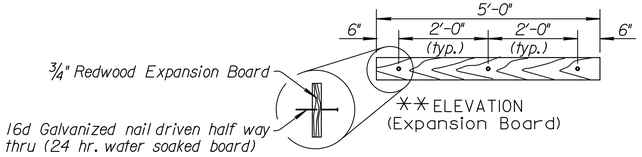


SECTION H-H

⊟ SECTION J-J see standard drawing RD725B for details of Steps and Handrail.



TYPICAL PLAN



For other than 5'-0" (width) use 6" from the end and balanced (3'-0" max.) nail spacing.

ϕ Expansion Joint (3/4" Redwood board) placed at either back of curb line or at sidewalk back of curb line or at sidewalk line.

XX Expansion Joint (3/4" redwood board) located as shown.

◊ Variable width (5'-0" min.). Entrance walk to be same width as approach walk. Use sidewalk width of 4'-0" where existing conditions prohibit use of 5'-0".

✡ New construction ramp slopes are 12:1 or flatter. Desirable ramp slopes for existing sites are 12:1 or flatter. Where space limitations prohibit construction of a 12:1 slope on existing sites, use the following slopes:
8:1 or flatter for a maximum rise of 3 inches
10:1 or flatter for a maximum rise of 6 inches

⏢ Use a landing slope of 50:1 or flatter. Landings are the same width as ramp and a minimum 5'-0" in length.

ψ Use a counter slope of 20:1 or flatter at the base of sidewalk ramps. Refer to standard drawing RD725A for additional curb and gutter details.

⊛ Detectable warning installation is typical and required on Sidewalk Ramps Type 1, Type 2, Type 3, median ramp crossings and other locations as shown in the plans. See standard drawing RD725A for additional details.

GENERAL NOTES

The details depicted here may not be appropriate for all locations. Designs shall meet this criteria on all new construction projects unless impracticable by site restrictions. For an existing sidewalk facility where the sidewalk will be replaced, sidewalk will be replaced according to this drawing to the maximum extent feasible.

Provide ramps at all corners street intersections where there is an existing or proposed sidewalk and curb. Provide ramps at mid-block walk locations for hospitals, medical centers and athletic stadiums.

Details shown on this sheet apply to all constrcution or reconstruction of streets, curbs or sidewalks. See standard specifications for additional information.

Restrict use of sidewalk ramp Type 2 to locations where it is not feasible to use Types 1 or 3.

Curb cut ramps are to be located as shown on the plans or as directed by the Engineer.

Do not place drainage structures in line with ramps except where existing drainage structures are being utilized in the new construction. Ramp locations should take precedence over the location of drainage structures.

Ramp sidewalks where the driveway curb is extended across the walk.

Construct ramp with uniform grade free of sags and short grade changes.

Place 3/4" redwood expansion joints flush with the surface at a maximum spacing of 125'. Place 3/4" redwood expansion joints at sidewalk junctions, see plan details. Where sidewalk abuts a curb place 3/4" redwood board expansion joint flush with the surface.

Place 1/2" premolded (Non-extruding, Type B) joint filler where sidewalk is parallel and adjacent to a rigid surface.

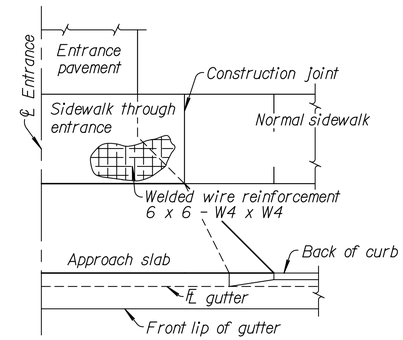
Slope sidewalk toward the street at a 50:1 or flatter. Slope or depress sidewalk where necessary to fit alleys and entrances see plans or at Engineer's direction.

Where clear width of sidewalk between top of ramp and building or other obstruction is less than 48 inches, slope the flared sides 12:1 or flatter.

Sidewalk shown to be constructed in back of an entrance shall be placed 6" thick with welded wire mesh reinforcement. Gauge and spacing of wires are the same as entrance pavement (See Reinforcement Diagram). Bid item will be "Sidewalk Construction (6") either with or without air entrainment.

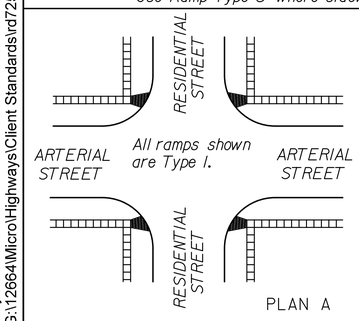
~~Contractor may opt to use Concrete Grade 3.0 (AE) throughout for construction of steps, but all work and materials are paid for under the bid item "Grade 3.0 Conc. (Misc.)".~~

Surface texture ramp slope with a wooden float or brushed finish.



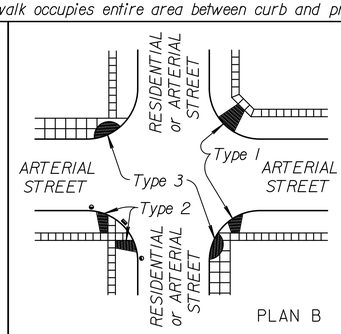
REINFORCEMENT DIAGRAM
SIDEWALK THROUGH ENTRANCE

See standard drawing RD726 for details of Alley and Entrance Pavement.



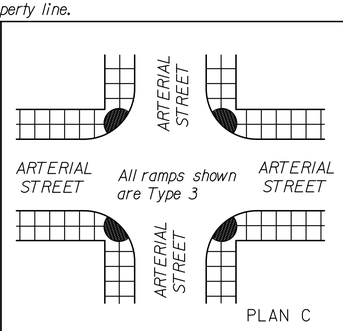
PLAN A

Heavy Traffic on Arterial
No Traffic Signal



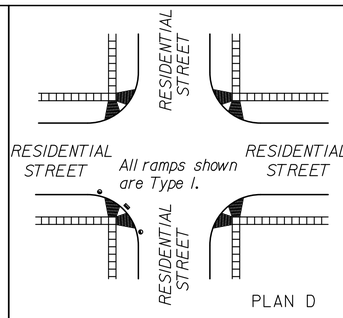
PLAN B

Signalized Intersection
Typical Ramp Treatments



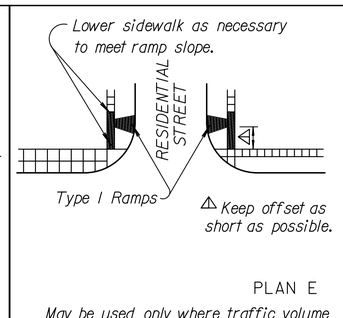
PLAN C

Traffic Control on Both Streets
Wide Sidewalks



PLAN D

Normal Treatment in Residential Area



PLAN E

May be used only where traffic volume is low and where other features make Plan D impractical.

LEGEND

- Sidewalk Ramp Type 1
- Sidewalk Ramp Type 2
- Sidewalk Ramp Type 3
- Preferred location of drainage inlet (Typical)
- Alternate location of drainage inlet (Typical)
- Sidewalk

NO.	DATE	REVISIONS	BY	APP'D
11	10-17-11	Revised General Note	S.W.K.	J.O.B.
10	5-23-11	Revised notes	S.W.K.	J.O.B.
9	2-10-10	Revised notes & General Note	S.W.K.	J.O.B.
8	7-30-08	Reorg. layout, added steps sheet	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

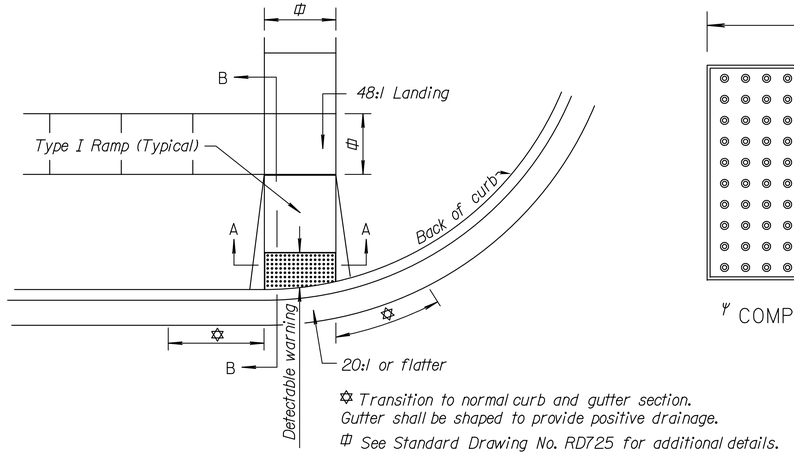
SIDEWALK & STEPS

RD725

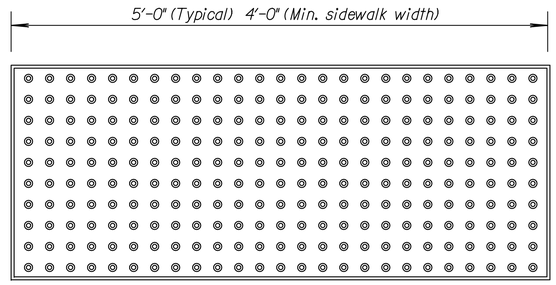
FHWA APPROVAL	12-5-11	APP'D.	James O. Brewer
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. King

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\Client Standards\rd725.dgn

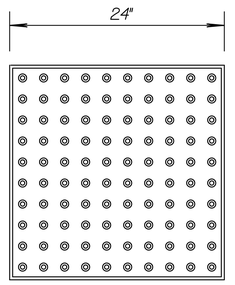
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	9	99



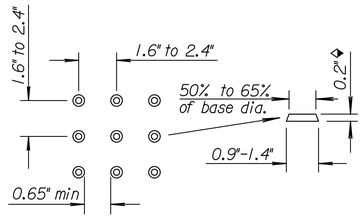
TYPICAL PLAN



COMPOSITE PANEL with TRUNCATED DOMES



PRESTRESSED RAMP PANEL with TRUNCATED DOME SURFACE



TRUNCATED DOME DIMENSIONS for SQUARE PATTERN (Parallel Alignment)

GENERAL NOTES

Details shown on Standard Drawing RD725 & RD725A may not be appropriate for all locations. Design new project construction to criteria shown on these drawings unless impracticable by site restrictions. For existing sidewalk replacement, follow these drawings to the maximum extent feasible.

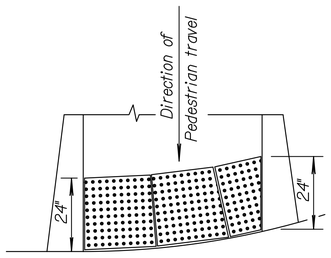
Use ~~Paving Brick, Prestressed or~~ Composite Panel (Truncated Dome Surface) units that meet the requirements of the Standard Specifications and comply with the American Disabilities Act. The panels shall be brick red in color.

Cover the ramp width and 24" length with truncated dome surface, see examples Standard Drawing RD725 & RD725A. Saw cut (only) bricks or panels with not less than 25% of a full brick or panel installed. Place Truncated Dome Bricks or Panels to align parallel in the direction of pedestrian travel.

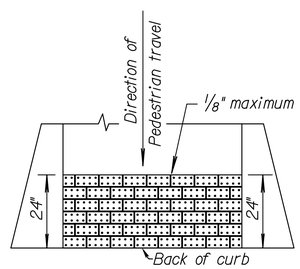
Prestressed or Composite Panels are installed in fresh concrete, Paving Brick require mortar bed and mortar sand, see Standard Specifications for requirements.

The entire ramp or median crossing will be bid as "Sidewalk Ramp" and payment is measured by the square yard. Materials and labor to install sidewalk ramp are subsidiary to the bid item "Sidewalk Ramp."

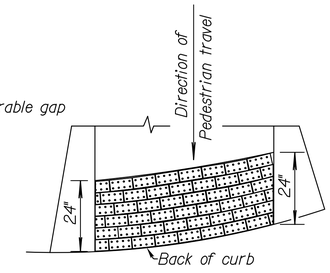
Composite Panel (Truncated Dome Surface) units shall be "Access Tile-Cast In Place Replaceable" or approved equal and shall be installed per manufacturer's recommendation.



RADIUS CURB DETAILS
Cut Prestressed Ramp Panels to fit.



STRAIGHT CURB DETAILS

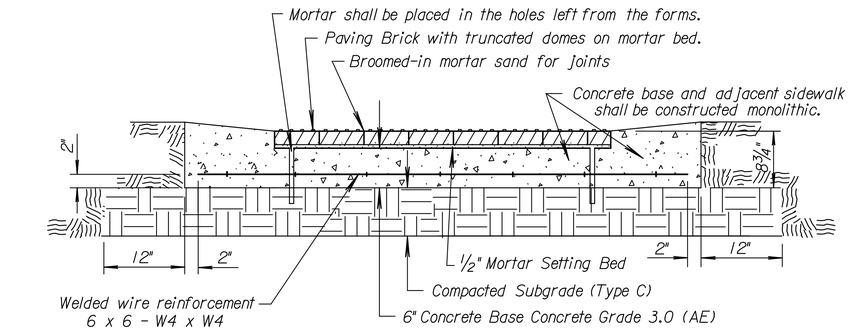


RADIUS CURB DETAILS

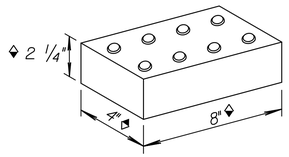
Installation shown for paving brick is running bond, use of other patterns is allowed with Engineer approval. Rotation 90° of Running Bond pattern is allowed to reduce space between bricks on radius installation, keep this space to a minimum. Place truncated domes on bricks in parallel alignment to pedestrian travel as shown.

Radius curb or greater than 5' width ramp will require more than one panel as approved by the Engineer. Limit number of panels installed at each site. Layouts shown are for example and may vary in the field.

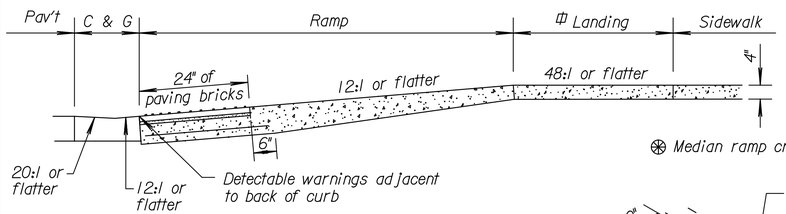
Acceptable with no mixing of types within an installation.



TYPICAL SECTION OF PAVER BRICK
SECTION A-A

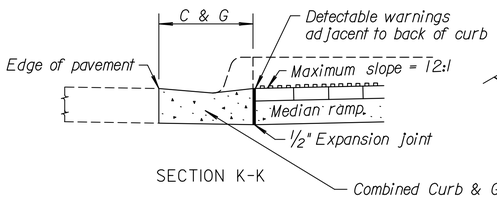


PAVER BRICK WITH TRUNCATED DOME SURFACE
These dimensions are nominal.

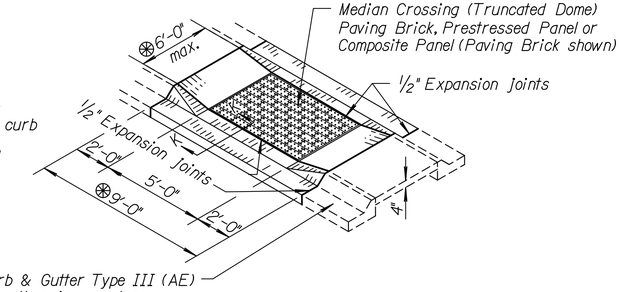


SECTION B-B

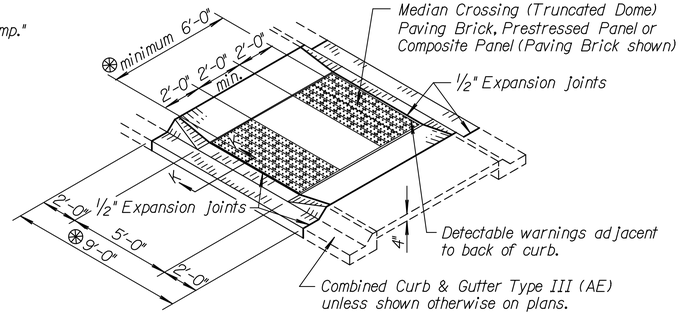
Median ramp crossing limits will be paid for as "Sidewalk Ramp."



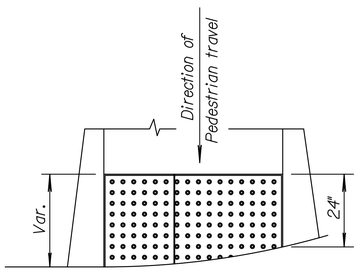
SECTION K-K



NARROW MEDIAN RAMP CROSSING
Note: A Median Ramp Crossing shall be constructed at Crosswalk locations.



WIDE MEDIAN RAMP CROSSING
Note: A Median Ramp Crossing shall be constructed at Crosswalk locations. Wider median islands will result in a 2' minimum gap between the truncated dome areas.



RADIUS CURB DETAILS (COMPOSITE)
Cut Radius from 3'-0" Long Composite Panel.

NO.	DATE	REVISIONS	BY	APP'D
4	2-10-10	Added Composite Panel	S.W.K.	J.O.B.
3	8-15-05	Added Prestressed Ramp Panel alt.	S.W.K.	J.O.B.
2	2-24-05	Class to Grade conc., wire reinf.	S.W.K.	J.O.B.
1	8-20-03	Revised details and notes	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

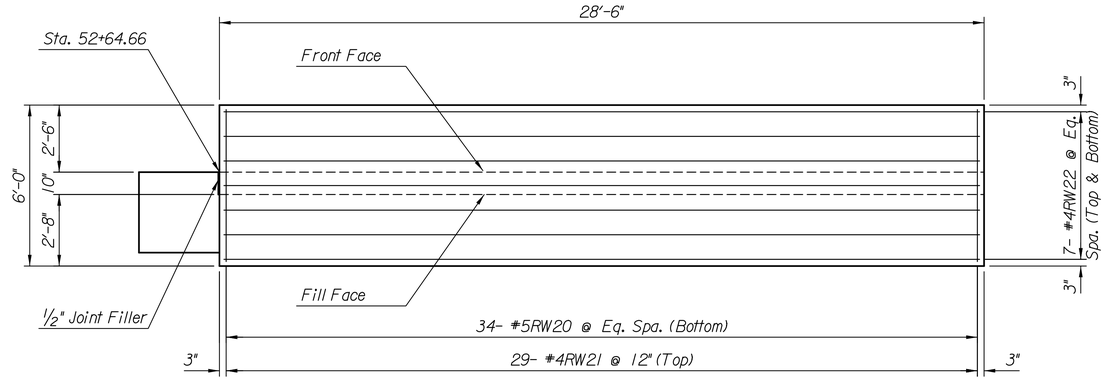
AUXILIARY DETAILS FOR SIDEWALK & STEPS

~~RD725A~~

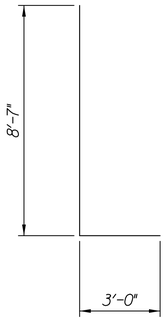
FHWA APPROVAL	4-7-10	APP'D. JAMES O. BREWER	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. King

Drawn By : cluedke
File : G:\12664\MicroHighways\Client Standards\rd725A.dgn
Plotted : 9/13/2013

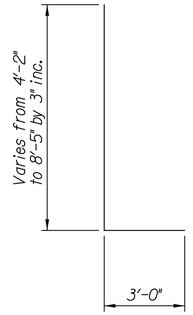
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	10	99



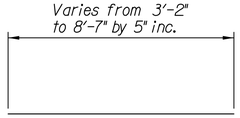
FOOTING PLAN



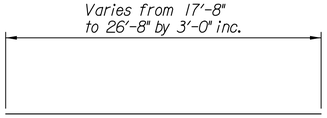
RW11



RW13
(Cut 1 Each Length)



RW12
(Cut 1 Each Length)



RW15
(Cut 1 Each Length)

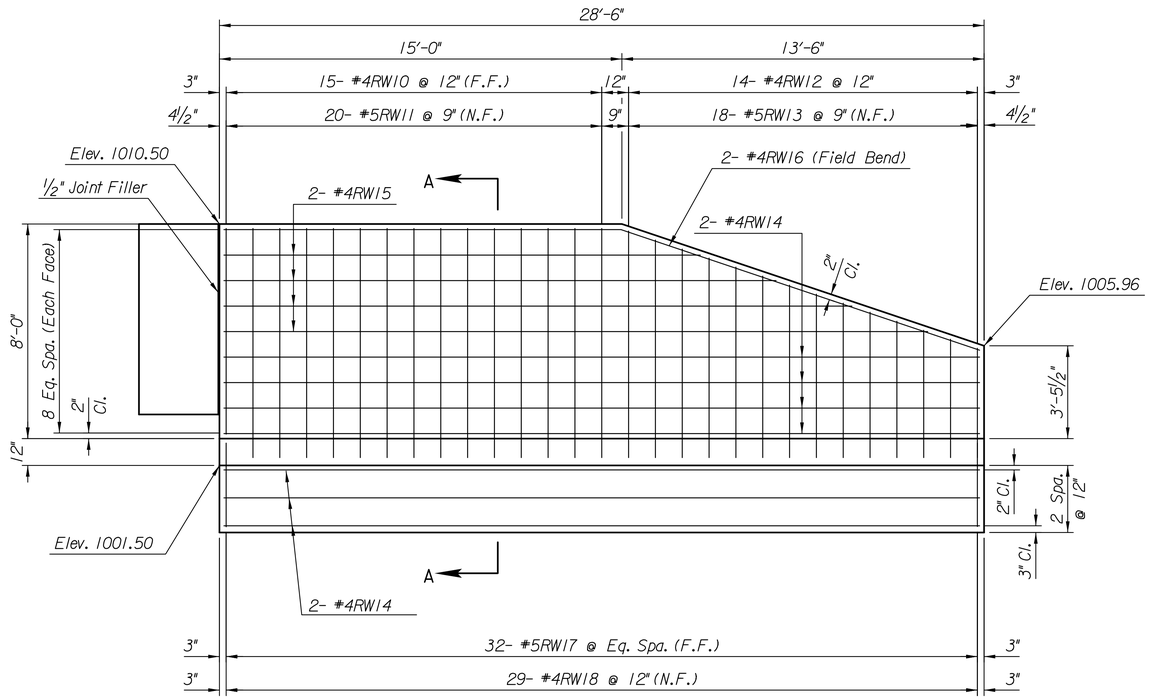
BILL OF REINFORCING STEEL
Grade 60

	Straight Bars				Bent Bars			
S.W. Retaining Wall	Mark	Size	Number	Length	Mark	Size	Number	Length
	RW17	#5	32	3'-3"	RW11	#5	20	11'-7"
	RW20	#5	34	5'-6"	RW13	#5	18	9'-3 1/2"
	RW10	#4	15	8'-8"				
	RW12	#4	14	5'-10 1/2"				
	RW14	#4	14	28'-2"				
	RW15	#4	8	22'-2"				
	RW16	#4	2	28'-11"				
	RW18	#4	29	3'-1"				
	RW21	#4	29	5'-6"				
	RW22	#4	14	28'-0"				

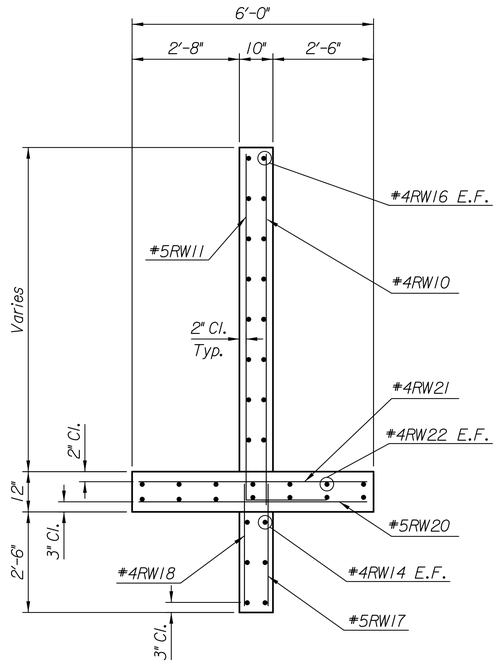
SUMMARY OF QUANTITIES *

Concrete (Grade 4.0) (AE) Cu. Yds.	Reinforcing Steel (Grade 60) Lbs.
14.6	1,710

* These quantities are included in the Road Quantities.



ELEVATION
(Looking at Fill Face)



SECTION A-A

Note:
Excavation, backfill and any required testing of in-situ materials is subsidiary to the bid item "Concrete (Grade 4.0) (AE)".

Wall design is based on an allowable bearing pressure of 1.5 ksf. Contractor shall verify that the in-situ soil has an allowable bearing pressure greater than 1.5 ksf.

N.F. = Near Face
F.F. = Far Face
E.F. = Each Face

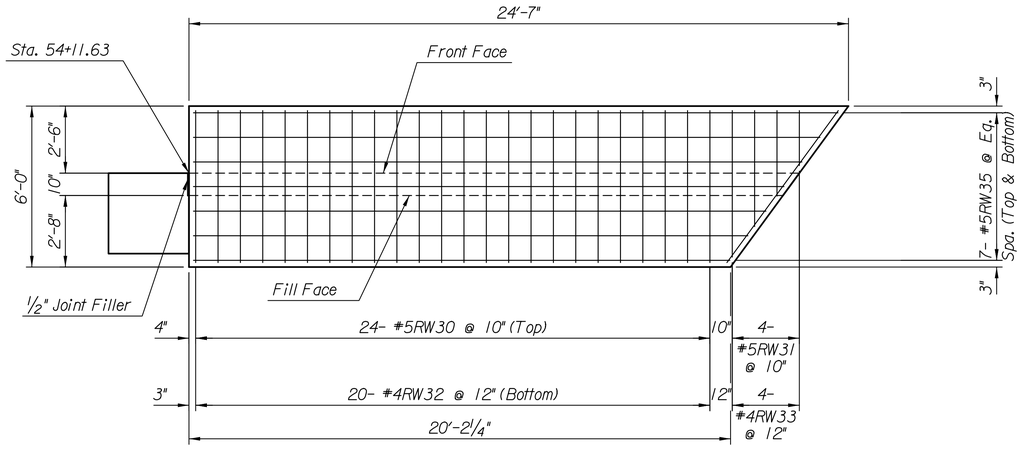
KANSAS DEPARTMENT OF TRANSPORTATION

SOUTHWEST RETAINING WALL
STA. 52+64.66 RT.

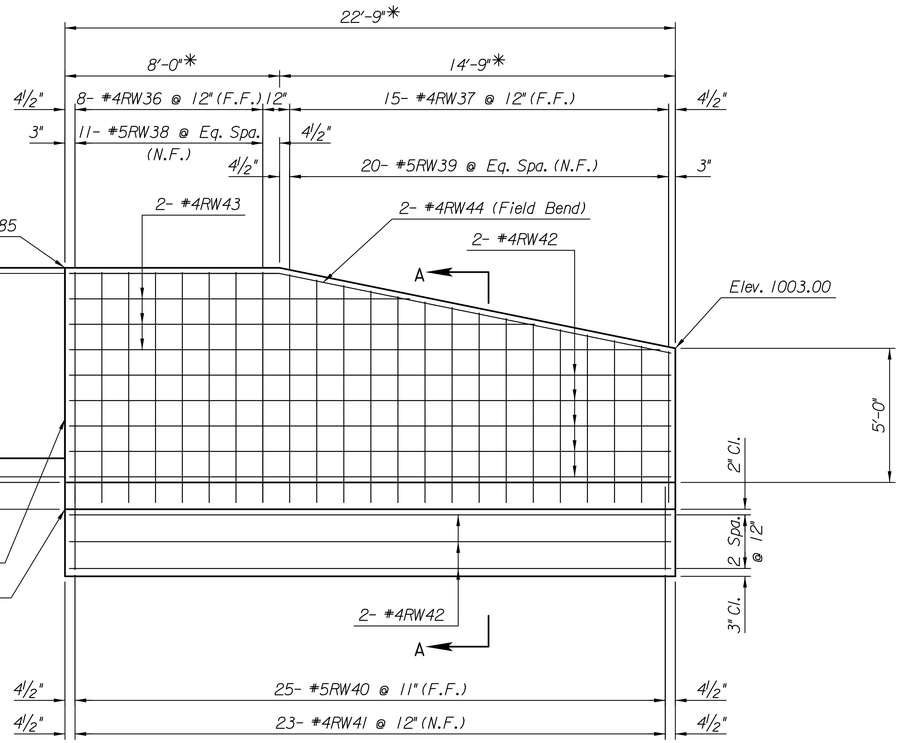
DATE
BY
REFERENCES NOTED
REFERENCES CHECKED

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\MicroHighways\N057601_retaining_wall.dgn

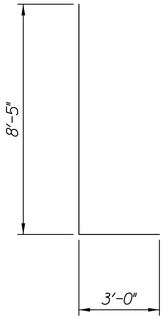
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	11	99



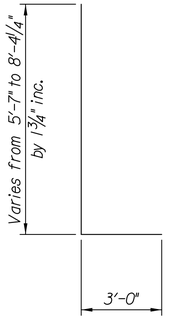
FOOTING PLAN



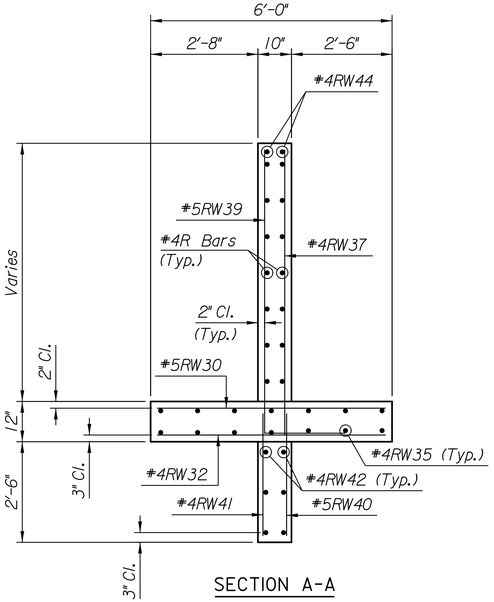
ELEVATION
(Looking at Fill Face)
* Dimension measured at Front Face



RW38



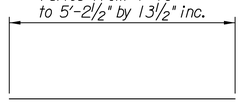
RW39
(Cut 1 Each Length)



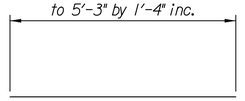
SECTION A-A

BILL OF REINFORCING STEEL
Non-Epoxy Coated - Grade 60

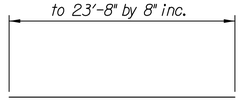
NE Retaining Wall	Straight Bars				Bent Bars			
	Mark	Size	Number	Length	Mark	Size	Number	Length
	RW30	#5	24	5'-6"	RW38	#5	11	11'-5"
	RW31	#5	4	3'-6 1/4"	RW39	#5	20	9'-11 5/8"
	RW40	#5	25	3'-3"				
	RW32	#4	20	5'-6"				
	RW33	#4	4	3'-3"				
	RW35	#4	14	21'-8"				
	RW36	#4	8	8'-5"				
	RW37	#4	15	7'-0 1/2"				
	RW41	#4	23	3'-1"				
	RW42	#4	16	22'-0"				
	RW43	#4	6	16'-11"				
	RW44	#4	2	22'-3"				



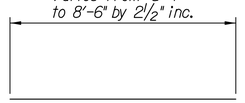
RW31
(Cut 1 Each Length)



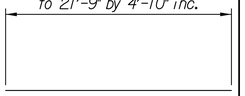
RW33
(Cut 1 Each Length)



RW35
(Cut 2 Each Length)



RW37
(Cut 1 Each Length)



RW43
(Cut 1 Each Length)

Note:
Excavation, backfill and any required testing of insitu material is subsidiary to the bid item "Concrete (Grade 4.0) (AE)".

Wall design is based on an allowable bearing pressure of 1.5 ksf. Contractor shall verify that the in-situ soil has an allowable bearing pressure greater than 1.5 ksf.

N.F. = Near Face
F.F. = Far Face

SUMMARY OF QUANTITIES *	
Concrete (Grade 4.0) (AE) Cu. Yds.	Reinforcing Steel (Grade 60) Lbs.
11.6	1,350

* These quantities are included in the Road Quantities.

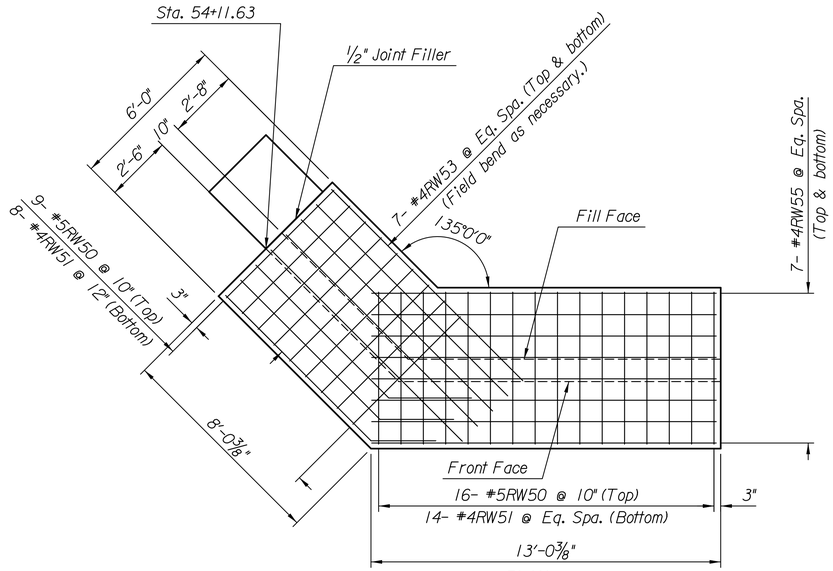
KANSAS DEPARTMENT OF TRANSPORTATION

NORTHEAST RETAINING WALL
STA. 54+11.63 LT.

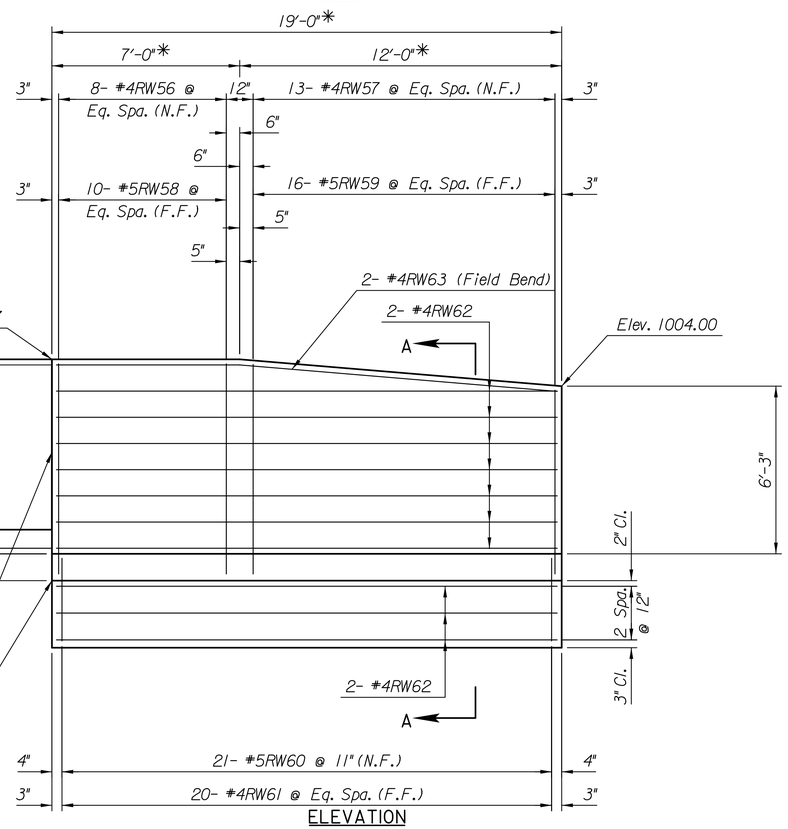
DATE
BY
REFERENCES NOTED
REFERENCES CHECKED

Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_retaining_wall_2.dgn
Plotted : 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	12	99



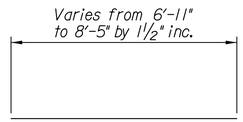
PLAN



ELEVATION

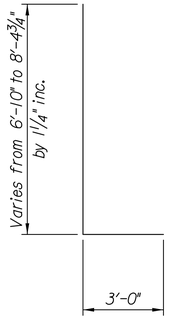
(Looking at Front Face)

* Dimension measured at Front Face



RW57

(Cut 1 Each Length)

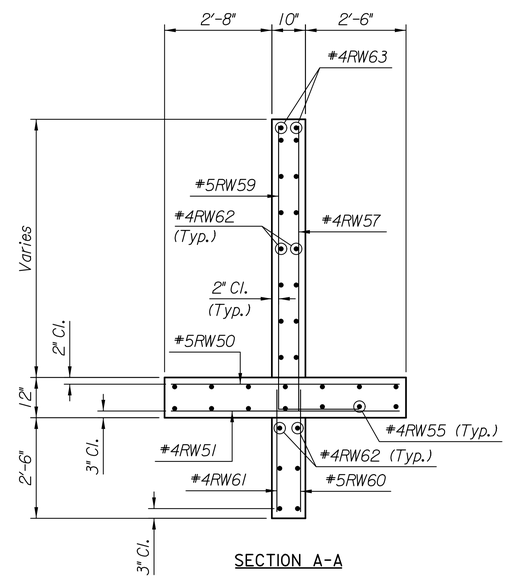


RW59

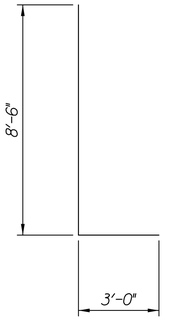
(Cut 1 Each Length)

BILL OF REINFORCING STEEL
Non-Epoxy Coated - Grade 60

	Straight Bars				Bent Bars			
	Mark	Size	Number	Length	Mark	Size	Number	Length
S.E. Retaing Wall	RW50	#5	25	5'-6"	RW58	#5	10	11'-6"
	RW60	#5	21	3'-3"	RW59	#5	16	10'-7 7/8"
	RW51	#4	22	5'-6"				
	RW53	#4	14	9'-4"				
	RW55	#4	14	12'-8"				
	RW56	#4	8	8'-7"				
	RW57	#4	13	7'-8"				
	RW61	#4	20	3'-1"				
	RW62	#4	20	17'-10"				
	RW63	#4	2	18'-2"				



SECTION A-A



RW58

Note:
Excavation, backfill and any required testing of insitu material is subsidiary to the bid item "Concrete (Grade 4.0) (AE)".

Wall design is based on an allowable bearing pressure of 1.5 ksf. Contractor shall verify that the in-situ soil has an allowable bearing pressure greater than 1.5 ksf.

N.F. = Near Face
F.F. = Far Face

SUMMARY OF QUANTITIES *	
Concrete (Grade 4.0) (AE) Cu. Yds.	Reinforcing Steel (Grade 60) Lbs.
9.7	1,210

* These quantities are included in the Road Quantities.

KANSAS DEPARTMENT OF TRANSPORTATION

SOUTHEAST RETAINING WALL
STA. 54+11.63 RT.

DATE	BY	
		REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_retaining_wall_3.dgn
Plotted : 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	13	99

SUMMARY OF QUANTITIES

Item / Location	Class III Excavation	Ø Concrete (Grade 4.0) (AE) (SA)	Reinforcing Steel (Gr. 60) (Epoxy Coated)	Structural Steel A709 (Gr. 50W)	Structural Steel M270 (Gr. 50WT3)	Electric Conduit (4.0") (Non-Metallic) (Installation Only) (Non-Participating)	Welded Stud Shear Connectors	Bearing (Steel Reinforced Elastomeric)	Abutment Aggregate Drain	Bridge Backwall Prot. System	Rip Rap (Reinforced Concrete) (4")	Drilling & Grouting	Falsework Inspection	Multi-Layer Polymer Concrete Overlay	Temporary Shoring	Fence (Pedestrian)	Bridge Handrail (Metal) (1'-10")	Bridge Drainage System
	Cu. Yds.	Cu. Yds.	Lbs.	Lbs.	Lbs.	Lin. Ft.	Each	Each	Cu. Yds.	Sq. Yds.	Sq. Yds.	Each	Lump Sum	Sq. Yds.	Lump Sum	Lin. Ft.	Lin. Ft.	Lump Sum
Abutment No. 1	82								31	45		186						
Pier No. 1								12										
Pier No. 2								12										
Abutment No. 2	82								31	45		186						
Aesthetic Columns		**	**															
Substr. Total	164								62	90		372						
Superstr. Total		389.6	93,420	32,984	158,443	1377	5208	24						1048		341	290	
Total	164	389.6	93,420	32,984	158,443	1377	5208	24	62	90	50	372	L.S.	1048	L.S.	341	290	L.S.

** Quantites are included in the Superstructure total quantity.
Ø Concrete shall be KCMMB 4K

INDEX TO BRIDGE DRAWINGS

13	General Notes
14	General Notes
15	Contour Map
16	Construction Layout
17	Phased Construction
18	Demolition Details
19	Abutment Details
20	Wingwall Details
21	Abutment Aggregate Drain
22	Bearing Devices
23	Framing Plan
24	Girder Details
25	Girder Details
26	Steel Erection, Fit Up & Bolting Procedure
27	Slab Section
28	Slab Plan
29	Bridge Drain Details
30	Barrier Details
31	North Barrier Elevation
32	South Barrier Elevation
33	Railing & Aesthetic Column Details
34	Column & Fence Details
35	Handrail & Barrier Details
36	Fence on Retaining Wall Details
37	Bill of Reinforcing Steel & Bar Bending Diagrams
	Standards
38	Bridge Excavation (LRFD)
39	Supports & Spacers for Reinforcing Steel

DESIGN DATA

DESIGN SPECIFICATIONS:
AASHTO Specifications, 5th Edition and 2010
Interim Specifications. Load and Resistance Factor Design.

DESIGN LOADING:
HL-93
Design Dead Load includes an allowance of 15 psf for future wearing surface.

UNIT STRESSES:

Concrete (Grade 4.0) (AE) (SA)	f'c = 4 ksi
Reinforcing Steel (Grade 60)	fy = 60 ksi
Structural Steel (A709 Gr. 36)	fy = 36 ksi
Structural Steel (A709 Gr. 50W)	fy = 50 ksi
Structural Steel (M270 Gr. 50WT3)	fy = 50 ksi

TEMPERATURE: The design temperature for all dimensions is 60°F

QUANTITIES: Items not listed separately in the Summary of Quantities are subsidiary to other items in the proposal.

DIMENSIONS: All dimensions shown on the design plans are horizontal dimensions unless otherwise noted. Make necessary allowances for roadway grade and cross slope.

REINFORCING STEEL: All reinforcing steel dimensions are to the centerline of bars unless otherwise noted. All reinforcing steel shall conform to the requirements of ASTM A615, Grade 60.

Where non-coated bars come in contact with epoxy coated bars, they need not be coated.

CONCRETE: All concrete is bid as Concrete (Grade 4.0) (AE) (SA). All concrete is to be KCMMB 4K and is included in the bid item "Concrete (Grade 4.0) (AE) (SA)". Bevel all exposed edges of all concrete with a ¾" triangular molding, except where noted on the plans.

CONCRETE PLACING SEQUENCE: The sequence of placing concrete in the slab and curbs shall be as shown, or the Contractor may submit an alternate placing sequence for review. Submit the alternate placing sequence to the Engineer at the Preconstruction Conference. Include the proposed rate of concrete placement in C.Y./h, the plant capacity, placement direction, construction joint location, a description of the equipment used in placing the concrete, proposed admixtures, and the quantity of concrete in each placing segment. Any additional cost for the Contractor's alternate plan of placing concrete, including admixtures, shall be at the Contractor's expense and shall be considered subsidiary to the bid item, "Concrete (Grade 4.0) (AE) (SA)". Approval of the Contractor's alternate sequence is required prior to placement of concrete in the deck.

Place and hand vibrate all concrete for the abutments above the construction joint to the bottom of deck elevation just prior to the normal paving train operations. Do this work in a manner to avoid cold joints in either the slab or in the abutment.

DECK PROTECTIVE SYSTEM: Epoxy coat all reinforcing steel in the deck slab and rails.

CONSTRUCTION LOADS: Limited traffic is permitted on the new one-course deck during the curing period. Keep any exposed deck wet during the curing period. See KDOT Specifications Section 710 Tables 710-1 & 710-2 for additional information.

MULTI-LAYER POLYMER CONCRETE OVERLAY: No concrete curing membrane will be used on structures with a polymer overlay. Roughen the bridge deck surface using a burlap drag attached to the finish machine. When the date and temperature requirements of the specifications are met, grind profile, place a polymer overlay, and apply permanent pavement markings on the bridge deck. When the date and temperature requirements are not met, complete any required grinding and apply temporary pavement markings. Apply the polymer overlay according to the next available date(s) and temperature allowed per the current specifications. See KDOT specification for complete information.

ANCHOR BOLTS: Anchor bolts will adhere to KDOT Standard Specification Section 1600 (Grade 105 at Piers and Grade 55 at Abutments) with the following exception: the threads may be rolled or cut.

BEARING (STEEL REINFORCED ELASTOMERIC): Elastomeric Bearing Device shall be factory bonded to the steel sole plate and the steel base plate by a vulcanization process. The steel sole plate, steel base plate and swedge anchor bolt are subsidiary to the bid item, "Bearing (Steel Reinforced Elastomeric)" and shall be furnished by the bearing device fabricator.

BEARING (STEEL REINFORCED ELASTOMERIC) (Method A): Bearing devices at piers shall be fabricated with an elastomer satisfying:
Shore A Durometer Hardness of 60
Low Temperature Grade 3 requirements
Type A certification for elastomeric bearing device acceptance is required.
Include design method and all material properties on shop drawings.

ABUTMENT AGGREGATE DRAIN: See the General Notes on the "Abutment Aggregate Drain" sheet.

BACKFILL COMPACTION: Compact backfill at the abutments.

CONDUIT SYSTEM INSTALLATION: The Contractor shall install the conduit, conduit hangers, and PVC conduit sleeves at the abutments for the AT&T utility as shown on the plans. Material required, including conduit, conduit hangers, threaded rods, nuts, bolts and washers shall be in accordance with KDOT specifications and be provided to the Contractor by AT&T. The Contractor will coordinate with AT&T and schedule the conduit installation. Conduit shall be installed by the Contractor in accordance with KDOT specifications. The bid item "Electric Conduit (4.0") (Non-Metallic) (Installation Only) (Non-Participating)" will be full compensation for the additional time and labor required by the Contractor to install the conduit system.

Plotted By: cluedke
File: N0576001BBR-01_notes.dgn
Plot Date: 9/13/2013
Plot Location:

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.519.5 STA. 53+38.14
GENERAL NOTES
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	14	99

FALSEWORK PLANS: A licensed Professional Engineer shall design the falsework details. Details shall bear the seal of a licensed Professional Engineer. See the Bridge Design Manual, Section 5.1 "Review and Approval of Falsework Plans", for a listing of items to be included on the falsework plan. Submit electronic plans conforming to 105.10(b) of the Standard Specification with details in compliance with KDOT Specifications to the Field Engineer for review.

FALSEWORK INSPECTION: This project has falsework plan requirements which are considered "Category 1" by KDOT specifications. The falsework designer of record will conduct an inspection of the as-built falsework. The bid item, "Falsework Inspection" is full compensation for all materials, labor and equipment. See KDOT specifications.

FALSEWORK PLANS AND SHOP DRAWINGS: Use the U.S. Customary system of units on falsework plans and shop drawing details.

STRUCTURAL STEEL: The rolled beams, bearing stiffeners and splice plates shall meet AASHTO M270 Gr. 50WT3 requirements except as noted. All other structural steel shall meet ASTM A709 Gr. 50W, unless noted otherwise. Shop and Field Splices shall be made only where shown on the Contract Plans as a "splice" or as an "optional splice." Elimination of any "splice" may be requested.

MISCELLANEOUS STEEL: Steel for the Girder Chairs may be A36. Payment for this item shall be subsidiary to ASTM A709 Grade 50W Steel.

SHOP DETAILS: Reference blocking diagrams on the shop details to a level line running the entire length of the girder.

FABRICATION OF FIELD SPLICES: Prepare joints for the field splices in accordance with KDOT Specifications. Use Type "B" shop laydown.

WELDING: Material, fabrication and construction shall conform to KDOT Specifications. On the shop drawings, show a code or symbol at the tail of the weld symbol, which refers to an approved pre-qualified weld procedure.

WELDED STUD SHEAR CONNECTORS: Weld Shear Stud Connectors with automatically timed stud welding equipment connected to a suitable power source. All stud welding shall conform to KDOT Specifications.

BOLTS: All bolts, nuts and hardened flat washers shall conform to the heavy hex structural requirements of ASTM A325, Type 3, and KDOT Specifications unless otherwise noted. Direct Tension Indicators (DTIs) are to comply with the requirements of the latest edition of ASTM F959. No allowance will be made for high strength bolts used for permanent or temporary connections. This work is subsidiary to the bid item, "Structural Steel".

BOLTED CONNECTIONS: Girder Connections: Use $\frac{7}{8}$" diameter heavy hex structural bolts for the main member connections. Use $\frac{15}{16}$" diameter bolt holes. Do not ream during field erection. Accurately align all connections by driving $\frac{15}{16}$" diameter drift pins in all corners and in 1/4 of the remaining holes in each plate. See KDOT Specifications.

Secondary Member Connections: Use $\frac{3}{4}$" diameter heavy hex structural bolts for the secondary member connections. Use $\frac{13}{16}$" diameter bolt holes. Oversized and/or slotted holes, as specified in the KDOT Specifications, may be used in only one of the two members connected and must be shown in the approved shop drawings. Oversized and/or slotted holes may require additional standard hardened washers or plate washers. Report to the Engineer prior to any required field reaming that will remove more than $\frac{1}{4}$" of material from one ply of the connected parts.

Use Direct Tension Indicators (DTIs) on all high strength bolts. Place the DTI under the bolt head and turn the nut to tighten. This method is preferred whenever possible. Face the protrusions on the DTI to the underside of the bolt head. Place a hardened flat washer under the nut. See KDOT Specifications.

FILLETS:
Construct the finished deck to plan grade by varying the depth of the fillet over the beam to provide for beam profile, concrete dead load deflection and, if necessary, vertical curvature. After the beams are completely erected and the falsework bents are removed, profile each beam. Correct any variation between the actual profile and the concrete dead load deflection shown in the plans by varying the depth of the concrete fillets over the beams so that the finished floor is constructed to the theoretical grade. The Contractor shall submit a fillet diagram that shows fillet depths at tenth points of each beam. The fillet diagram shall be submitted to the Engineer 14 days prior to pouring the bridge deck. The minimum depth of the slab over the beam shall be $8\frac{1}{2}$ inches.

The theoretical amount of concrete required for the fillets is 11.6 C.Y. This amount of concrete is included in the Summary of Quantities. Any additional concrete required to construct the fillets will be subsidiary.

ERECTION: Bring each line of girders to the correct line, grade (or relative grade) and camber, and secure in place prior to connection of the girder field splices. Provide falsework bents as necessary to maintain the correct line and elevation. Leave the falsework bents in place until all girder splice connections are completed. Submit information which clearly shows the proposed layout and use of falsework bents. The Engineer shall approve such information prior to erection of structural steel.

ERECTION PLANS:
This is a Category B Structure. Submit detailed Erection Plans to the State Bridge Office (or Bureau of Local Projects) at least 4 weeks before beginning the erection process. Portions of the submitted details shall bear the seal of a licensed Professional Engineer. Identify, on the Erection Plans, the Erection Supervisor required by KDOT Specifications. No structural erection work will begin without approved erection plans.

PAINTING: Blast clean all surfaces of all weathering steel, including all contact surfaces of bolted connections, to meet SSPC-SP6 Specifications (latest Revision). Blast clean to meet SSPC-SP10 Specifications and prime coat the embedded portion of the girders, including the abutment diaphragms; the top flanges, including the shear studs; and the top flange splice plates. Paint the exterior surfaces of the exterior girders in accordance with KDOT Specifications.

PAINTING: The shop coats applied to Structural Steel shall conform to an inorganic zinc primer.

"Touch-Up": Prepare and paint all bolts, nuts, studs, and other small areas of damaged paint (1 yd^2 or less), requiring touch-up, with an approved organic zinc primer.

Top Flanges: (Studs applied in the shop) Apply a 3 mil primer coat of an approved inorganic zinc primer to the tops of the top flanges and to the studs.

(Studs applied in the field) Shop Work - Blast clean the tops of the top flanges to SSPC-SP10 Specifications (latest Revision).

Field Work - Blast clean the tops of the top flanges to SSPC-SP6 Specifications (latest Revision) before the studs are applied. After the studs are applied, blast clean the tops of the top flanges and the studs to SSPC-SP6 Specifications and paint with an approved organic zinc primer to a minimum dry film thickness of 3 mils.

BRIDGE EXCAVATION: All excavation shall be Class III. See the Bridge Excavation sheet for the limits of pay excavation.

BRIDGE BACKWALL PROTECTION SYSTEM: Apply a Bridge Backwall Protection System to the approach side of the abutments and the wings in accordance with KDOT Specifications and the manufacturer's recommendations. Cover the abutments and wings to the limits shown on the details, not including the top of the pavement rest. Prior to backfilling, repair any damage done to the system at no extra cost to the State. See the General Notes on the "Abutment Aggregate Drain" sheet.

BRIDGE DRAINS: Galvanize and apply a two-component, 98% polymeric epoxy, amido-amine primer tiecoat of approximately 1-$2\frac{1}{2}$ mils in thickness to the drains, prior to application of a water-borne acrylic finish coat. Follow the manufacturer's recommendations for application and cure time.

REMOVAL OF EXISTING STRUCTURE: Removal of existing superstructure is included in the bid item, "Removal of Existing Structures", Lump Sum. All materials removed from the existing structure shall become the property of the Contractor. Remove this material from the site.

DRILLING AND GROUTING: This item shall consist of grouting reinforcing steel, anchor bolts, tie bars, or dowel bars into the existing concrete, where required by the Engineer, with an epoxy grout. Follow KDOT Specifications 842 and any associated Special Provisions. Follow the manufacturer's directions for mixing, application and curing. The tools, materials, labor and incidentals necessary to complete the work shall be paid for per each by the bid item "Drilling and Grouting".

DEMOLITION PLANS: This is a Category B Demolition. Submit detailed Demolition Plans to the Field Engineer at least 2 weeks before the demolition meeting. Identify, on the plans, the Demolition Supervisor meeting the requirements of the KDOT Specifications. No Demolition work will begin without approved Demolition Plans. A Licensed Professional Engineer is not required.

TEMPORARY SHORING: The bid item "Temporary Shoring" includes all labor and material necessary to furnish shoring at the locations shown on the plans for the temporary bracing of the embankment during excavation. Maintain the temporary shoring until the Engineer authorizes its removal. The temporary shoring plans are to be designed and sealed by a registered Professional Engineer. Submit design calculations and shoring plans to the Field Engineer for review 3 weeks before work is scheduled to begin. Work shall not begin until the Engineer grants approval.

SLOPE PROTECTION: Remove and replace existing concrete slope protection to the limits required for new construction. Removal and replacement of existing concrete slope protection shall be subsidiary to the bid item "Rip Rap (Reinforced Concrete) (4")".

TRAFFIC CONTROL: The bridge will be closed to vehicular traffic during construction. Pedestrians will use the bridge sidewalks during construction. See traffic control sheets for details.

STREAM BED: Work done on the project shall not impact the stream bed.

Plotted By: dluedke | Plot Location:
File: N0576018BR-02_notes.dgn
Plot Date: 9/13/2013

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

GENERAL NOTES
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	15	99

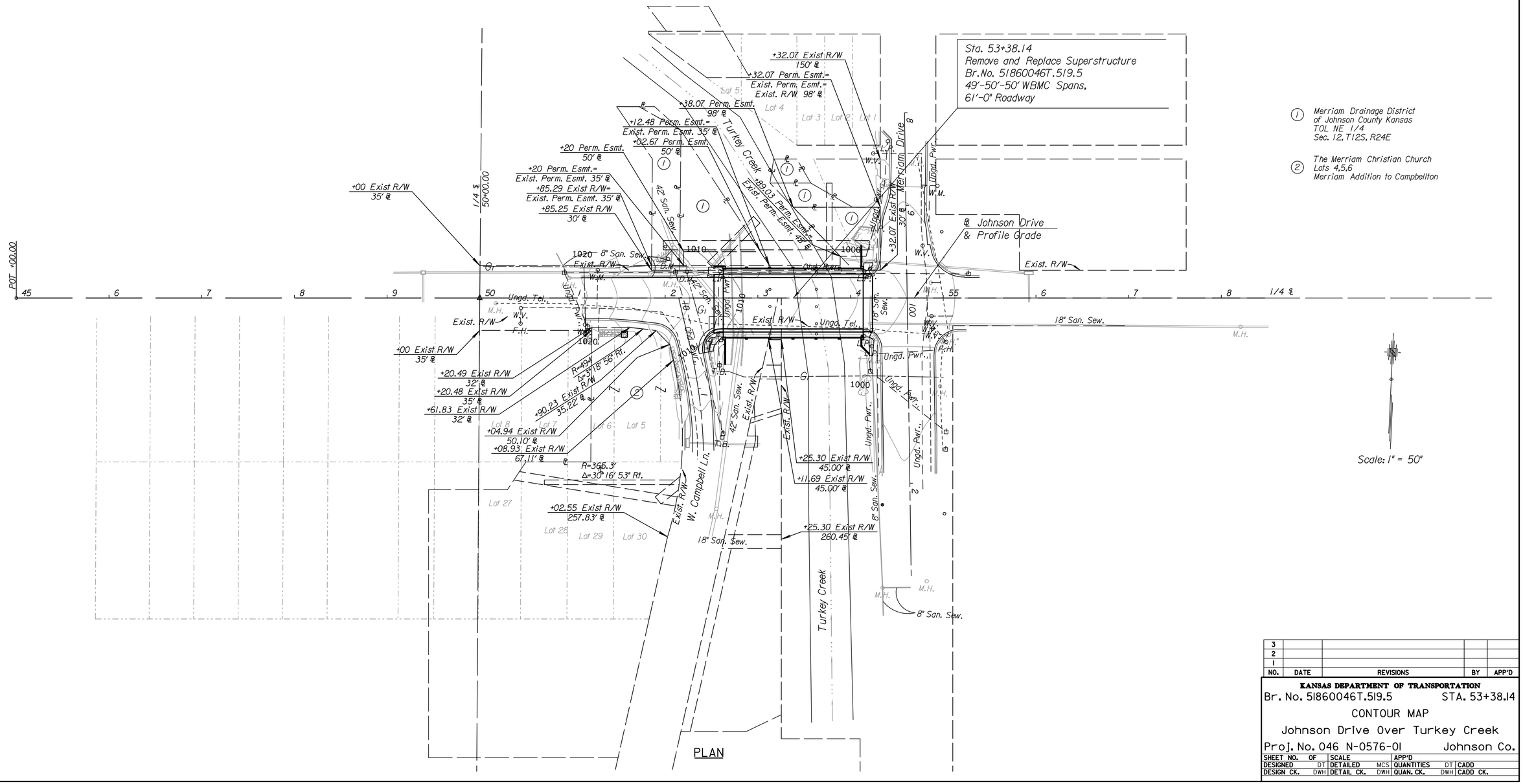
Control Point 100
55.51' Lt. of ℄ Sta. 52+00.25
N= 268,090.11 E= 2,251,231.51
1. Set 1/2" bar with GBA cap on property at 9404 Johnson Drive at NW quadrant of bridge.
2. Back asphalt curb east side of parking lot. 1.85' W.
3. Near face of steel bird house pole. 14.10' N.E.
4. ℄ manhole. 7.55' E.N.E.

Control Point 101
232.90' Rt. of ℄ Sta. 55+01.31 Elev.=1002.25'
N= 267,815.85 E= 2,251,545.52
1. Set 1/2" bar with GBA cap on property at 5911 Merriam Drive.
2. Near Face North Sign post Merriam Feed and Garden Center. 8.38' E.
3. Northeast corner curb inlet east side Merriam Drive. 13.00' N.N.W.
4. ℄ Water meter. 3.20' W.

℄ P.O.T. Sta. 52+00.00
N 268,034.65 E 2,251,233.82
1. Not Set.

Control Point 102
71.59' Lt. of ℄ Sta. 54+98.33 Elev.=1003.51'
N= 268,119.88 E= 2,251,528.54
1. Chiselled "+" cut top back of curb at corner handicap parking stalls on property 9320 Johnson Drive.
2. Near face 6" maple tree in island. 8.45' N.E.
3. Near face at top of 48" diameter concrete planter. 4.45' W.
4. North edge concrete handicap walk. 7.76' S.

℄ P.O.T. Sta. 55+00.00
N 268,048.44 E 2,251,533.50
1. Not Set.



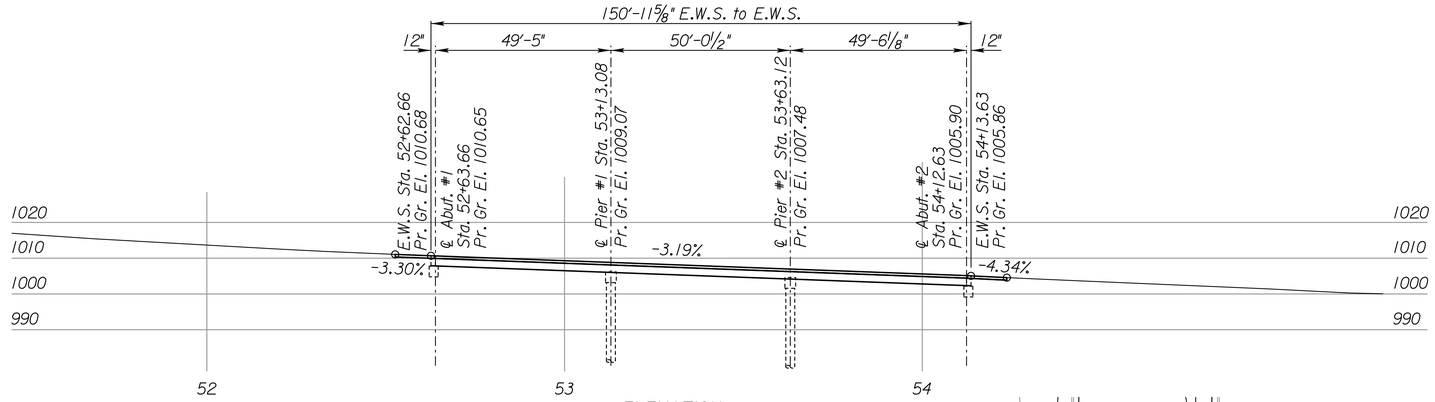
Plotted By: cluedke
File: N0576018BR-03_Conmap.dgn
Plot Date: 9/13/2013
Plot Location:

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

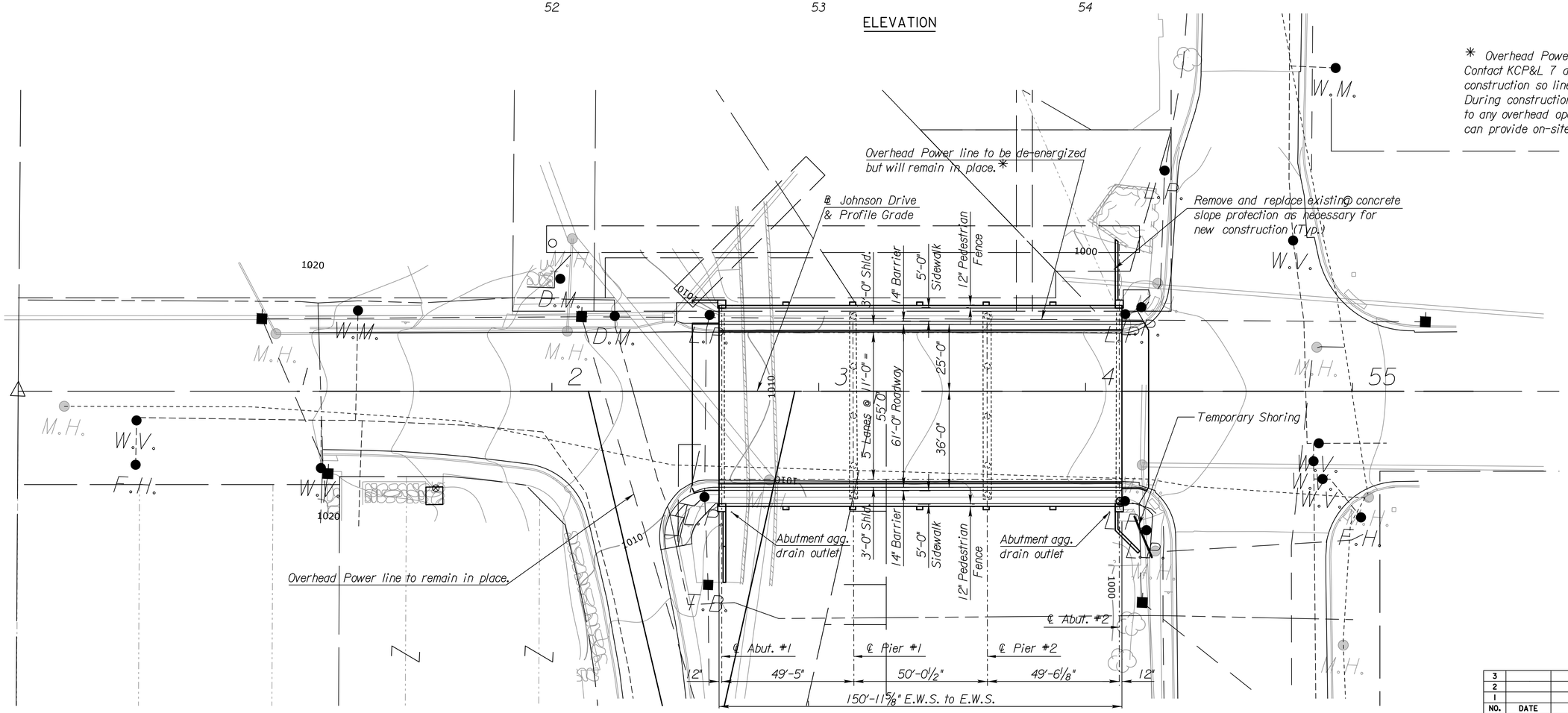
KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.519.5 STA. 53+38.14
CONTOUR MAP
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF	SCALE	APP'D	
DESIGNED DT	DETAILED MCS	QUANTITIES DT	CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH	CADD CK.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	16	99



ELEVATION



PLAN

* Overhead Power line will remain in place. Contact KCP&L 7 days prior to beginning construction so lines can be de-energized. During construction, contact KCP&L 7 days prior to any overhead operations so that KCP&L can provide on-site personnel.

Scale: 1" = 20'

Plotted By: cluedke
Plot Location:
File: N057601BBR-04_Conlay.dgn
Plot Date: 9/13/2013

BM #825 2" Aluminum Johnson County Public Works monument at S.E. corner Johnson Drive Bridge over Turkey Creek. 41.26' Rt. ℄ Sta. 54+12.80 Elevation= 1006.06'

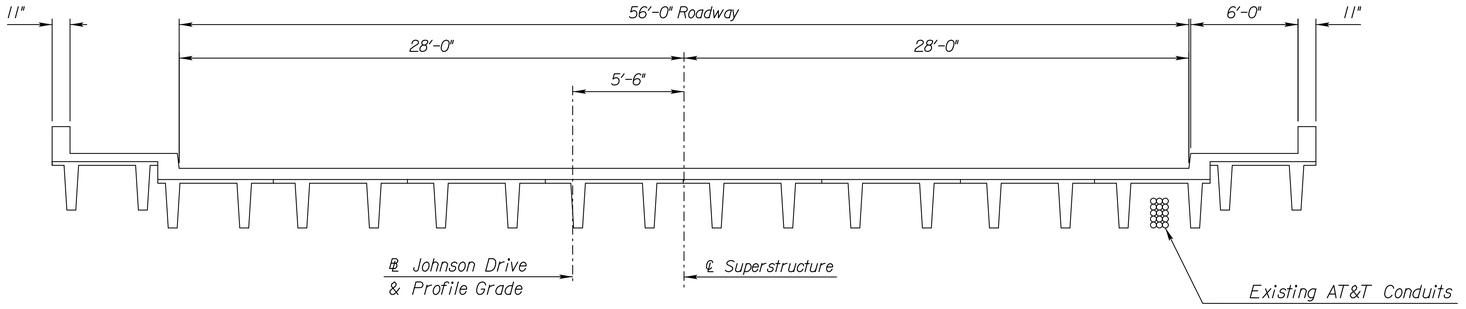
BM #103 Chiselled "□" top N.E. corner curb inlet west side Merriam Drive. 1st inlet south of Johnson Dr. 223.25' Rt. ℄ Sta. 54+34.39 Elevation= 1001.74'

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

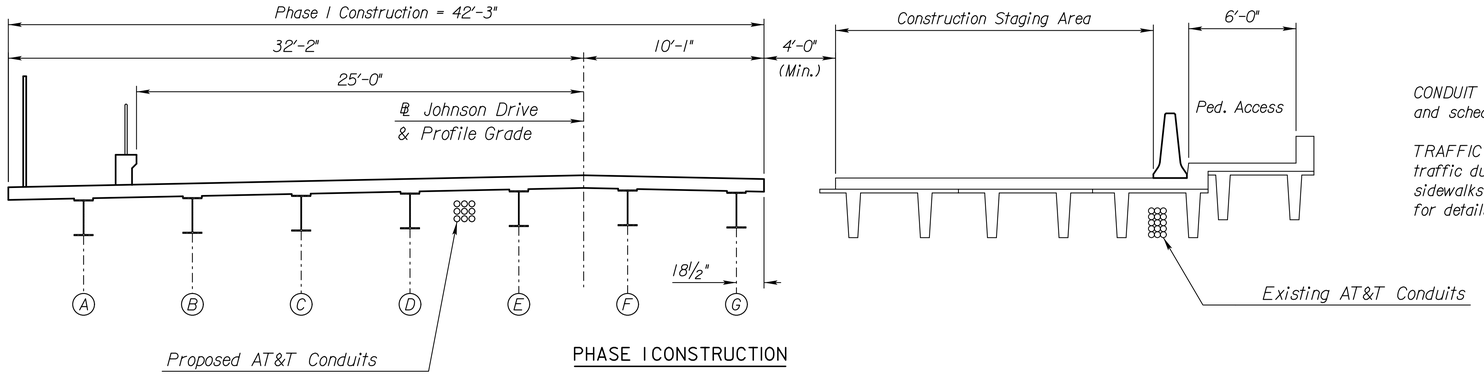
KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.519.5 STA. 53+38.14
CONSTRUCTION LAYOUT
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF	SCALE	APP'D
DESIGNED DT	DETAILED MCS	QUANTITIES DT CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH CADD CK.

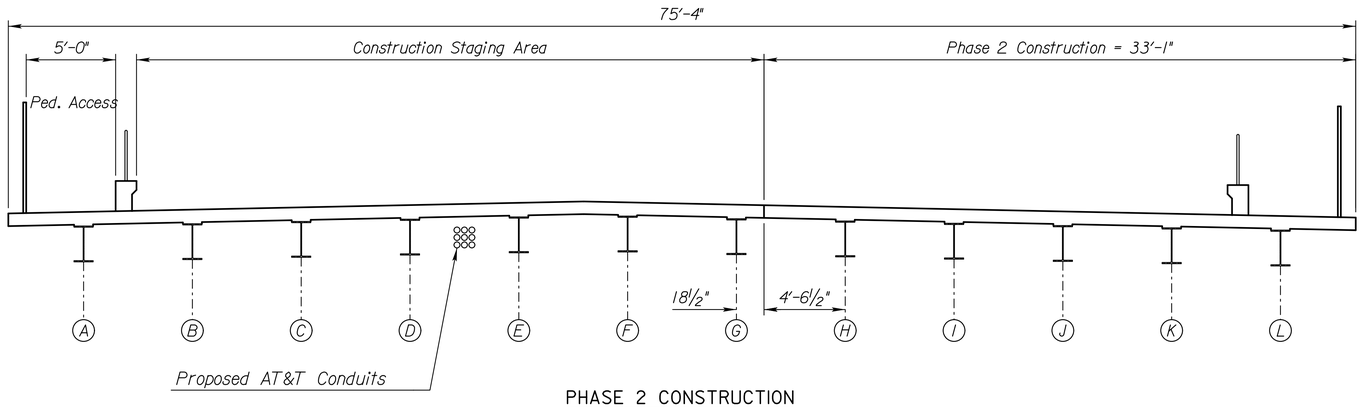
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	17	99



EXISTING DECK SECTION



PHASE I CONSTRUCTION



PHASE 2 CONSTRUCTION

CONDUIT SYSTEM: The contractor will coordinate with AT&T and schedule the conduit installation.

TRAFFIC CONTROL: The bridge will be closed to vehicular traffic during construction. Pedestrians will use the bridge sidewalks during construction. See traffic control sheets for details.

Plot Location:
Plotted By: cluedke
File: N0576101BBR-05_Phased_Construction.dgn
Plot Date: 9/13/2013

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

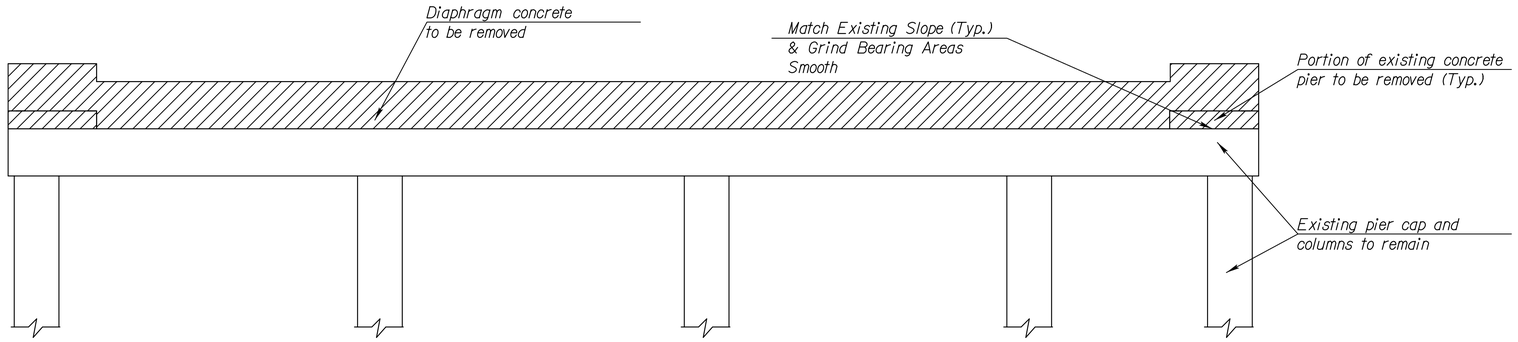
PHASED CONSTRUCTION

Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	18	99



TYPICAL PIER DEMOLITION

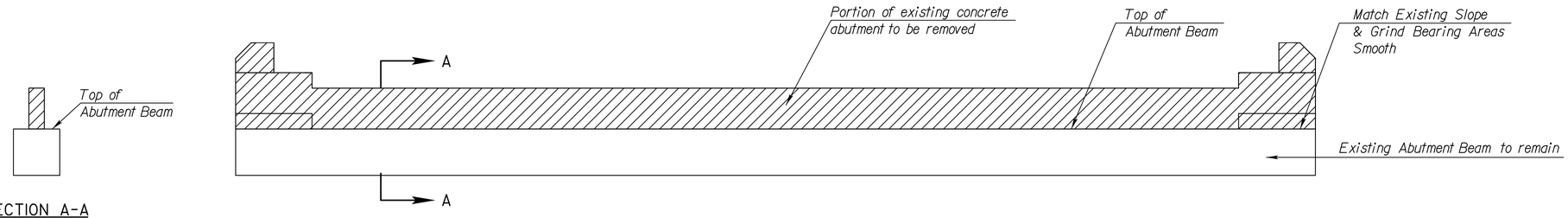
Notes:

Remove existing pier beam and abutment beam reinforcement within limits of concrete removal to avoid conflict with new construction.

Removal shall conform to KDOT Standard Specifications, Section 202.

The contractor shall take care not to damage existing concrete that is to remain. Use chipping hammers no larger than 15 pound class to remove concrete within 6" of concrete that is to remain. Use chipping hammers no larger than 35 pound elsewhere. Damage done to concrete that is to remain will be repaired at the contractor's expense.

Removal of existing superstructure and partial removal of existing substructure is included in the bid item, "Removal of Existing Structures", Lump Sum. All materials removed from the existing structure shall become the property of the Contractor. Remove this material from the site.



SECTION A-A

TYPICAL ABUTMENT DEMOLITION

Plotted By: oluedke | Plot Location:
File: N0576018BR-06_Demolition.dgn
Plot Date: 9/13/2013

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

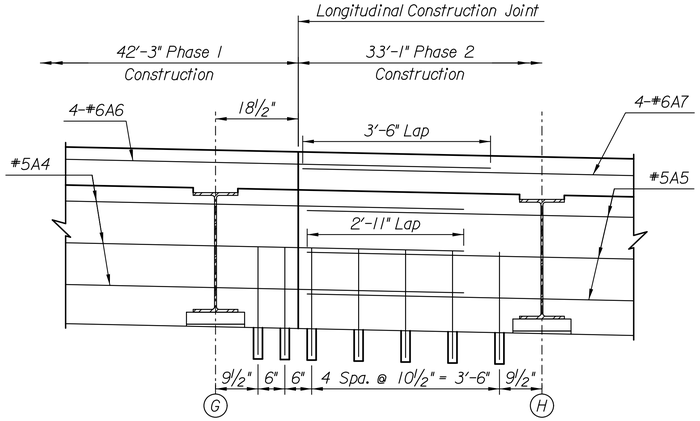
Br. No. 51860046T.519.5 STA. 53+38.14

DEMOLITION DETAILS
Johnson Drive Over Turkey Creek

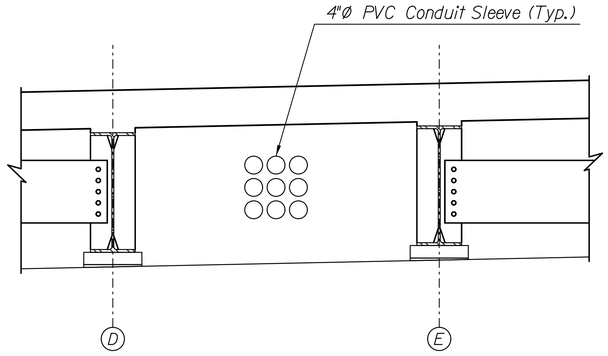
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

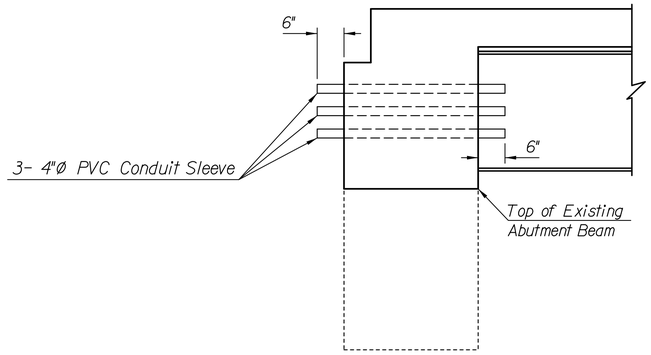
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	19	99



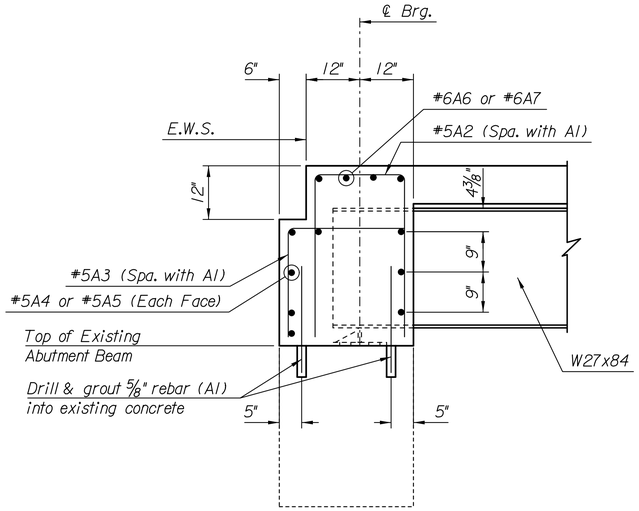
ELEVATION AT LONGITUDINAL CONSTRUCTION JOINT



ABUTMENT ELEVATION AT CONDUIT LOCATION



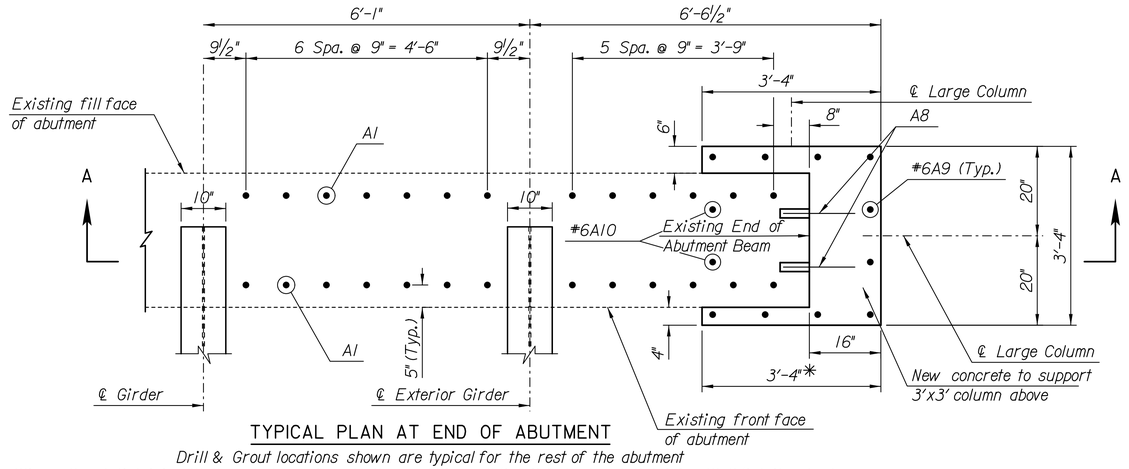
ABUTMENT SECTION AT CONDUIT LOCATION



TYPICAL ABUTMENT SECTION

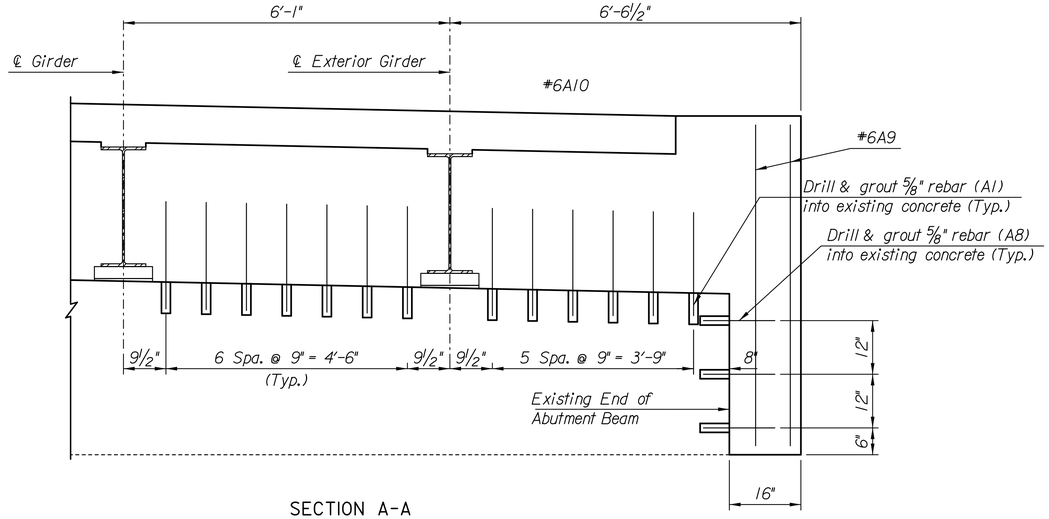
Drill & grout bars into the existing abutment. This work will be paid for per each in the bid item "Drilling & Grouting".

Depth of Drill & Grout for A1 and A8 bars shall be in accordance with the grout manufacturers recommendations. Lengths of the A1 and A8 bars in the bill of reinforcing assume a 6" embedment depth. Adjust the length of the A1 and A8 bars accordingly. Use a pachometer prior to drilling to locate and avoid existing reinforcing steel.



TYPICAL PLAN AT END OF ABUTMENT

Drill & Grout locations shown are typical for the rest of the abutment
Wingwall and Retaining Walls not shown for clarity, see "Wingwall Details" sheet and Retaining Wall Details sheets.



SECTION A-A

Drill & Grout locations shown are typical for the rest of the abutment
Wingwall and Retaining Walls not shown for clarity, see "Wingwall Details" sheet and Retaining Wall Details sheets.

Note:
Cast 4"Ø PVC conduit sleeves into the new diaphragm concrete. Coordinate layout of sleeves with AT&T. Shift abutment reinforcing as necessary to avoid conduit sleeves. Cost of providing and installing conduit sleeves shall be subsidiary to the bid item "Electric Conduit (4.0") (Non-Metallic) (Installation Only) (Non-Participating)".

See "Railing & Aesthetic Column Details" sheet for column reinforcing embedded in abutment.

* Formline this surface to match large column details shown on aesthetic sheets. (Above the beam seat elevation only.)

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

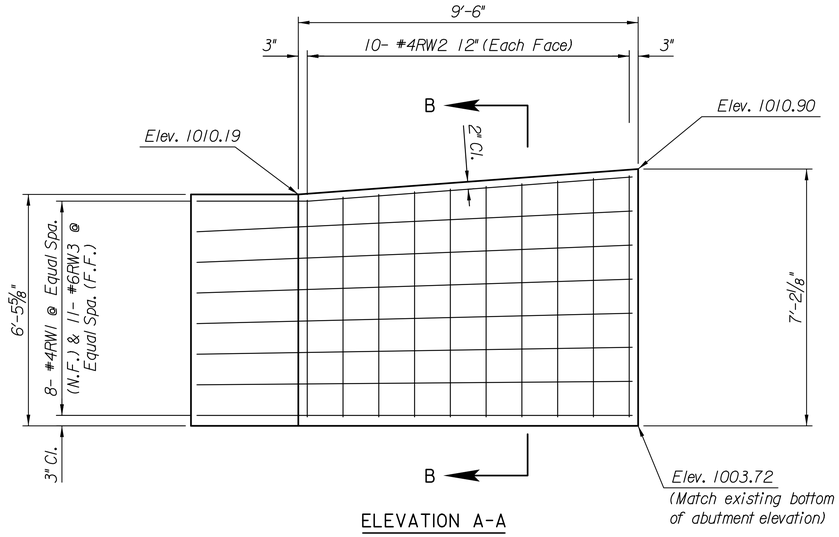
ABUTMENT DETAILS
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

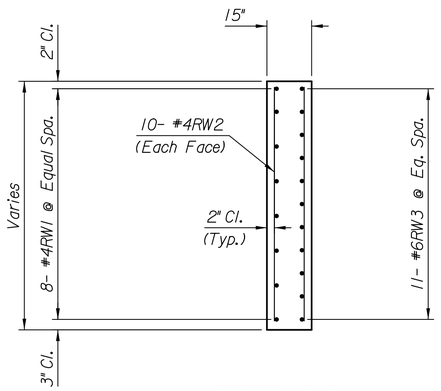
SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

Plotted By: cluedke Plot Location:
File: N0576O1BBR-07_Abutment Details.dgn
Plot Date: 9/13/2013

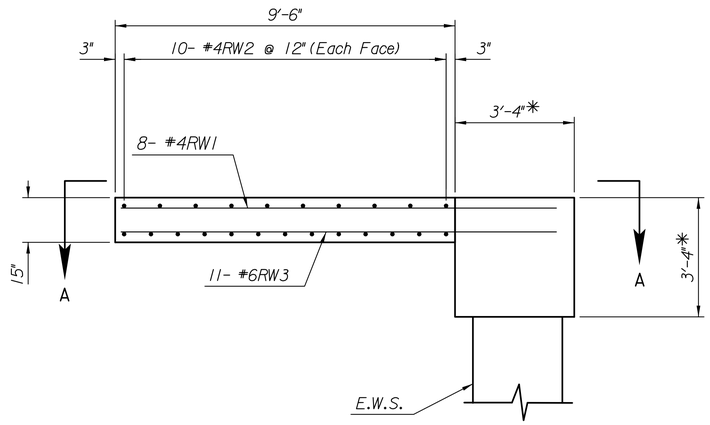
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	20	99



ELEVATION A-A



SECTION B-B



PLAN- NORTH END ABUTMENT #1

* *Formline this surface to match large column details shown on aesthetic sheets. (Above the beam seat elevation only.)*

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

WINGWALL DETAILS
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF	SCALE	APP'D
DESIGNED DT	DETAILED MCS	QUANTITIES DT	CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH	CADD CK.

Plotted By: dluedke Plot Location:
File: N0576018BR-08_Retaining Wall.dgn
Plot Date: 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	21	99

GENERAL NOTES

GEOSYNTHETICS: Use material that complies with KDOT Specification Section 1710 Class 2 subsurface drainage fabric. Place the Class 2 subsurface drainage fabric on graded and compacted material shaped as shown. Allow for enough material so that the top can be overlapped and the end folded to completely enclose the aggregate drain. Place the perforated drain pipe and couple to non-perforated pipe as shown. Allow the non-perforated pipe to pass through a hole carefully cut in fabric. Place aggregate within fabric to just leave the top of the pipe visible. Verify the slope of the pipe, that it is not damaged or displaced and that the couplers are firmly coupled. Continue to back fill to the elevation and shape shown. Lap the top of the fabric a minimum of 3'-0", fold and wrap the ends to enclose the drainage materials. Secure the folds and wraps by sewing or approved methods.

AGGREGATE: Use aggregates that complies with KDOT Specifications for SB-1, SB-2 or SB-3.

BASE COURSE REINFORCEMENT: Use "Base Course Reinforcement" that complies with KDOT Specification Section 1710 or approved material. Place this material in uniform layers without gaps or sags per the manufacturer's recommendations.

GEOFOAM: Use "Geofoam" that complies with KDOT Specification 07-2005 latest revision or approved material. Bond this material to the back wall protection using materials recommended by the manufacturer.

SOIL CAP: The soil will have a Unified Soil Classification of CL or ML according to ASTM D2487. Compact to Type A, MR-90.

PIPE: Place perforated pipe within the limits and use non-perforated pipe outside the limits of the Abutment Aggregate Drain.

ABUTMENT AGGREGATE DRAIN: The Bridge Contractor shall excavate to the limits shown on the Bridge Excavation Sheet. Backfill, compact & grade the cohesive soil to the limits shown. Place the bridge backwall protection, geofoam, geotextile, perforated pipe, alternating layers of aggregate and base course reinforcement as shown. Place the outlet pipe, the CMP, and the backfill. Guide post and coarse aggregate are subsidiary to this bid item. Guide post and coarse aggregate are not required if the CMP empties onto Slope Protection. Enclose the entire Abutment Aggregate Drain with the geotextile

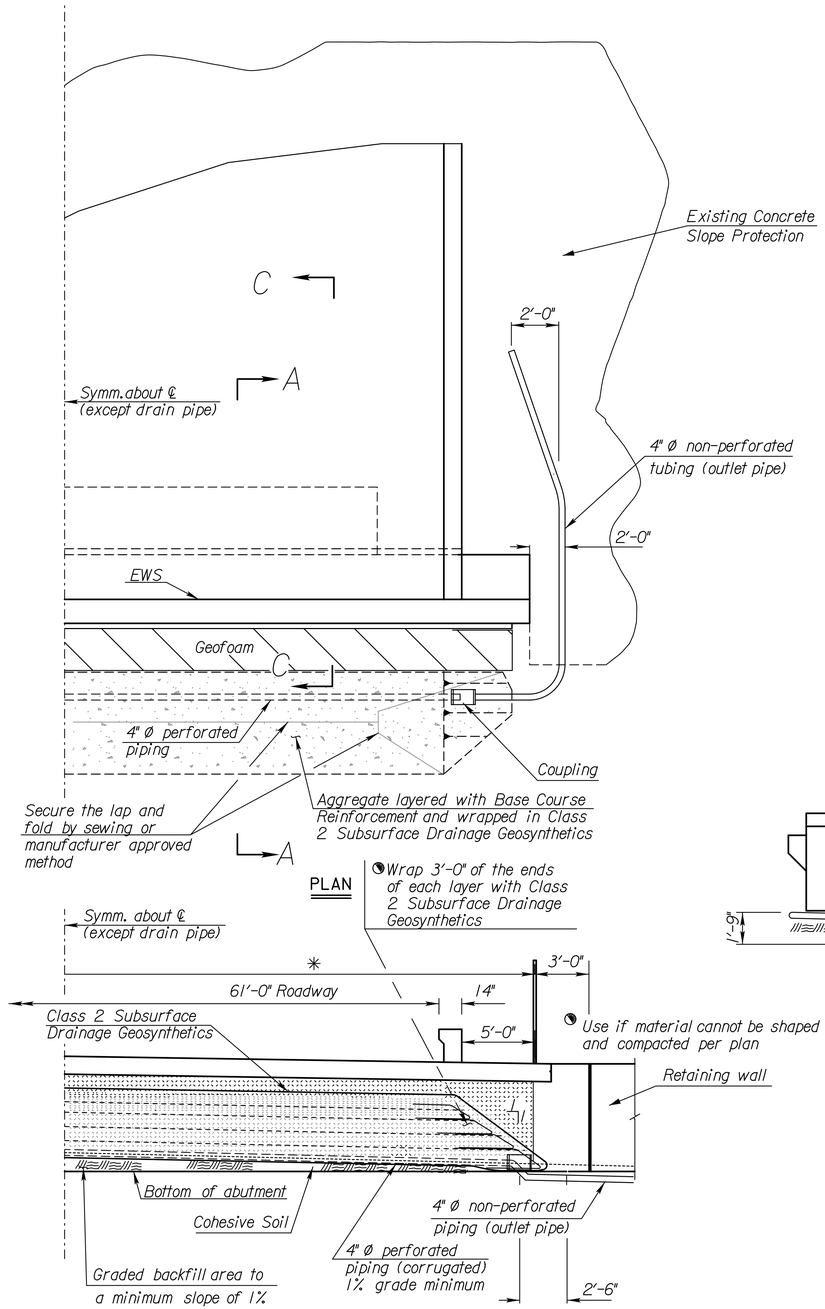
BRIDGE BACKWALL PROTECTION SYSTEM: Apply a non coal-tar Bridge Backwall Protective System to the approach side of the abutments and the wings in accordance with KDOT Specifications and the manufacturer's recommendations. Cover the abutments and wings to the limits shown on the details. Repair any damage done at no charge to the state.

Compact the abutment backfill. See the KDOT Specifications.

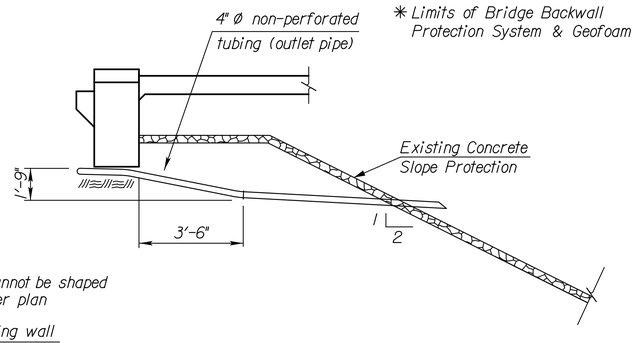
Perforated pipe and non-perforated outlet pipe shall be corrugated polyethylene tubing conforming to the KDOT Specifications.

Fit the outlet pipe end section with 1/4" galvanized mesh screen to prevent the entrance of rodents. Seal the joint between the outlet pipe and the end section with a joint sealer. Place Coarse aggregate at the outlet end as shown.

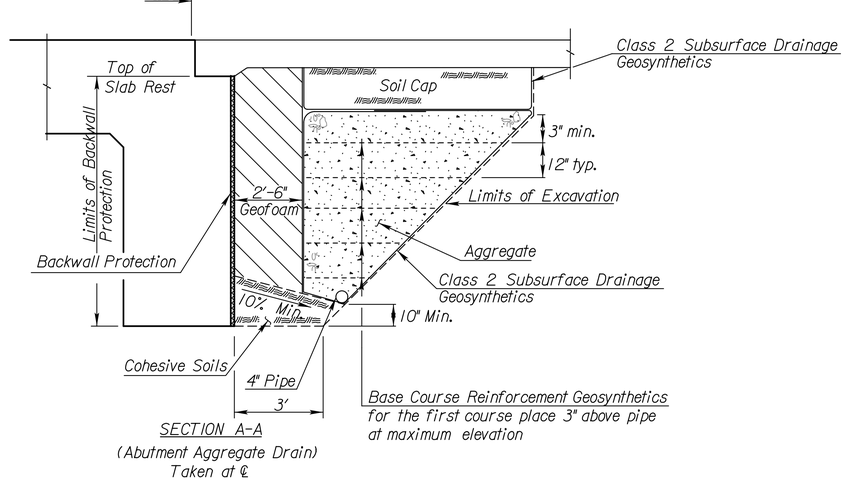
COHESIVE SOILS: Grade the bottom surface of the excavated area to drain as shown. Backfill this area with a cohesive type of soil. The soil will have a Unified Soil Classification of CL, CH, ML or MH according to ASTM D2487. Classification System with a minimum plasticity index of 13. Compact the material to Type A, MR-90 specifications. If the plasticity index cannot be met add and mix Bentonite, to the soil prior to placement and compaction so that the PI≥ 13.



ELEVATION



SECTION C-C



SECTION A-A
(Abutment Aggregate Drain)
Taken at ℄

SUMMARY OF QUANTITIES (2 Abutments)		
Abutment Aggregate Drain	62	Cu. Yds.
Bridge Backwall Protection System	90	Sq. Yds.
Items subsidiary to Abutment Aggregate Drain		
4" Ø Perforated Pipe	148	Lin. Ft.
4" Ø Outlet Pipe	44	Lin. Ft.
6" Ø CMP		Lin. Ft.
Guide Post		Each
Soil Cap	31	Cu. Yds.
Geosynthetics (Class 2 Subsurface Drainage)	244	Sq. Yds.
Geosynthetics (Base Course Reinforcement)	878	Sq. Yds.
Geofoam	65	Cu. Yds.

NO.	DATE	REVISIONS	BY	APP'D
4	7/23/12	Added Soil Cap to Summary	JPJ	TLF
3	5/17/12	Added Aggregate Drain	JPJ	TLF
2	7/14/08	Change Type 'C' Compaction to 'B'	JPJ	KFH
1	4-01-04	Current release		

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

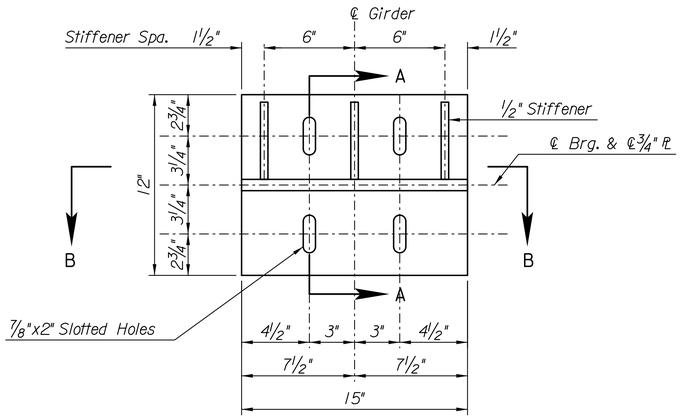
ABUTMENT AGGREGATE DRAIN

Proj. No. 046 N-0576-01 Johnson Co.

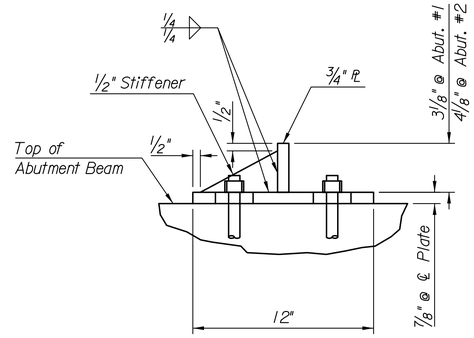
SHEET NO. OF	SCALE	APP'D	
DESIGNED JPJ	DETAILED JPJ	QUANTITIES	CADD RAA
DESIGN CK.	DETAIL CK.	QUAN. CK.	CADD CK.

Std. Base File: br104h.dgn
Plotted By: cluedke
File: N057601BBR-09_Abutment Aggregate Drain.dgn
Plot Date: 9/13/2013
Plot Location:

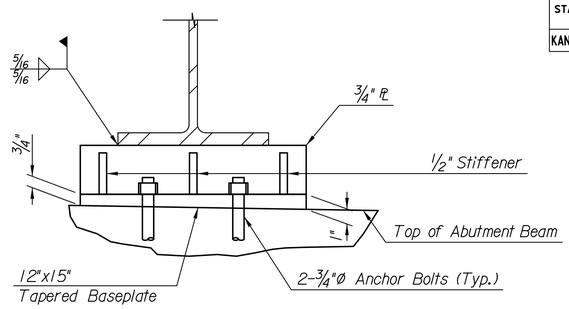
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	22	99



PLAN



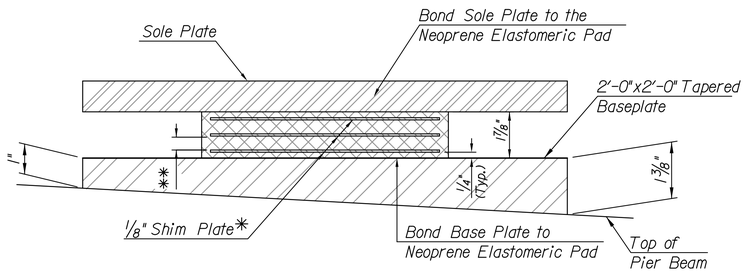
SECTION A-A



SECTION B-B

GIRDER CHAIR AT ABUTMENTS

(At the Contractor's option, A709 Gr. 36 Steel may be used for girder chairs.)

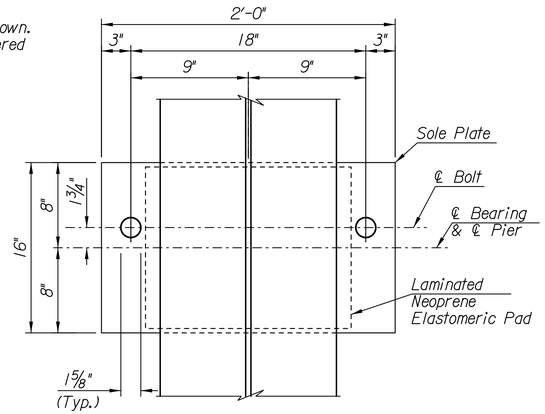


* *The required shim plates shall be placed between layers of Elastomer and molded together to form an integral unit.*

** *Layers of ½" Elastomer Alternating with ⅛" Steel Shim Plate.*

LAMINATED NEOPRENE ELASTOMERIC BEARING DEVICE @ PIERS

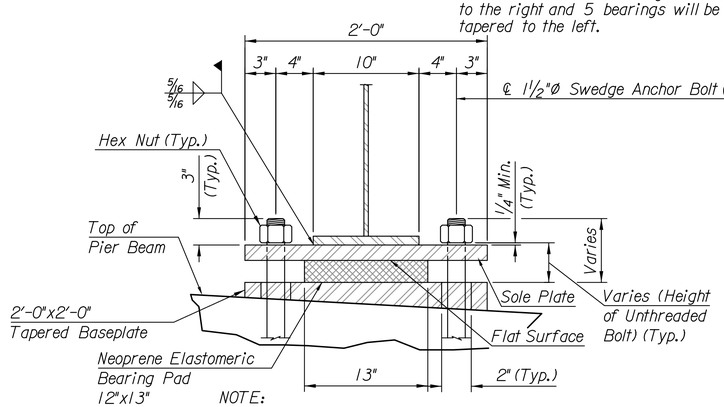
Note:
Baseplate tapered to the right is shown. At each pier, 7 bearings will be tapered to the right and 5 bearings will be tapered to the left.



PART PLAN VIEW SHOWING SOLE PLATE

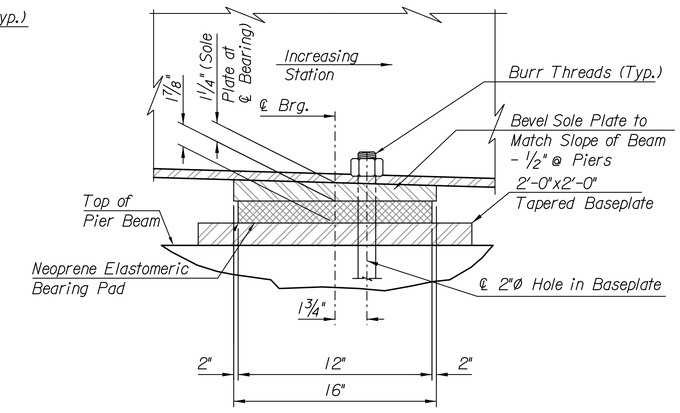
GIRDER CHAIRS AT ABUTMENTS

Note:
Baseplate tapered to the right is shown. At each abutment, 7 bearings will be tapered to the right and 5 bearings will be tapered to the left.



NOTE:
Limit Maximum Temperature at face of Sole Plate next to the neoprene pad to 300°F during welding of the beam.

END VIEW



SIDE VIEW

GENERAL NOTES:

Elastomeric Bearing Device shall conform to KDOT Standard Specifications and current Special Provisions.

The sole plate shall be furnished with the bearing and field welded to the girders. The base plate shall be furnished with the steel bearing.

Structural steel for sole plate and base plate shall be ASTM A709 Grade 50W and shall be coated with a minimum of two coats of inorganic zinc primer (125 micrometers minimum). After erection, apply the water-borne acrylic finish coat to the exposed surfaces of the sole plate and base plate.

The accepted quantity of elastomeric bearing assemblies, complete-in-place, will be paid for per each for "Bearing (Steel Reinforced Elastomeric)".

Anchor bolts shall meet the requirements of KDOT Specifications for Swedged Anchor Bolts, Type II (105 ksi @ Piers & 55 ksi @ Abutments). Anchor Bolts shall be furnished with heavy hexagonal nuts and washers per KDOT Specifications.

All structural steel for the anchor bolts and heavy hexagon nuts shall be hot dipped galvanized in accordance with ASTM A153.

Payment for the sole plate, base plate, anchor bolts, heavy hexagon nuts and washers, drilling and grouting shall be included in the cost of the bearing device and be subsidiary to "Bearing (Steel Reinforced Elastomeric)".

Anchor bolts shall be installed by drilling and grouting. Depth of drill and grout shall be according to the grout manufacturer's recommendations. Use a pachometer to locate and avoid existing reinforcing steel prior to drilling.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

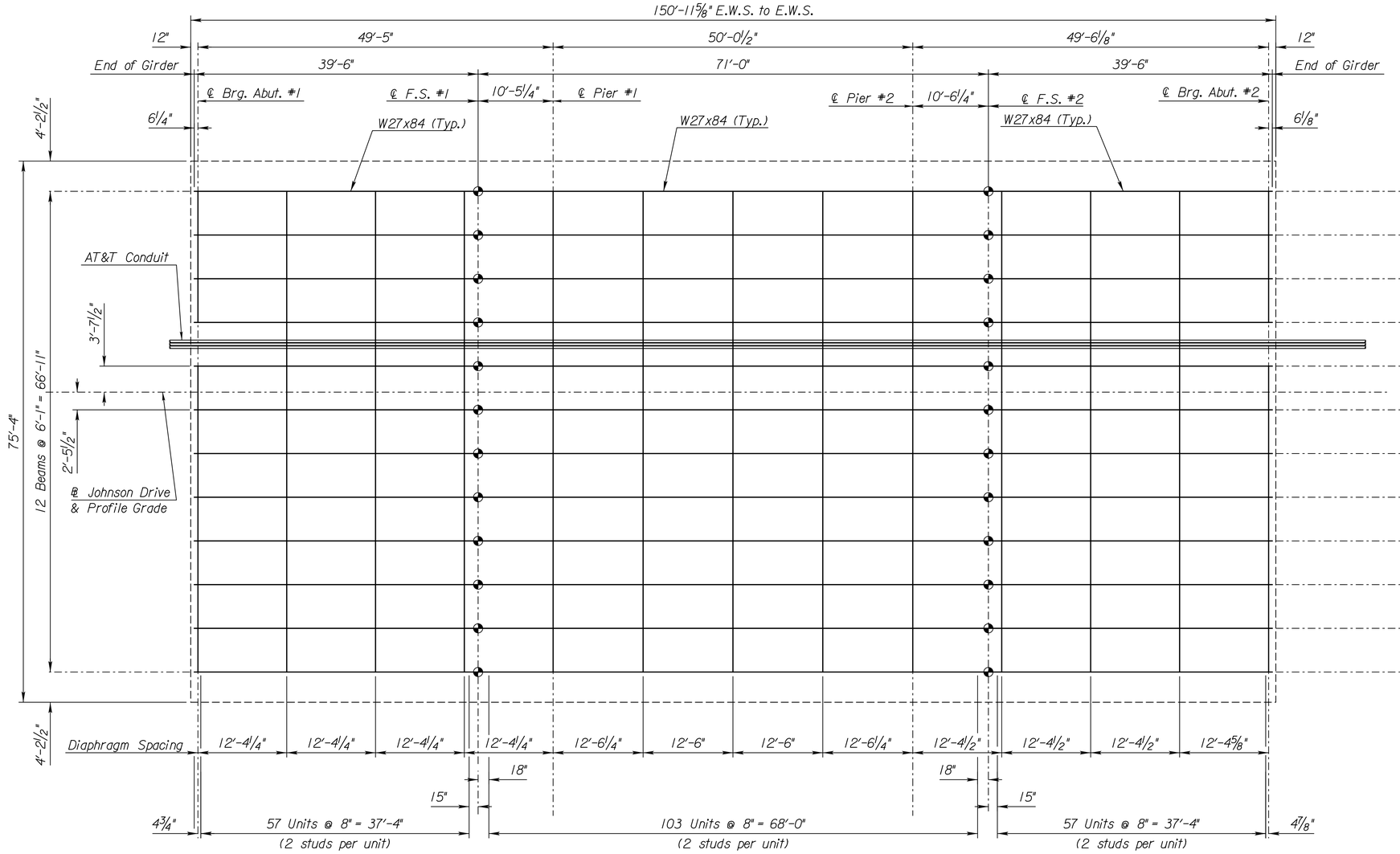
BEARING DEVICES
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

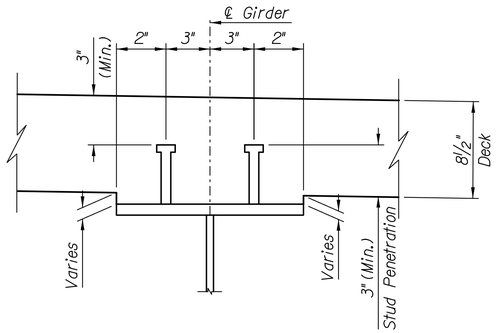
Plot Location:
Plotted By: cluedke
File: N0576018BR-10_Bearings.dgn
Plot Date: 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	23	99



FRAMING PLAN

(All dimensions are horizontal.)



All Welded Stud Shear Connectors shall be 7/8" Ø x 6" Welded Stud Shear Connectors shall conform to KDOT Specs. Length of stud shall satisfy stud penetration after girder erection.

SHEAR CONNECTOR DETAILS

HOLES AT GIRDER ENDS

* *Blast and Prime only*

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

FRAMING PLAN

Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

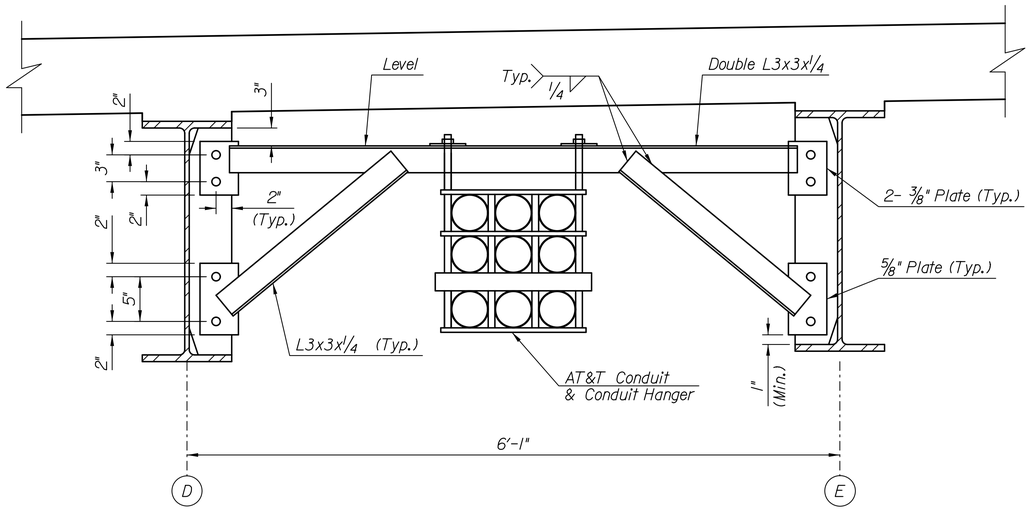
SHEET NO. OF	SCALE	APP'D	
DESIGNED DT	DETAILED MCS	QUANTITIES DT	CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH	CADD CK.

Plotted By: *dluedke* Plot Location:

File: *N0576001BBR-11_Framing_Plan.dgn*

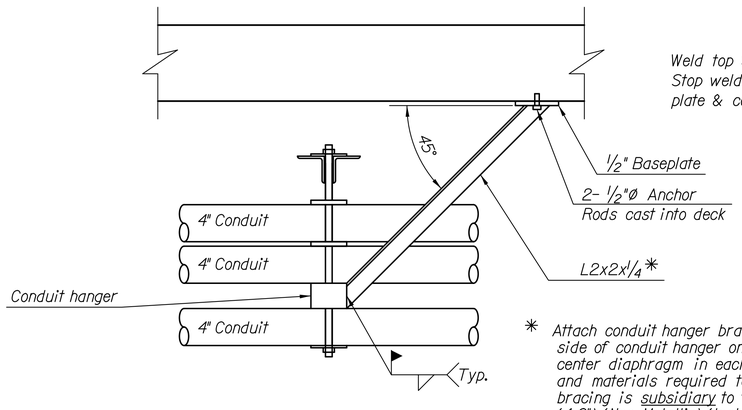
Plot Date: *9/13/2013*

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	24	99



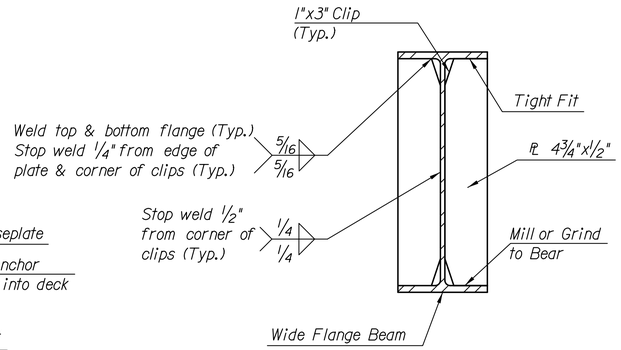
DIAPHRAGM AT CONDUIT LOCATIONS DETAIL

AT&T conduit and conduit hangers to be provided by AT&T and installed by Bridge Contractor.

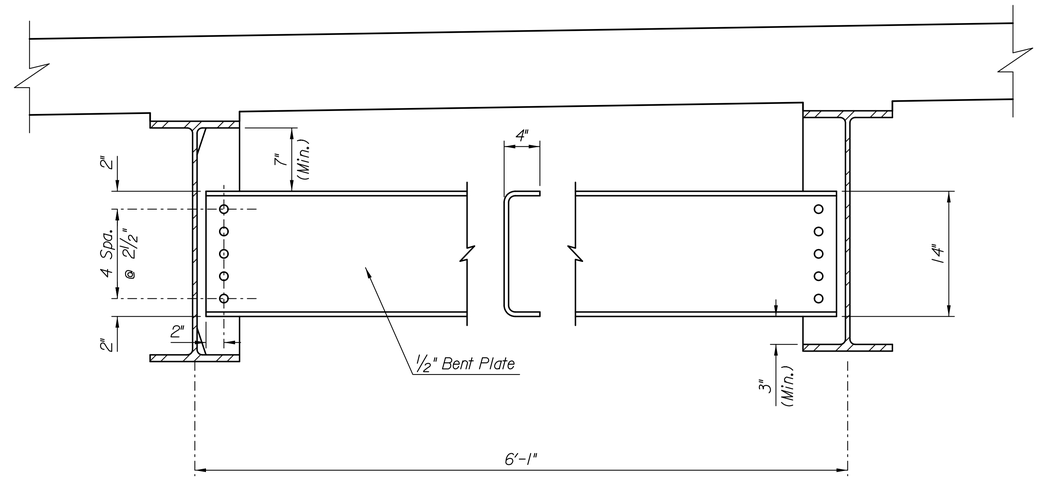


SECTION OF CONDUIT HANGER

* *Attach conduit hanger bracing on each side of conduit hanger only at center diaphragm in each span. Cost of labor and materials required to install conduit bracing is subsidiary to "Electric Conduit (4.0") (Non-Metallic) (Installation Only) (Non Participating)".*



TYPICAL BEARING STIFFENER AND DIAPHRAGM CONNECTION PLATE



TYPICAL DIAPHRAGM DETAIL

Notes:
All main load-carrying members shall meet AASHTO M270 (Gr. 50WT3) requirements.
Main load-carrying members include all rolled beams, bearing stiffeners and splice plates.
All other structural steel shall meet ASTM A709 (Gr. 50W) requirements.

All bearing stiffeners shall be placed normal to girder. All intermediate diaphragm connection plates shall be placed parallel to intermediate diaphragms.

All bearing stiffeners and girder ends, except at field splices, shall be vertical after final erection. All other stiffeners and all field splices shall be normal to the top flange.

See "Drain Details" sheet for details and locations of Drain Tube Mounting Connection Plate.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

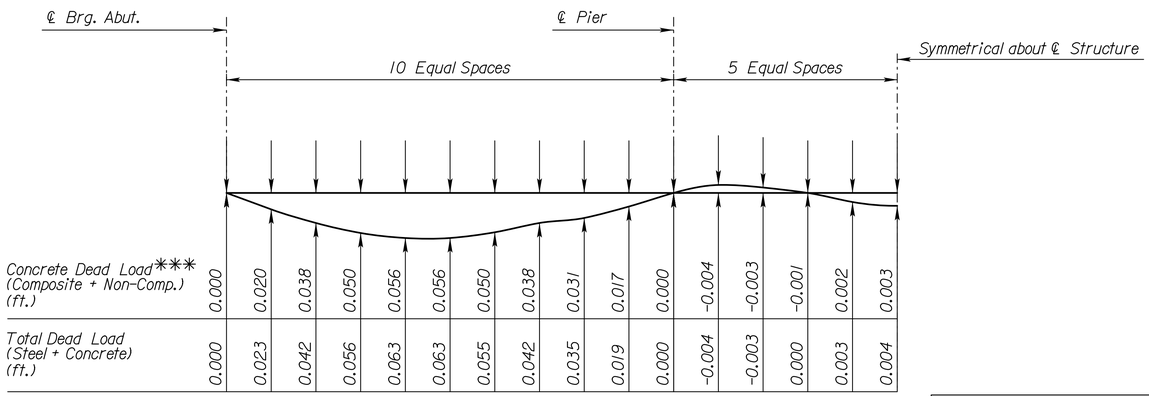
GIRDER DETAILS
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO.	OF	SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

Plotted By: oluedke | Plot Location:
File: N0576OIBBR-12_Girder Details.dgn
Plot Date: 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	25	99

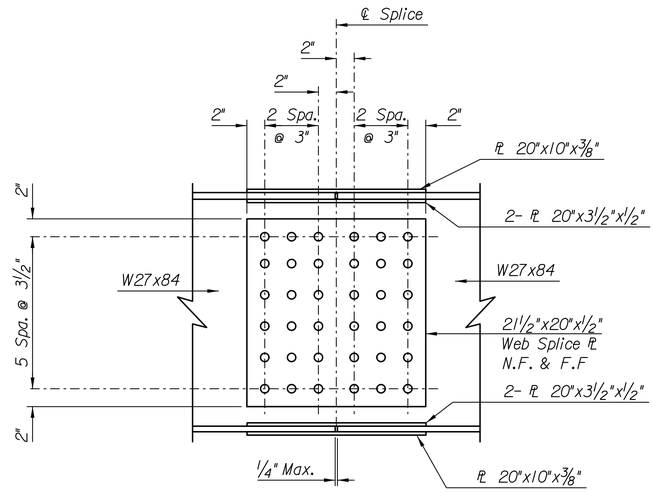


DEAD LOAD DEFLECTION
AT TENTH POINTS FOR EXTERIOR GIRDER

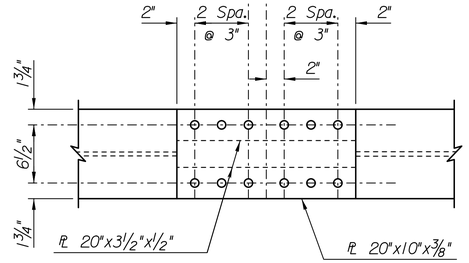
*** Use these ordinates to compute beam fillet depth.

STRUCTURAL STEEL QUANTITIES (lbs.)		
MEMBER	M270 Gr. 50WT3	A709 Gr. 50W
Rolled Beams	151,899	
Bearing Stiffeners	1,644	
Flange Splice Plate	1,974	
Web Splice Plate	2,926	
Connection Plates		3,871
Diaphragms		27,682
Abutment Girder Chairs		1,431
Total	158,443	32,984

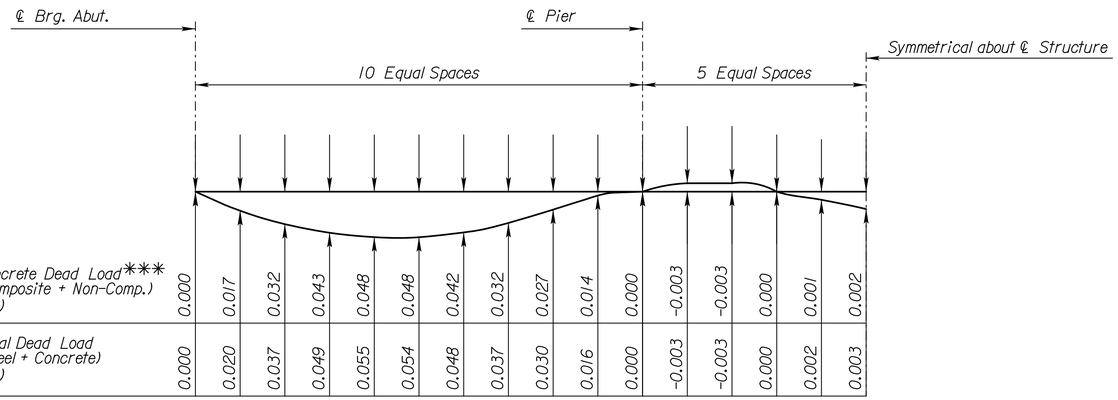
Number of Welded Stud Shear Connectors (7/8" dia. x 6" Length) = 5,208



TYPICAL WEB SPLICE PLATE
AND BOLT LAYOUT



TYPICAL FLANGE SPLICE PLATE



DEAD LOAD DEFLECTION
AT TENTH POINTS FOR INTERIOR GIRDER

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

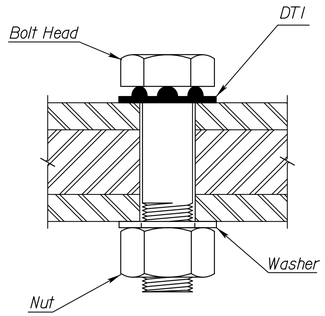
KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.519.5 STA. 53+38.14

GIRDER DETAILS
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

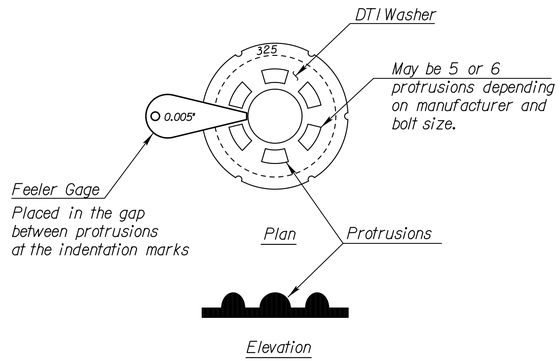
SHEET NO. OF	SCALE	APP'D
DESIGNED DT	DETAILED MCS	QUANTITIES DT CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH CADD CK.

Plotted By: dluedke Plot Location:
File: N0576OIBBR-13_Camber.dgn
Plot Date: 9/13/2013

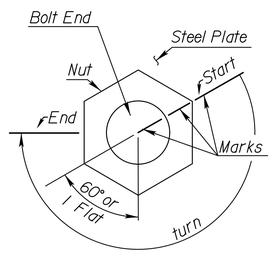
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	26	99



TYPICAL BOLT AND
DIRECT TENSION INDICATOR (DTI)

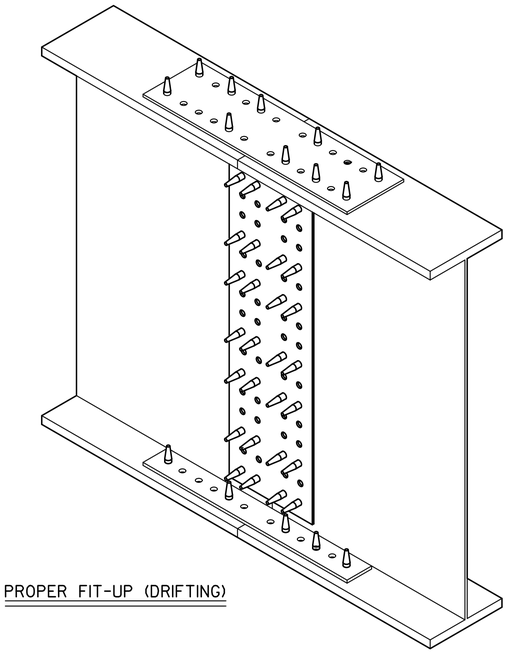


TYPICAL DIRECT TENSION INDICATOR (DTI)



REQUIRED MARKING DETAIL

(shows calibrated turn = 3½ flats from snug tight condition). Example only, calibrated turn may be more or less than shown.



PROPER FIT-UP (DRIFTING)

Production Bolt Tightening

1. Install bolts and tighten to "snug tight" in a pattern, starting at the center of the splice and working toward the edge. On large girders this may have to be done twice, as the center bolts will become loose as plates are "Ironed out". This step is important because typically, any variation in results during production bolting is the result of a change in the materials, lubricant or equipment used to take the bolts to a "snug tight" condition during the calibration process.

2. Mark all of the bolts, nuts and the plate as shown in the marking detail. Mark the socket with a start and stop point. The stop point corresponds to the target rotation determined earlier.

3. Align the start mark on the socket with the line on the plate. While the bolt is being backed up, turn the nut until the stop mark on the socket lines up with the start mark on the plate.

4. Repeat with all bolts of the same length in the splice.

Acceptance and Rejection of Bolts

1. The Engineer will check all bolts with a feeler gage.

2. All nuts must be turned at least the target rotation beyond "snug tight".

3. All DTI's must have at least 3 refusals of the 0.005" gage.

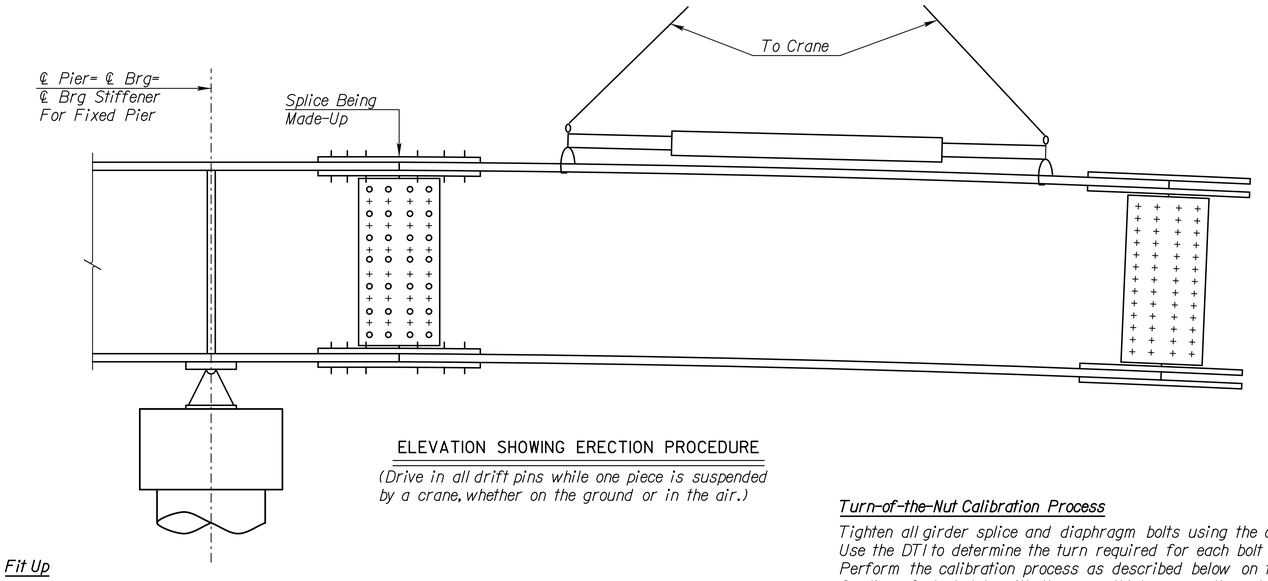
4. If all gaps refuse the 0.005" gage, and the nut, plate and bolt are not marked, reject the bolt.

5. If all gaps refuse the 0.005" gage, and the turned element has not been rotated more than 45° beyond the calibrated turn, accept the bolt.

6. If all gaps refuse the 0.005" gage, and the turned element has been rotated more than 45° beyond the calibrated turn, reject the bolt.

For additional information see the structural steel section of the Bridge Construction Manual.

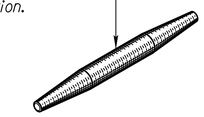
Suggested Impact wrench models:
CP 611
IR 2940
Cleco WS2110
ATP 1011/1040
Norbar PT1500



ELEVATION SHOWING ERECTION PROCEDURE

(Drive in all drift pins while one piece is suspended by a crane, whether on the ground or in the air.)

Drift Pin Diameter = Hole Diameter
Hole Ø = 15/16" for girder splice
Hole Ø = 13/16" for X-frame or diaphragm connection.



DRIFT PIN

Fit Up

During the fit up, install drift pins in all corner bolt holes, plus 25 percent of the bolt holes (as a min.), evenly distributed throughout the splice. Fill at least 25 percent of the bolt holes with high strength bolts. Fully tighten these bolts by the calibrated turn-of-the-nut method before removing any drift pins or moving the members. These bolts may be either erection bolts or production bolts. Erection bolts are used during fit up, to compress the plies of the splice to achieve a snug condition. Erection bolts are the property of the Contractor and do not remain in the bridge permanently. Erection bolts must be A325, and can be reused. Erection bolts are required when the abutting plates are of different thickness and no fill plate is provided. This situation usually results in a slight bending of the splice plates. If erection bolts are not used, the DTI's may fully compress before the plates are in firm contact. This would be cause for rejecting the splice. Clearly mark the erection bolts so that they are not left in the splice.

Erection

Two independent crews will survey the bearing seat elevations. The Engineer will verify that the results of those surveys show that the bearing seat elevations are within ± ¼ inch of the plan elevations before erection begins. Use the blocking diagram, as shown on the shop drawings, when erecting the beams/girders on the ground. Do not lift the assembled pieces into position until at least 25 percent of the holes are filled with fully tightened bolts. Locate the centerline of the bearing stiffener with the centerline of bearing device. Secure the beams/girders to the top of the pier cap prior to placement of the bearing device anchor bolts.

Turn-of-the-Nut Calibration Process

Tighten all girder splice and diaphragm bolts using the calibrated turn-of-the-nut method. Use the DTI to determine the turn required for each bolt diameter & length. Perform the calibration process as described below on the actual beam splice or using 3 plies of steel plate with the same thickness as the actual splice.

1. Bring at least 25 percent of the bolts in the splice to a "snug-tight-condition". "Snug tight condition" is defined as (with all plies in firm contact) "the full effort of a man on a spud wrench". Usually a smaller impact gun (½" drive) is used to snug the splice and a larger impact gun (1" drive) is used for final tightening. This is preferred over the use of a spud wrench. Production bolting and calibration must use the same tools and lubricating procedures. If an impact wrench is used to "iron the plates" and snug the bolts for calibration, then an impact wrench must be used during the snugging process during production bolting.

2. See "Required Marking Detail" (choose a bolt at the center of the splice and recheck snug on adjacent bolts)
- *a. Mark the outside of the socket at one of the corners.*
- *b. Mark the bolt, plate, and nut at a corner with a start line.*
- *c. Align the mark on the socket with the start mark on the bolt end.*
- *d. While holding a backup wrench on the head of the bolt, turn the nut ½ turn (3 flats).*
- *e. Record the number of refusals.*
- *f. If all of the gaps refuse, go to another bolt and turn the nut 2 flats (1/3 turn).*
- *g. If there are fewer than 3 refusals turn the nut an additional ¼ of a flat (15 degrees).*
- *h. Repeat step g, turning the nut 1/3 of a flat or less each time, until all of the gaps refuse the feeler gage. Record the amount required to cause all of the gaps to refuse the feeler gage. This is the target rotation.*

3. Repeat this process for each bolt diameter and length.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

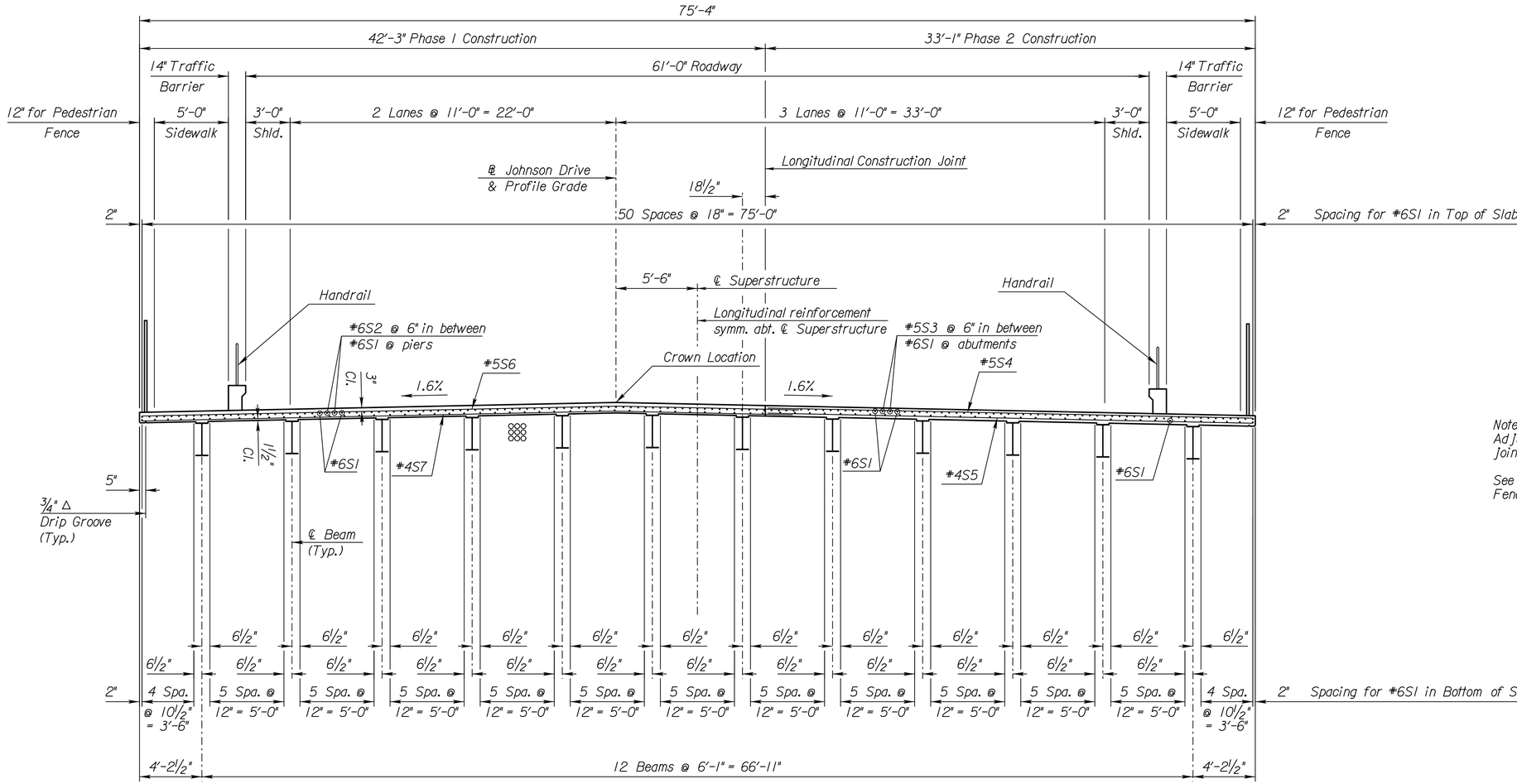
KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.5I9.5 STA. 53+38.14
STEEL ERECTION, FIT UP & BOLTING PROCEDURE
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

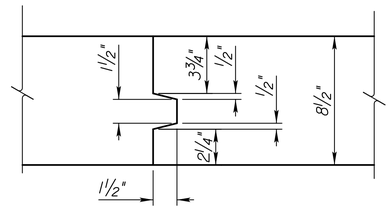
SHEET NO.	OF	SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

Plotted By: oluedke | Plot Location:
File: N0576OIBBR-14_ Steel Erection, fit up, bolting procedure.dgn
Plot Date: 9/13/2013

CADconform Certify This File

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	27	99





CONSTRUCTION JOINT DETAIL

Note: Use 2'-1" min. lap for #5S4 & #5S6.
Use 1'-8" min. lap for #4S5 & #4S7.

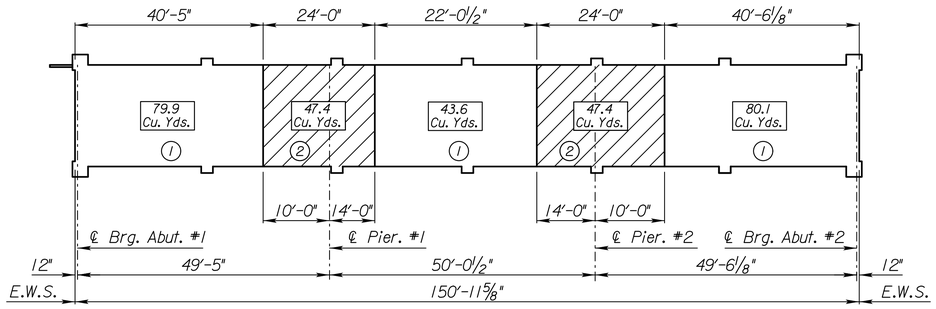
Note:
Adjust bar spacing as necessary @ longitudinal joint locations.

See architectural sheets for details of Pedestrian Fence and Handrail.

Half Section Near Pier

Half Section Near Abutment

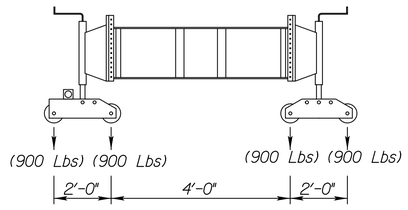
SLAB SECTION



CONCRETE PLACING SEQUENCE
Segmental, combined or continuous pours are allowed by an approved alternate placing sequence. Any discontinuous pour must stop at a construction joint short of a pier.

The Contractor may place the barriers continuously from one end of the bridge to the other.

CONCRETE PLACING SEQUENCE



ASSUMED FINISHING MACHINE VALUES LOADING DIAGRAM

Note: Provide Temporary Bracing at the top and bottom flanges of the exterior beams during concrete placement of the deck. The temporary bracing and labor for installation is subsidiary to the bid item "Concrete (Grade 4.0) (AE) (SA). Details for proposed temporary bracing shall be submitted with the falsework plans

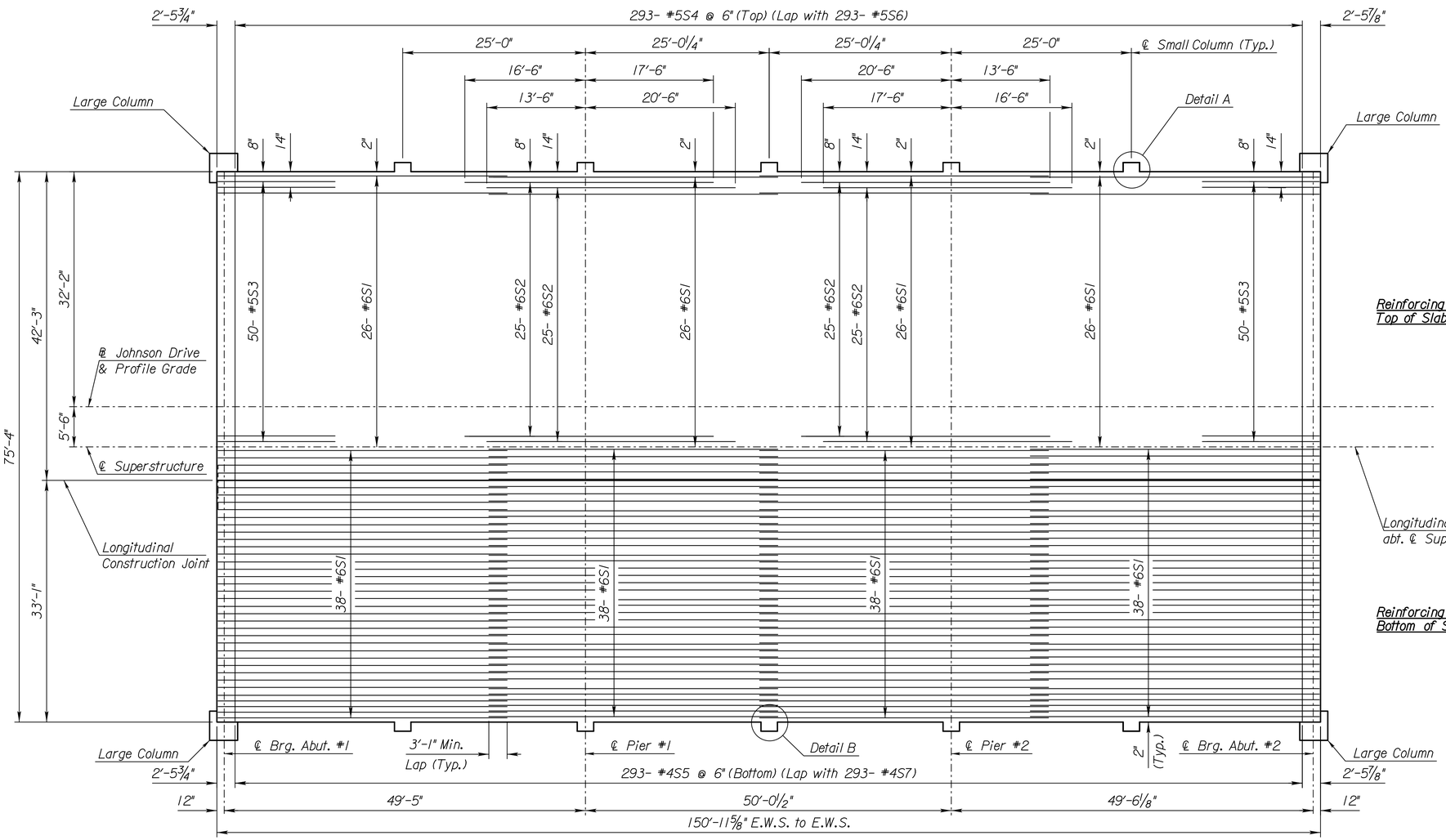
NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.519.5 STA. 53+38.14
SLAB SECTION
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

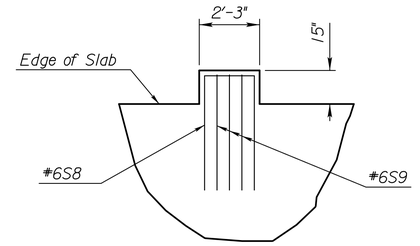
SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

Plotted By: oluedke | Plot Location:
File: N0576101BBR-15_Slab_Section.dgn
Plot Date: 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	28	99

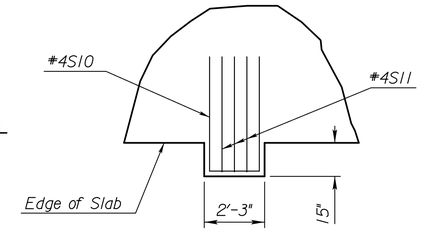


SLAB PLAN



DETAIL A

(Alternate #6S9 with typical slab reinf.)
(Typical slab reinf. not shown for clarity.)



DETAIL B

(Alternate #4S11 with typical slab reinf.)
(Typical slab reinf. not shown for clarity.)

Reinforcing in Top of Slab

Longitudinal reinforcement symm. abt. ℄ Superstructure

Reinforcing in Bottom of Slab

Plot Location:
Plotted By: dluedke
File: N0576018BR-16_Slab_Plan.dgn
Plot Date: 9/13/2013

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

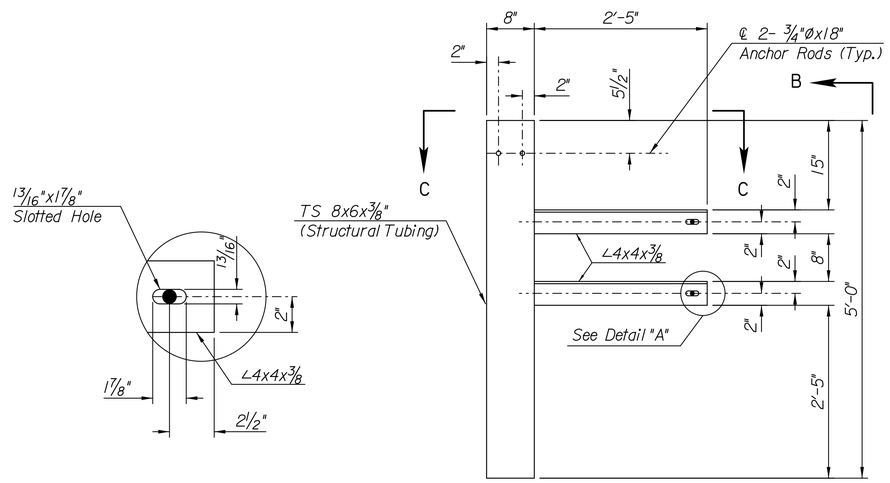
Br. No. 51860046T.519.5 STA. 53+38.14

SLAB PLAN
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

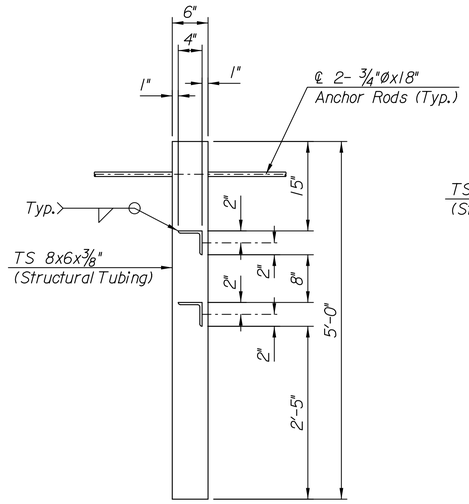
SHEET NO. OF	SCALE	APP'D	
DESIGNED DT	DETAILED MCS	QUANTITIES DT	CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH	CADD CK.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	29	99

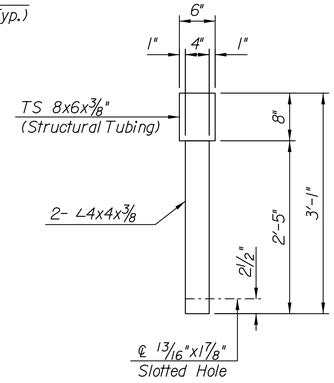


DETAIL A

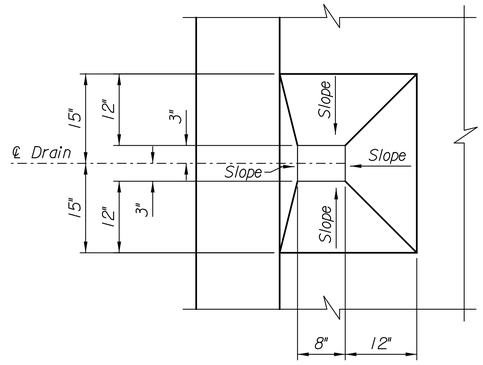
DRAIN TUBE



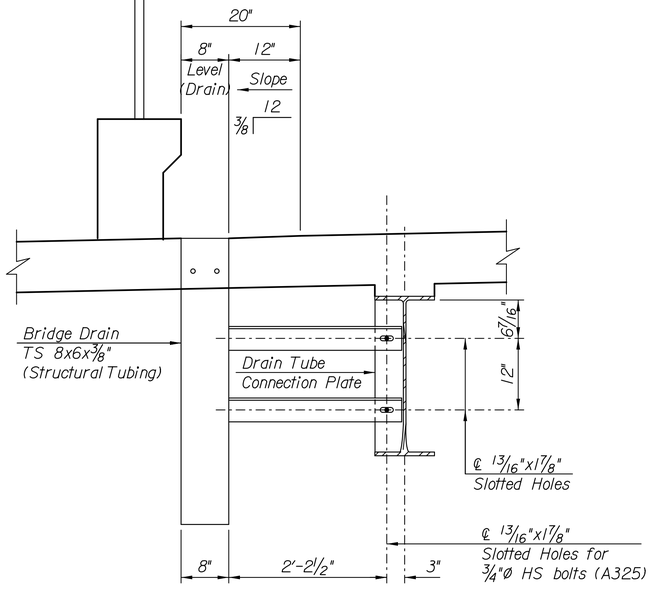
SECTION B-B



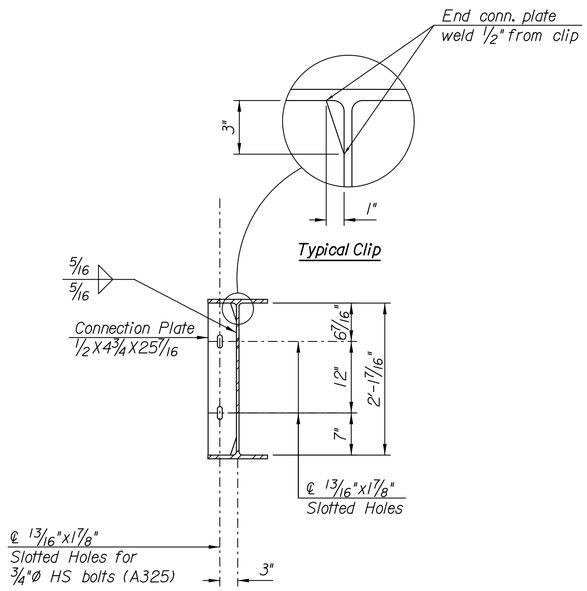
SECTION C-C



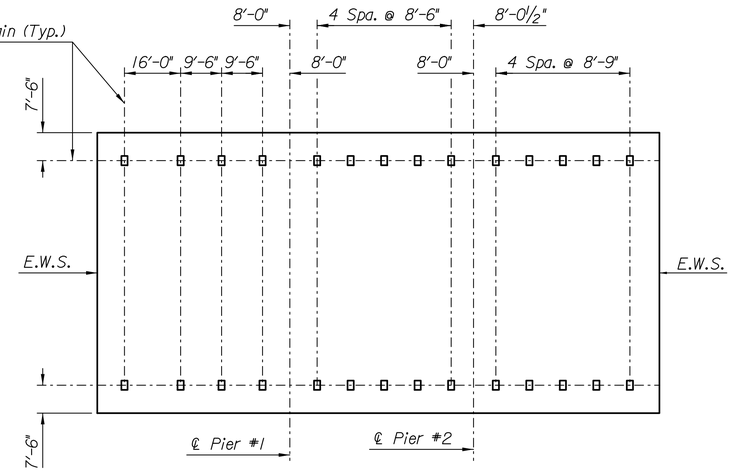
PLAN



ELEVATION OF DRAIN



DRAIN TUBE MOUNTING CONNECTION PLATE



PLAN SHOWING DRAIN SPACING

BRIDGE DRAINS: This work shall consist of providing bridge drains and drain tubes. Drain tubes shall be ASTM A500 Gr. B steel. Mounting angles and anchor bolts shall be ASTM A709 Gr. 36 and bolts shall be A325 HS bolts with heavy hex nuts. All material shall be galvanized and painted with Inorganic Zinc-Vinyl Paint System. See KDOT Specifications. Material and labor required to install bridge drains is subsidiary to "Bridge Drainage System".

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

BRIDGE DRAIN DETAILS

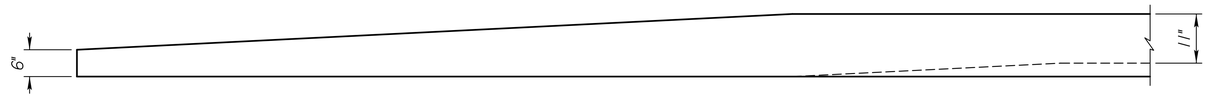
Johnson Drive Over Turkey Creek

Proj. No. 046 N-0576-01 Johnson Co.

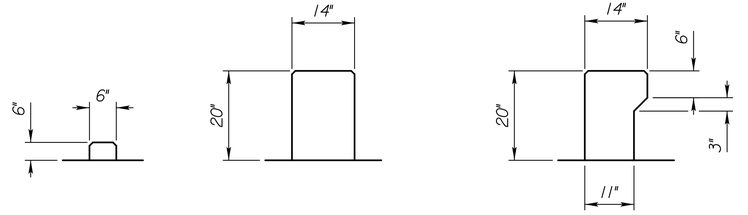
SHEET NO.	OF	SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

Plot Location:
Plotted By: dluedke
File: N0576OIBBR-17_Drain.dgn
Plot Date: 9/13/2013

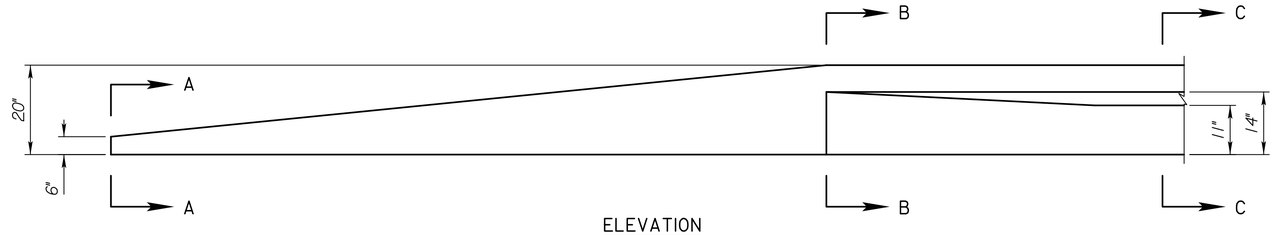
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	30	99



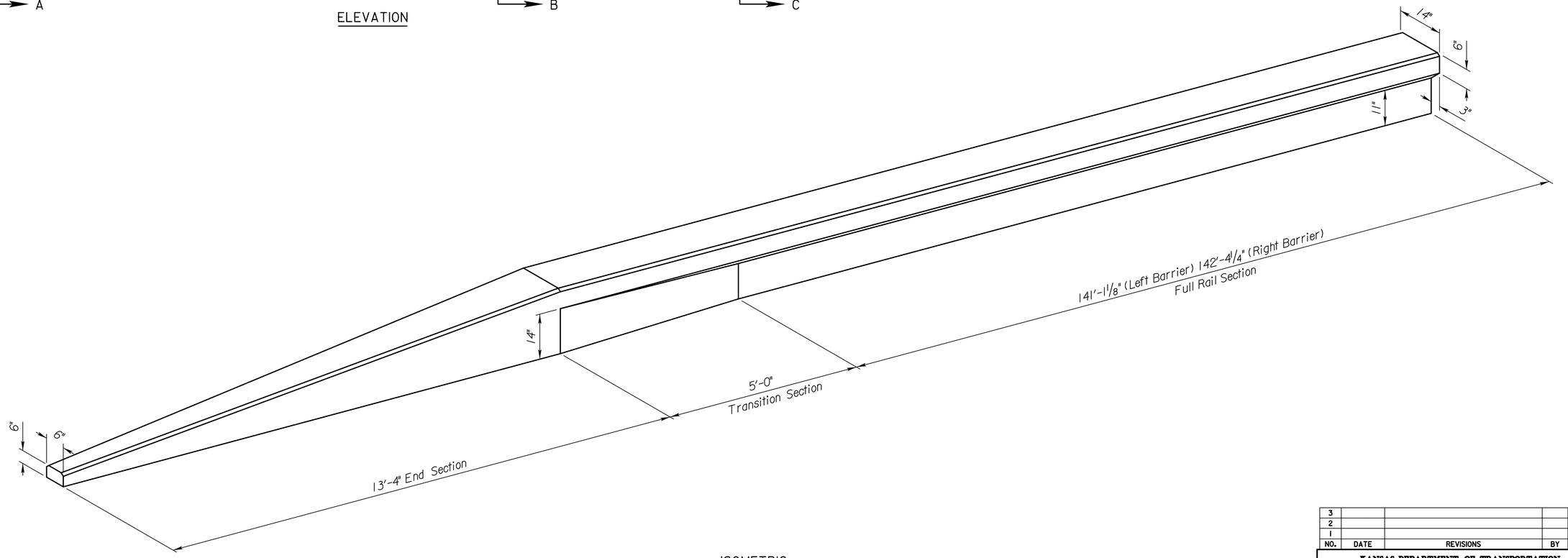
PLAN



SECTION A-A SECTION B-B SECTION C-C



ELEVATION



ISOMETRIC

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

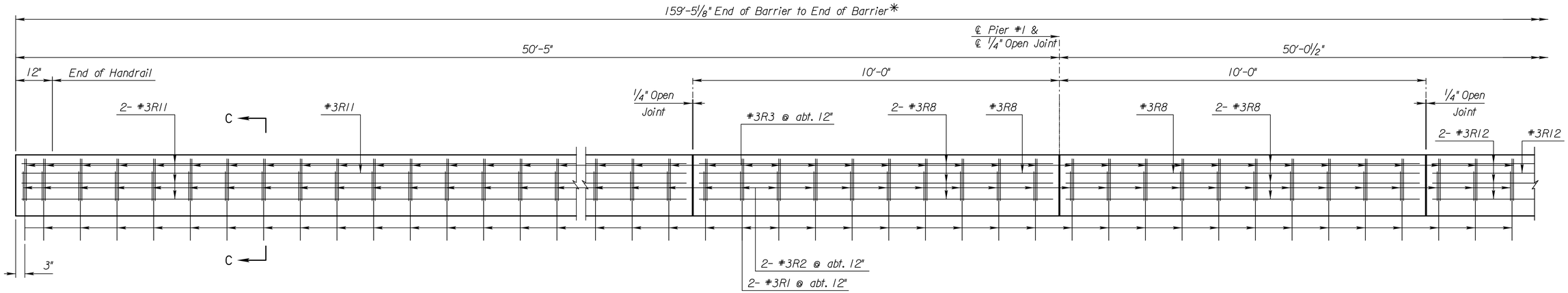
BARRIER DETAILS
Johnson Drive Over Turkey Creek

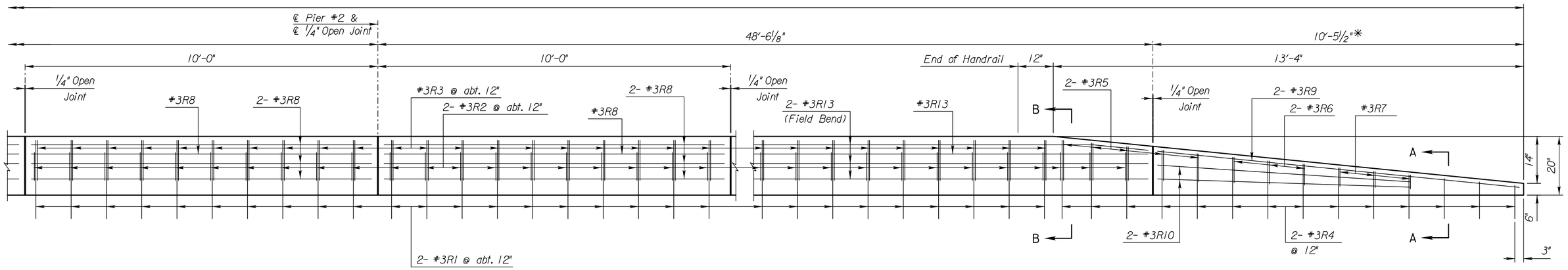
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF	SCALE	APP'D
DESIGNED DT	DETAILED MCS	QUANTITIES DT CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH CADD CK.

Plotted By: dluedke Plot Location:
File: N0576OIBBR-18_Barrier_Details.dgn
Plot Date: 9/13/2013

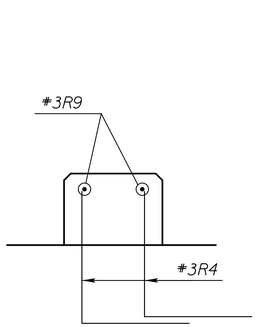
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	31	99



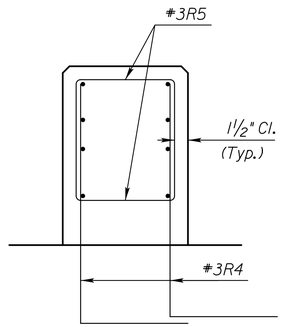


ELEVATION OF LEFT BARRIER CURB

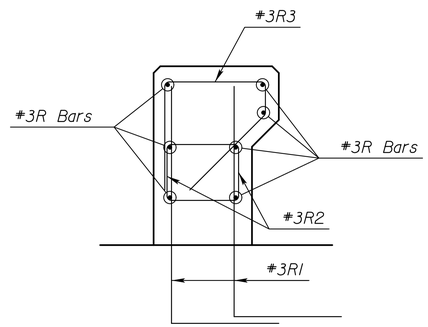
(Looking at Traffic Face)



SECTION A-A



SECTION B-B



SECTION C-C

Note:
Handrail not shown for clarity, see "Handrail and Barrier Details" sheets for Handrail and Barrier Aesthetic treatments.

KCMMB Concrete for the barrier shall use the slag option. Add integral color to the concrete as shown on the "Handrail and Barrier Details" sheets. Submit a sample of colored concrete to the owner for approval.

Aesthetic concrete treatments to Barrier are subsidiary to "Concrete (Grade 4.0) (AE) (SA)".

* *Measured along traffic face*

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

NORTH BARRIER ELEVATION

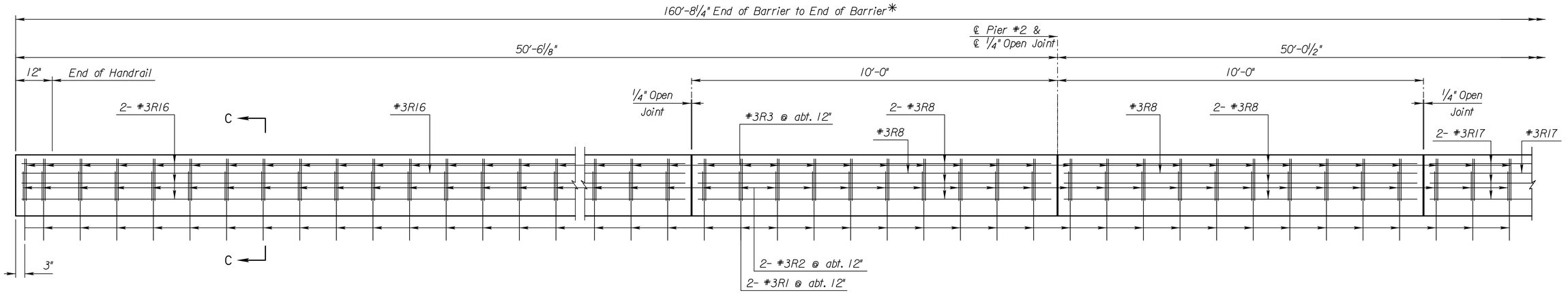
Johnson Drive Over Turkey Creek

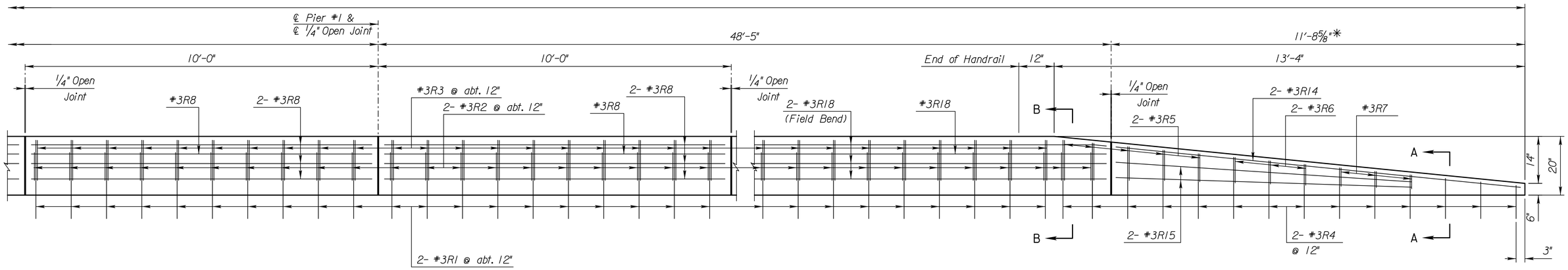
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D	
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT CADD
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH CADD CK.

Plotted By: dluedke | Plot Location:
File: N0576101BBR-19_Left_Barrier.dgn
Plot Date: 9/13/2013

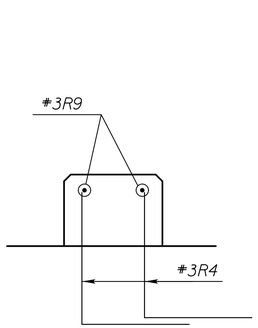
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	32	99



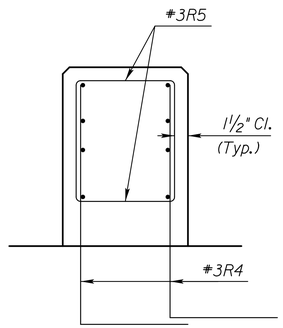


ELEVATION OF RIGHT BARRIER CURB

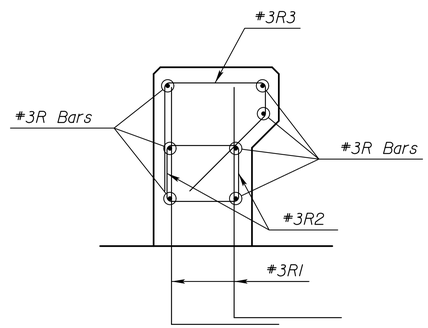
(Looking at Traffic Face)



SECTION A-A



SECTION B-B



SECTION C-C

Note:
Handrail not shown for clarity, see "Handrail and Barrier Details" sheets for Handrail and Barrier Aesthetic treatments.

KCMMB Concrete for the barrier shall use the slag option. Add integral color to the concrete as shown on the "Handrail and Barrier Details" sheets. Submit a sample of colored concrete to the owner for approval.

Aesthetic concrete treatments to Barrier are subsidiary to "Concrete (Grade 4.0) (AE) (SA)".

* Measured along traffic face

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

SOUTH BARRIER ELEVATION

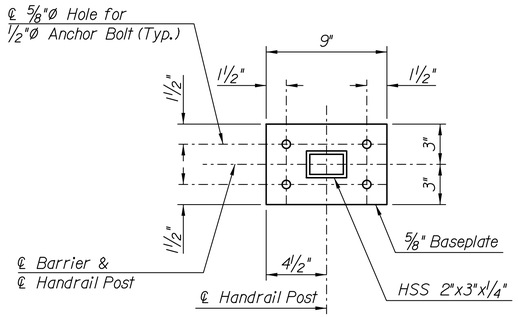
Johnson Drive Over Turkey Creek

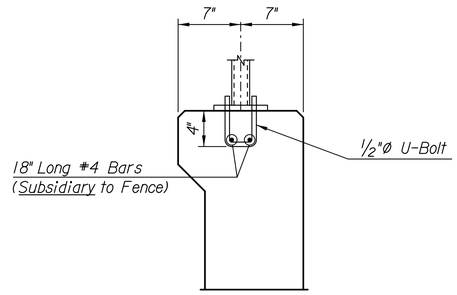
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF	SCALE	APP'D	
DESIGNED DT	DETAILED MCS	QUANTITIES DT	CADD
DESIGN CK. DWH	DETAIL CK. DWH	QUAN. CK. DWH	CADD CK.

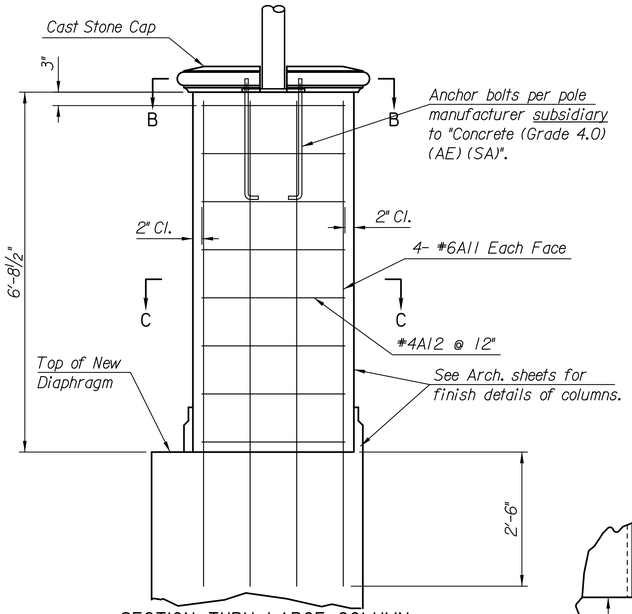
Plotted By: dluedke | Plot Location:
File: N0576018BR-20_Right Barrier.dgn
Plot Date: 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	33	99

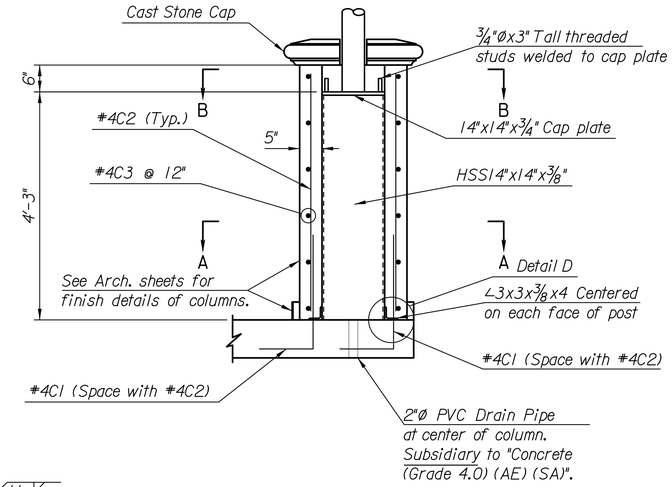




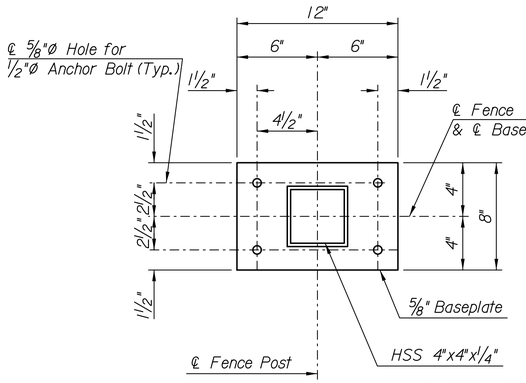
BASEPLATE DETAILS FOR HANDRAIL ON BARRIER

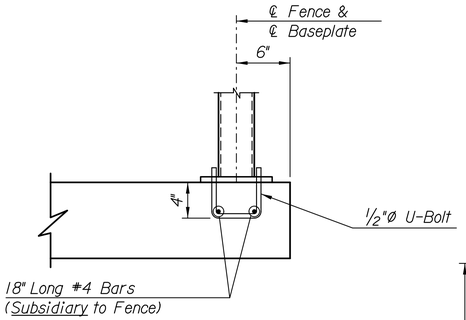


SECTION THRU LARGE COLUMN ON ABUTMENT

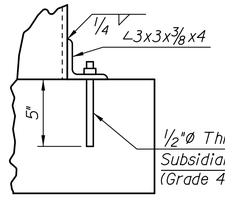


SECTION THRU SMALL COLUMN ON BRIDGE

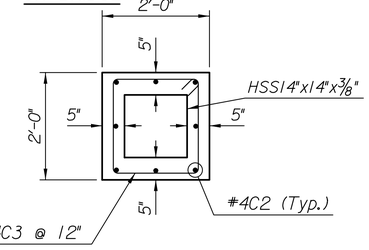


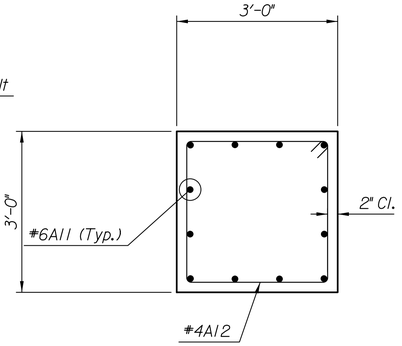


BASEPLATE DETAILS FOR TYPICAL FENCEPOST ON BRIDGE AND RETAINING WALL



DETAIL D



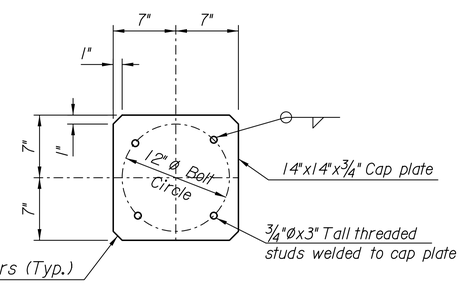


SECTION C-C

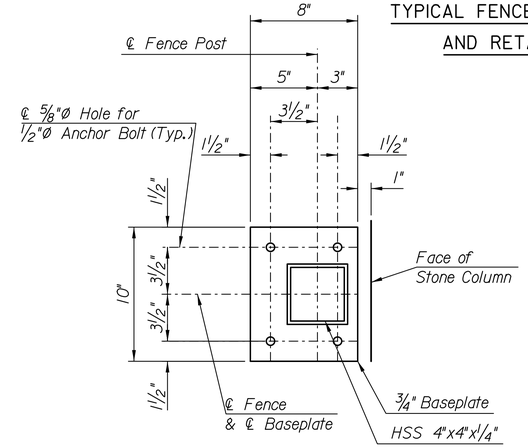
COLUMN QUANTITIES (For Information Only)		
	Concrete (Grade 4.0) (AE) (SA)	Reinforcing Steel (Gr. 60) (Epoxy Coated)
Large Column	9.0 yd.^{3}	1,250 lbs.
Small Column	4.6 yd.^{3}	1,490 lbs.

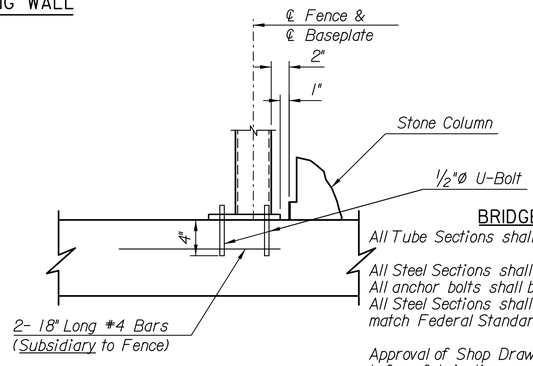
Note:
Concrete and reinforcing steel required for the large and small columns is included in the superstructure quantities. All materials & labor required to install HSS14x14x3/8 and cap plate as shown on the plans shall be subsidiary to "Concrete (Grade 4.0) (AE) (SA)".

SECTION A-A



SECTION B-B





BASEPLATE DETAILS FOR FENCE END POST ON BRIDGE

Note:
See architectural sheets for details of fence, handrail and columns. All materials & labor required to construct the aesthetic details on the columns, including cast stone caps, formliner, stain, tiles, cast stone bases & cast stone bands shall be subsidiary to "Concrete (Grade 4.0) (AE) (SA)".

BRIDGE HANDRAIL, PEDESTRIAN FENCE & CONCRETE COLUMN GENERAL NOTES:

All Tube Sections shall be ASTM A500 Gr. B.

All Steel Sections shall be Hot-Dip Galvanized per ASTM A123. All anchor bolts shall be Galvanized per ASTM A153. All Steel Sections shall be Powder Coated to match Federal Standard 595B, Color 27038.

Approval of Shop Drawings and Materials is required before fabrication or installation begins.

Provide Galvanized Shims to erect the Fence Posts vertically. The cost of Shims shall be subsidiary.

All Labor and Materials required to Construct and Install the Fence on the bridge & on the retaining walls is considered completely covered by the bid item "Fence (Pedestrian)".

All Labor and Materials required to Construct and Install the Handrail on the bridge is considered completely covered by the bid item "Bridge Handrail (Metal) (1'-10")".

The assemblies shall be shop welded and ground smooth prior to painting. No field welding will be permitted.

The Contractor shall field verify the as-built dimensions, grade and profile of the bridge prior to the fabrication of the Pedestrian Fence and Handrail. The rails of the Pedestrian Fence and Handrail shall be parallel to the roadway profile. All posts shall be plumb.

All misc. steel and plates shall be ASTM A36.

All welding shall conform to the latest edition of AWS D1.1 and shall use E70XX electrodes.

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

KANSAS DEPARTMENT OF TRANSPORTATION

Br. No. 51860046T.519.5 STA. 53+38.14

RAILING & AESTHETIC COLUMN DETAILS

Johnson Drive Over Turkey Creek

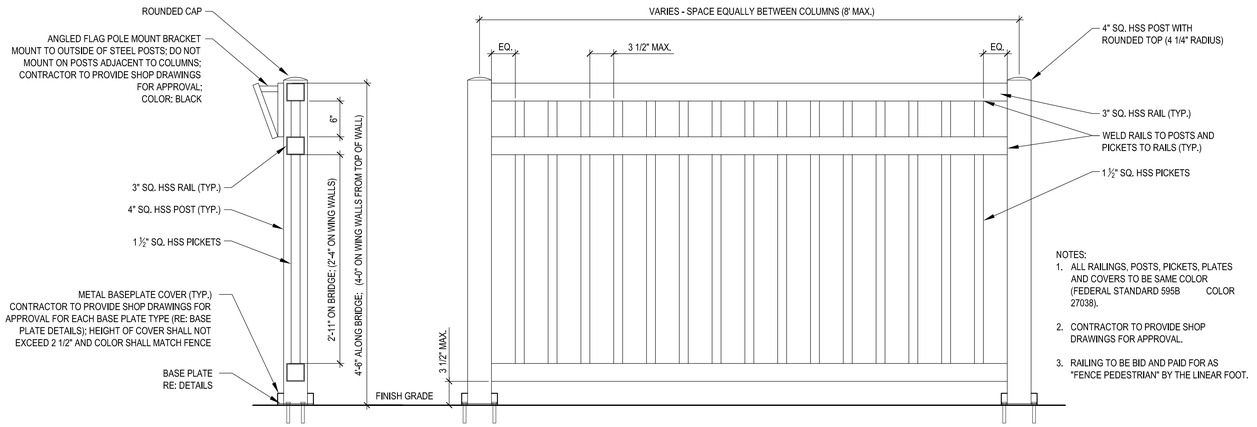
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D	
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH
				CADD	
				CADD CK.	

Plotted By: dluedke | Plot Location:
File: N0576018BR-18_Railing & Aesthetic Column Details.dgn
Plot Date: 9/13/2013

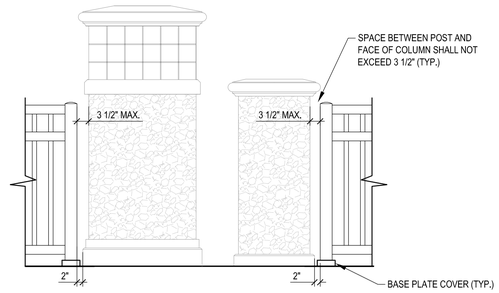
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	34	99

CONTRACTOR SHALL PROVIDE SHOP DRAWINGS FOR APPROVAL



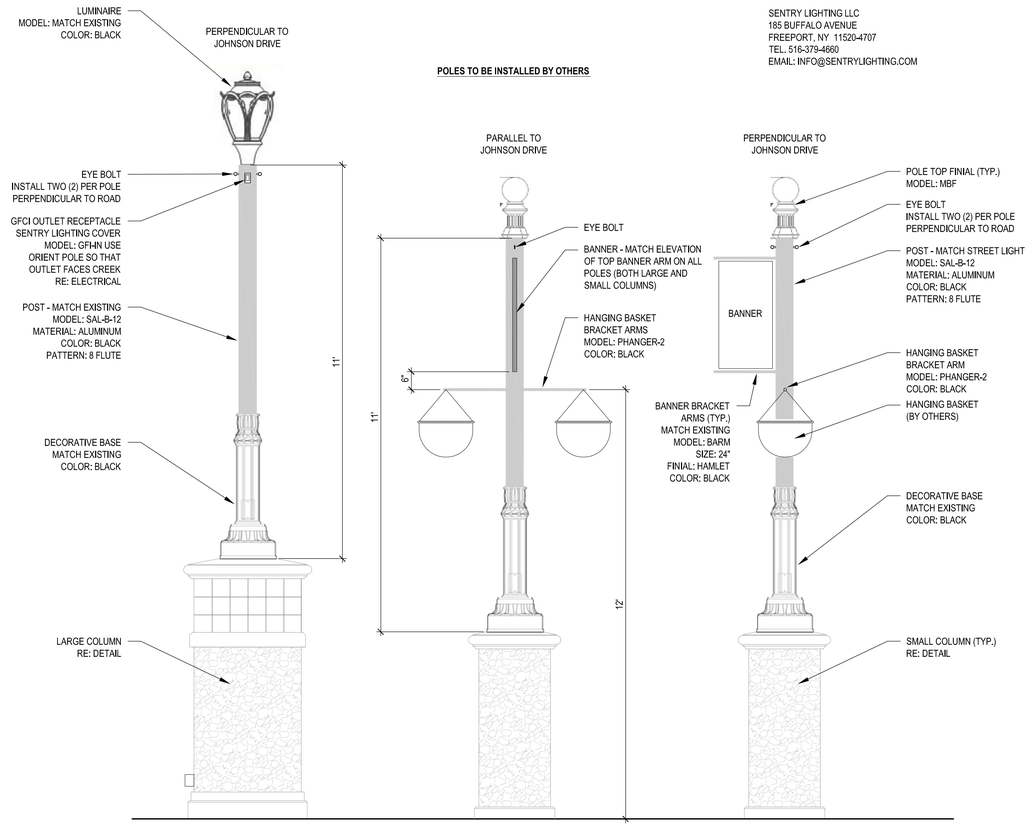
NOTES:
1. ALL RAILINGS, POSTS, PICKETS, PLATES AND COVERS TO BE SAME COLOR (FEDERAL STANDARD 595B COLOR 27038).
2. CONTRACTOR TO PROVIDE SHOP DRAWINGS FOR APPROVAL.
3. RAILING TO BE BID AND PAID FOR AS "FENCE PEDESTRIAN" BY THE LINEAR FOOT.

RAILING DETAIL 01
N.T.S.



NOTE:
1. GAP BETWEEN POST AND COLUMN SHALL BE CONSISTENT AT ALL POST/COLUMN LOCATIONS.

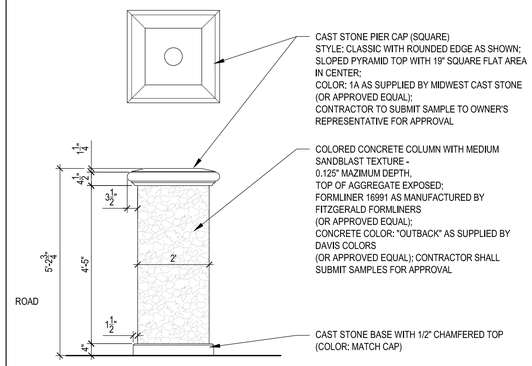
RAILING AT COLUMNS 02
N.T.S.



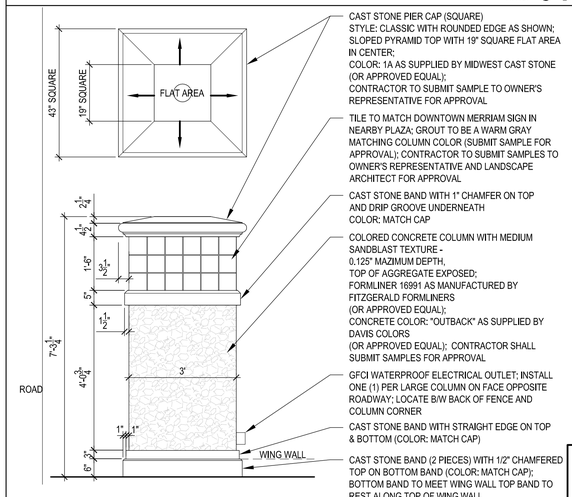
NOTE:
1. ALL LIGHT POLES, BRACKETS, ARMS, FINIALS, LUMINAIRES, ETC. TO BE AS SUPPLIED BY THE FOLLOWING MANUFACTURER:

SENTRY LIGHTING LLC
185 BUFFALO AVENUE
FREEPORT, NY 11520-4707
TEL. 516-379-4660
EMAIL: INFO@SENTRYLIGHTING.COM

LIGHT POLES 03
N.T.S.



SMALL COLUMN 04
N.T.S.



LARGE COLUMN 05
N.T.S.

KANSAS DEPARTMENT OF TRANSPORTATION

COLUMN & FENCE DETAILS

DATE	BY

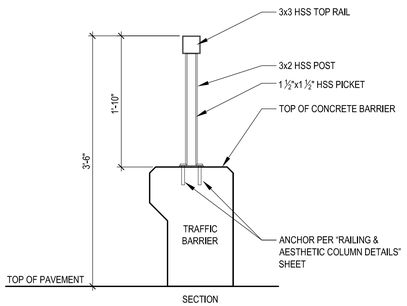
REFERENCES NOTED
REFERENCES CHECKED

Drawn By: cbuedke
Plotted: September 12, 2013
File It: \P-12121 Johnson Drive Bridge Improvements\04_Dwg\CD\Bridge Aesthetics_DETAILS.dwg

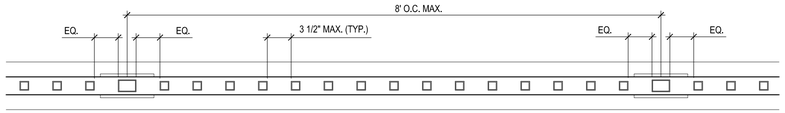
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	35	99

CONTRACTOR SHALL PROVIDE SHOP DRAWINGS FOR APPROVAL

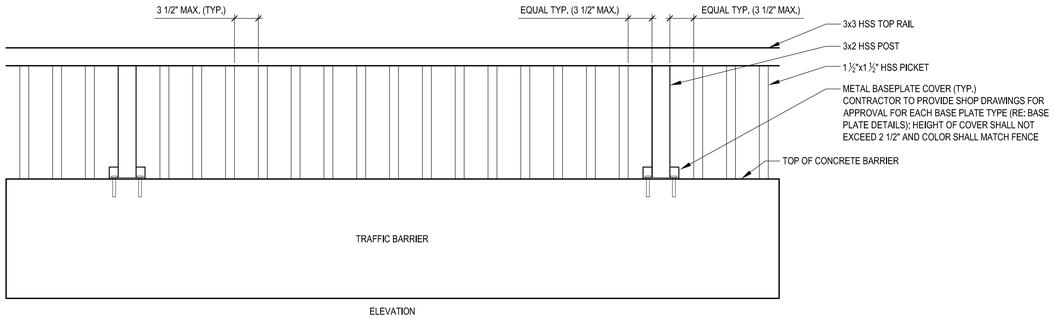
NOTE:
1. ALL RAILINGS, POSTS, PICKETS, PLATES AND COVERS TO BE SAME COLOR (FEDERAL STANDARD 595B COLOR 27038).
2. CONTRACTOR TO PROVIDE SHOP DRAWINGS FOR APPROVAL.
3. RAILING TO BE BID AND PAID FOR AS "BRIDGE HANDRAIL (METAL) (1'-10")" BY THE LINEAR FOOT.



SECTION

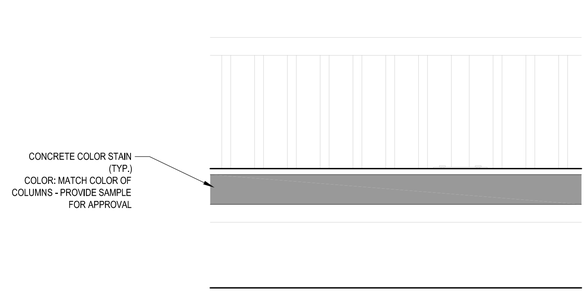


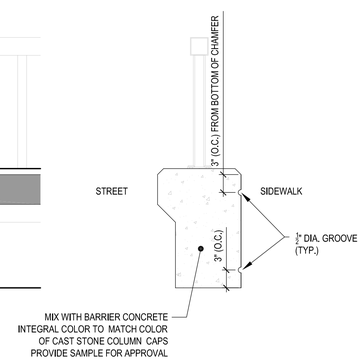
PLAN

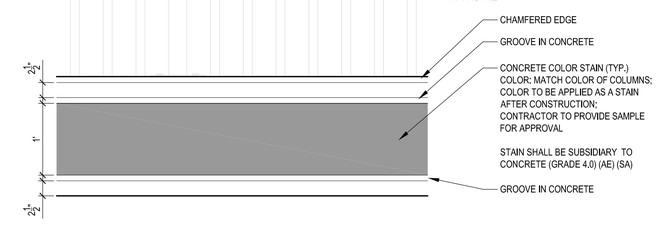


ELEVATION

NOTE:
1. CONCRETE COLOR STAIN SHALL BE A WATER BASED ACRYLIC POLYMER EMULSION PRODUCT WITH UV STABLE PIGMENT DISPERSION WHICH WILL PROMOTE ADHESION TO CONCRETE SURFACES. SUBMIT PRODUCT DATA TO OWNER AND/OR OWNER'S REPRESENTATIVE FOR APPROVAL.
2. CONTRACTOR TO PROVIDE SHOP DRAWINGS FOR APPROVAL.







	BY	DATE
REFERENCES NOTED		
REFERENCES CHECKED		

Drawn By: tuedke Plotted: September 12, 2013
File H: \P-12121 Johnson Drive Bridge Improvements\04_Dwg\CD\35 HANDRAIL & BARRIER DETAILS.dwg

KANSAS DEPARTMENT OF TRANSPORTATION

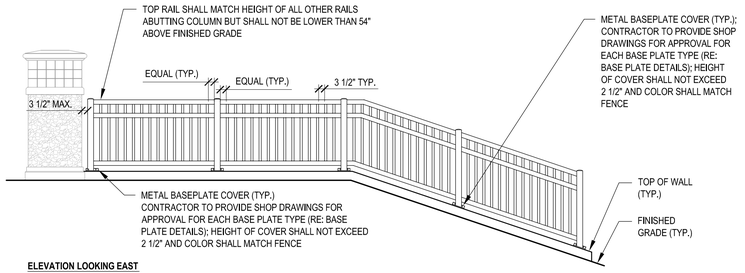
HANDRAIL & BARRIER DETAILS

BARRIER AESTHETIC TREATMENTS N.T.S. **01**

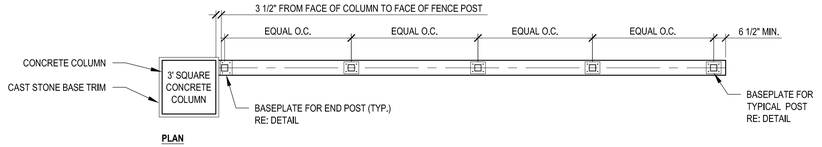
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	36	99

CONTRACTOR SHALL
PROVIDE SHOP DRAWINGS
FOR APPROVAL

NOTES:
1. CONTRACTOR TO FIELD VERIFY AFTER WALL IS IN PLACE.
2. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS TO OWNER AND/OR OWNER'S REPRESENTATIVE FOR APPROVAL.
3. DISTANCE BETWEEN FENCE POSTS SHALL NOT EXCEED 8'-6".
4. NO SPACE BETWEEN COLUMNS, POSTS OR SPINDLES SHALL ALLOW A 4" SPHERE TO PASS.



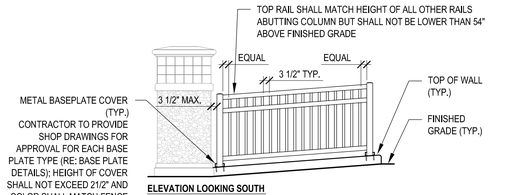
ELEVATION LOOKING EAST



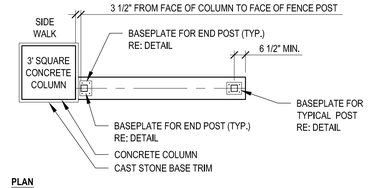
PLAN

SW WING WALL FENCE N.T.S. 01

NOTES:
1. CONTRACTOR TO FIELD VERIFY AFTER WALL IS IN PLACE.
2. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS TO OWNER AND/OR OWNER'S REPRESENTATIVE FOR APPROVAL.
3. DISTANCE BETWEEN POSTS SHALL NOT EXCEED 8'-6".
4. NO SPACE BETWEEN COLUMNS, POSTS OR SPINDLES SHALL ALLOW A 4" SPHERE TO PASS.



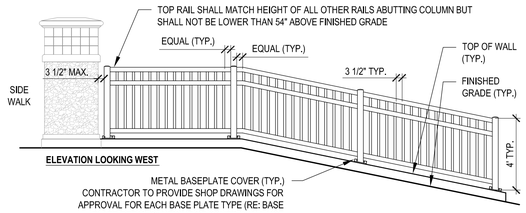
ELEVATION LOOKING SOUTH



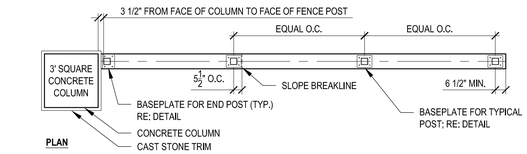
PLAN

NW WING WALL FENCE N.T.S. 02

NOTES:
1. CONTRACTOR TO FIELD VERIFY AFTER WALL IS IN PLACE.
2. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS TO OWNER AND/OR OWNER'S REPRESENTATIVE FOR APPROVAL.
3. DISTANCE BETWEEN POSTS SHALL NOT EXCEED 8'-6".
4. NO SPACE BETWEEN COLUMNS, POSTS OR SPINDLES SHALL ALLOW A 4" SPHERE TO PASS.

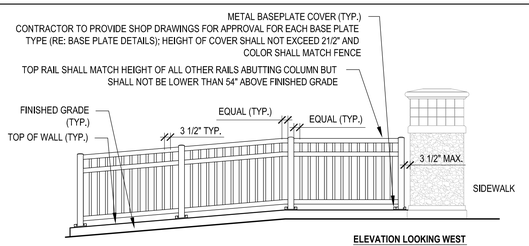


ELEVATION LOOKING WEST



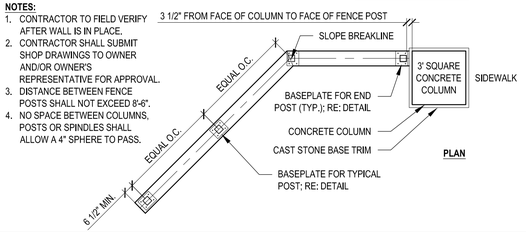
PLAN

NE WING WALL FENCE N.T.S. 03



ELEVATION LOOKING WEST

NOTES:
1. CONTRACTOR TO FIELD VERIFY AFTER WALL IS IN PLACE.
2. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS TO OWNER AND/OR OWNER'S REPRESENTATIVE FOR APPROVAL.
3. DISTANCE BETWEEN FENCE POSTS SHALL NOT EXCEED 8'-6".
4. NO SPACE BETWEEN COLUMNS, POSTS OR SPINDLES SHALL ALLOW A 4" SPHERE TO PASS.



PLAN

SE WING WALL FENCE N.T.S. 04

KANSAS DEPARTMENT OF TRANSPORTATION

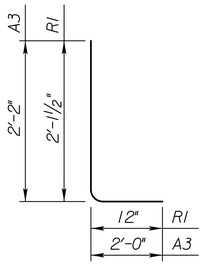
FENCE ON RETAINING WALL
DETAILS

BY	DATE
REFERENCES NOTED	
REFERENCES CHECKED	

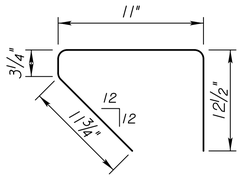
RETAINING WALL DETAILS.dwg

Plotted: September 12, 2013
Drawn By: cbuedke
File H:\P-12121 Johnson Drive Bridge Improvements\04_Dwg\CD\36 FENCE ON RETAIN

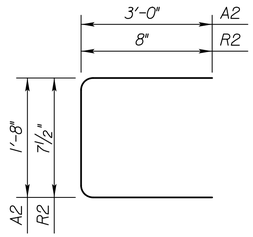
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	37	99



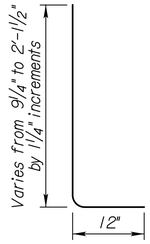
R1 & A3



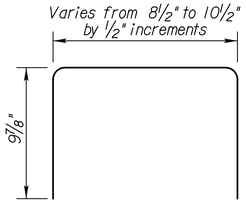
R3



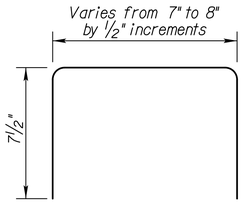
R2 & A2



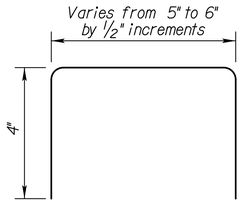
R4
(Cut 4 each Length)



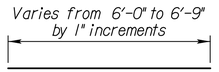
R5
(Cut 4 each Length)



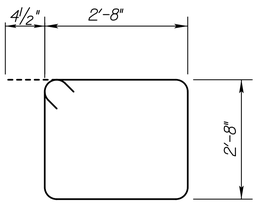
R6
(Cut 4 each Length)



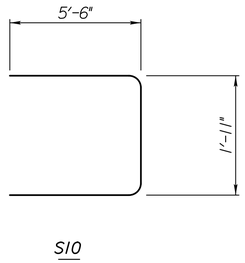
R7
(Cut 4 each Length)



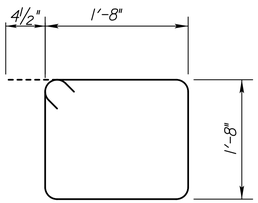
RW2
(Cut 2 each Length)



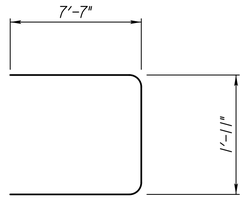
A12



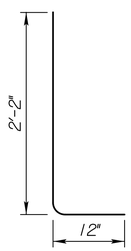
S10



C3



S8



C1

BILL OF REINFORCING STEEL
Epoxy Coated - Grade 60

Abutment - Deck - Rail Superstructure

Straight Bars				Bent Bars			
Mark	Size	Number	Length	Mark	Size	Number	Length
A6	#6	8	48'-1"	S8	#6	10	17'-1"
A7	#6	8	35'-1"				
A9	#6	40	5'-10"	A2	#5	178	7'-8"
A10	#6	8	3'-0"	A3	#5	178	4'-2"
A11	#6	48	9'-1"				
RW3	#6	11	11'-10"	A12	#4	28	11'-5"
S1	#6	508	40'-0"	C1	#4	80	3'-2"
S2	#6	200	34'-0"	C3	#4	60	7'-5"
S9	#6	30	7'-7"	S10	#4	10	12'-11"
A1	#5	348	1'-9"	R1	#3	594	3'-1 1/2"
A4	#5	16	47'-6"	R2	#3	604	1'-11 1/2"
A5	#5	16	35'-1"	R3	#3	297	3'-2 1/2"
A8	#5	24	1'-3"	R4	#3	112	2'-5 3/8"
S3	#5	200	16'-0"	R5	#3	20	2'-5 1/4"
S4	#5	293	32'-9"	R6	#3	12	1'-10 1/2"
S6	#5	293	44'-4"	R7	#3	12	1'-1 1/2"
C2	#4	80	4'-5"				
RW1	#4	8	11'-10"				
RW2	#4	20	6'-4 1/2"				
S5	#4	293	32'-9"				
S7	#4	293	43'-11"				
S11	#4	30	5'-6"				
R8	#3	56	9'-8"				
R9	#3	2	10'-2"				
R10	#3	4	7'-0"				
R11	#3	7	40'-1"				
R12	#3	7	29'-8"				
R13	#3	7	38'-2"				
R14	#3	2	11'-4"				
R15	#3	4	8'-0"				
R16	#3	7	40'-2"				
R17	#3	7	29'-8"				
R18	#3	7	38'-1"				

⊗ See Bending Diagram

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

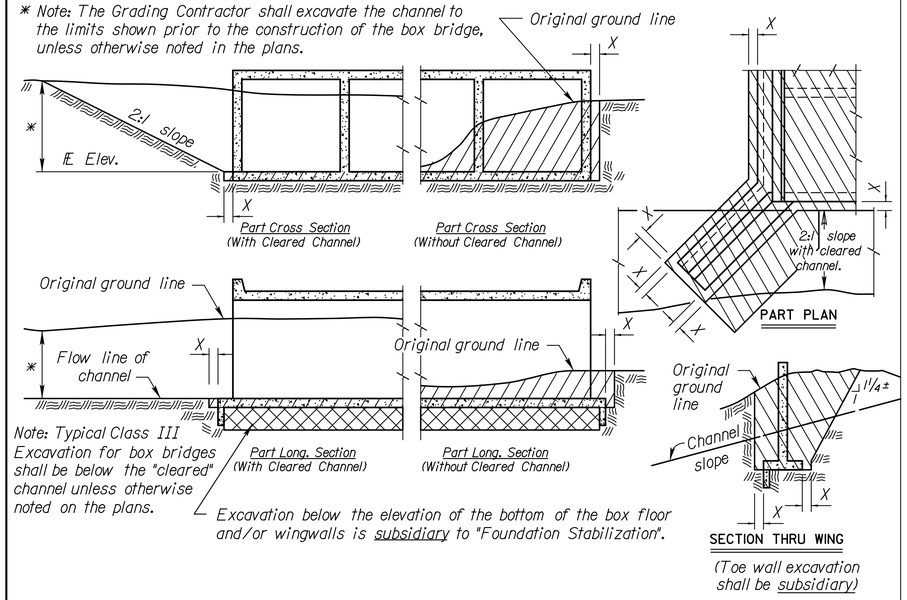
KANSAS DEPARTMENT OF TRANSPORTATION
Br. No. 51860046T.519.5 STA. 53+38.14
BILL OF REINFORCING
STEEL & BAR BENDING DIAGRAMS
Johnson Drive Over Turkey Creek
Proj. No. 046 N-0576-01 Johnson Co.

SHEET NO. OF		SCALE		APP'D			
DESIGNED	DT	DETAILED	MCS	QUANTITIES	DT	CADD	
DESIGN CK.	DWH	DETAIL CK.	DWH	QUAN. CK.	DWH	CADD CK.	

Plotted By: dluedke | Plot Location:
File: N057601BBR-22_barbill.dgn
Plot Date: 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	38	99

* Note: The Grading Contractor shall excavate the channel to the limits shown prior to the construction of the box bridge, unless otherwise noted in the plans.

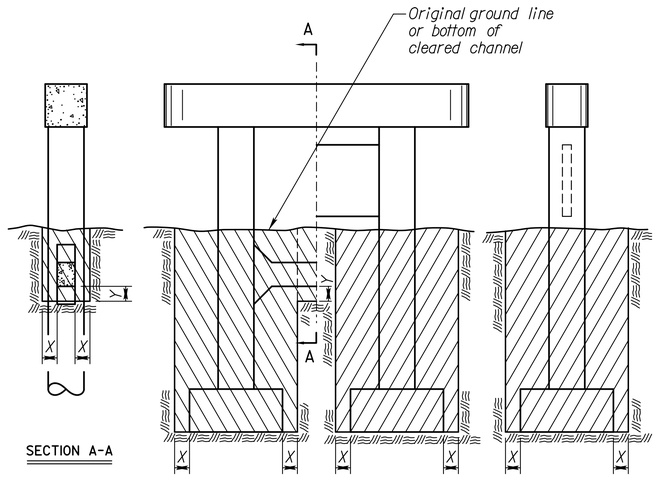


Note: Typical Class III Excavation for box bridges shall be below the "cleared" channel unless otherwise noted on the plans.

Excavation below the elevation of the bottom of the box floor and/or wingwalls is subsidiary to "Foundation Stabilization".

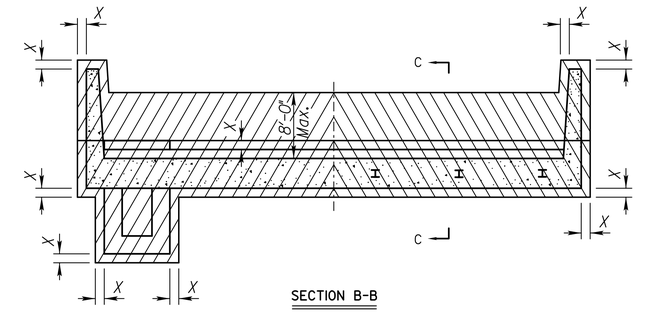
EXCAVATION DETAILS FOR REINFORCED CONCRETE BOX CULVERT

Note: Excavation for culverts less than bridge length and the additional excavation for "Embedded Structures" shall not be paid for as Class III Excavation, but shall be subsidiary to Grade 4.0 Concrete.



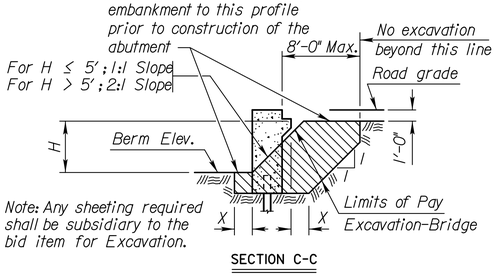
EXCAVATION DETAILS FOR TYPICAL PIERS

See detail when rock or shale (rock) is encountered. ⊛



Note: Bridge Contractor shall finish the embankment and berms after the construction of the abutment and dispose of any excess material as approved by the Engineer.

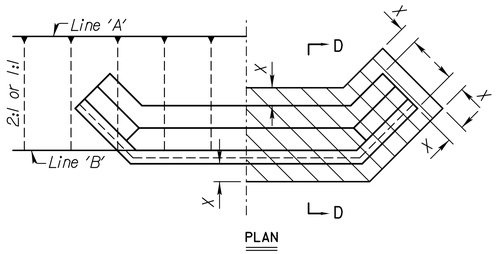
Grading Contractor to complete embankment to this profile prior to construction of the abutment

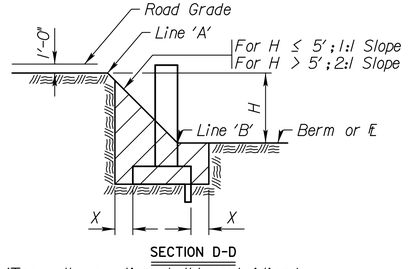


Note: Any sheeting required shall be subsidiary to the bid item for Excavation.

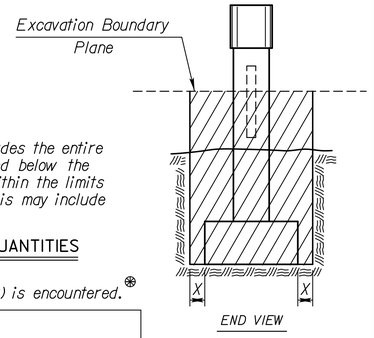
EXCAVATION DETAILS FOR TYPICAL ABUTMENTS

See detail when rock or shale (rock) is encountered. ⊛





EXCAVATION DETAILS FOR ABUTMENTS WITH FLARED WINGWALLS



Note: Class II Excavation includes the entire volume of whatever nature found below the "Excavation Boundary Plane", within the limits specified for measurement. This may include water or air.

CLASS II EXCAVATION QUANTITIES

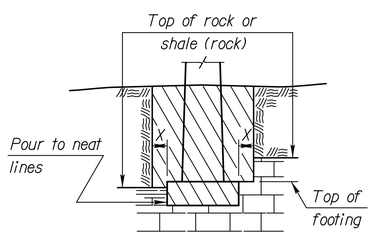
See detail when rock or shale (rock) is encountered. ⊛

Note: All bridge excavation shall be computed on the basis of the cross-hatch areas and boundary lines indicated on this sheet and the Excavation Boundary Plane on the Construction Layout.

Sides of trenches in hard or compacted soil including embankments shall be shored, sheeted, braced or otherwise supported when the trench is more than 5 feet in depth and 8 feet or more in length. In lieu of the shoring, the sides of the trench above the 5 foot level may be sloped to preclude collapse. The slope for average soils shall be 1:1. If the angle of repose of the soil is less, flatter slopes shall be required.

Dimension "X" shall be 2'-0" unless indicated otherwise on the general plans.

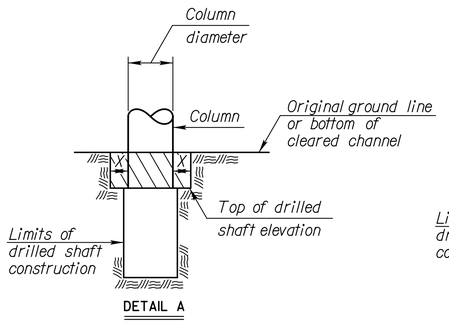
Dimension "Y" shall be 1'-6" unless indicated otherwise on the general plans.

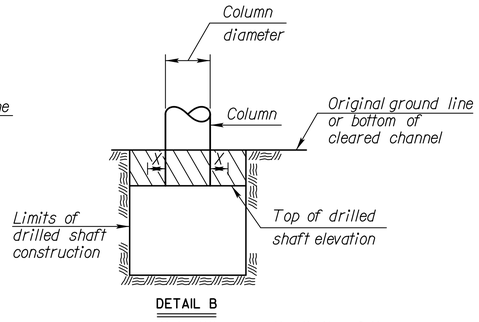


⊛ EXCAVATION DETAIL FOR FOOTINGS IN ROCK OR SHALE (ROCK)

(Piers and Abutments)

Note: Excavation below top of rock, hard shale or below top of footing, whichever is lower, shall be to neat lines of the concrete construction.





DRILLED SHAFT DETAILS

Note: Whenever the limits of the drilled shaft construction are greater than the Column Diameter + 2X, the limits of Class I, II or III Excavation shall be the limits of the drilled shaft construction. (See Detail B)

NO.	DATE	REVISIONS	BY	APP'D
7				
6	8-15-12	Embeddement Excavation Subsidiary	JPJ	TLF
5	5-15-12	Revised Wing Excavation	JPJ	TLF
4	3-3-10	Revised Wing Excavation	JPJ	TLF
3	10-16-06	Revised "Foundation Stab." Note	JPJ	KFH
2	10-19-04	Concrete - Class to Grade	RAM	KFH
1	4-10-02	Added "Foundation Stab." Note	RAM	KFH

KANSAS DEPARTMENT OF TRANSPORTATION

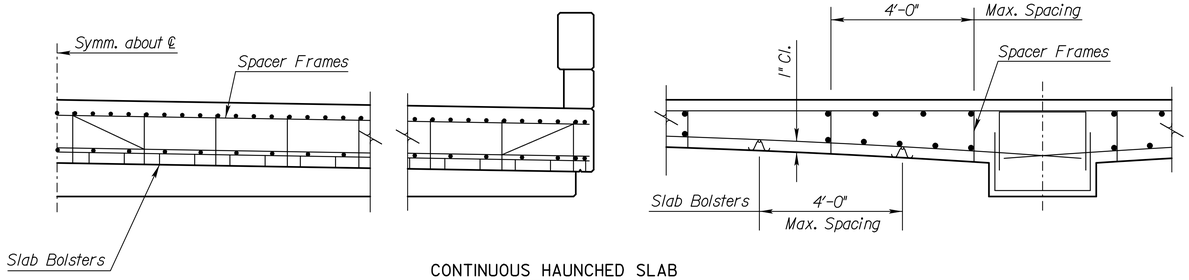
BRIDGE EXCAVATION (LRFD)

BRIOOB

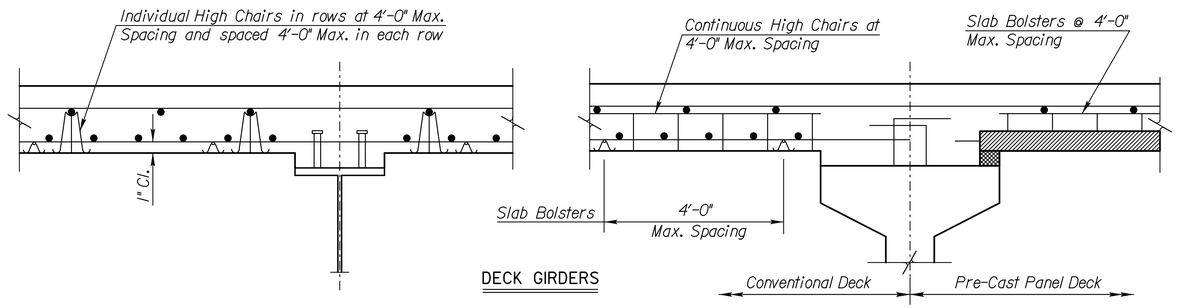
FHWA APPROVAL			4/17/10	APP'D	TERRY L. FLECK
DESIGNED	DETAILED	RDR	QUANTITIES	CADD	
DESIGN CK.	DETAIL CK.	LRR	QUAN. CK.	CADD CK.	

Std. Base File: br100.dgn
Plotted By: dluedke
File: br100b.dgn
Plot Date: 9/13/2013
Plot Location:

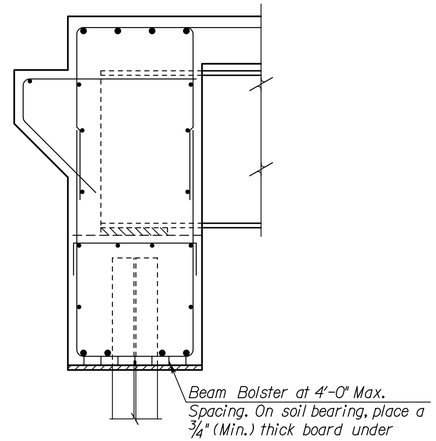
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS		2013	39	99



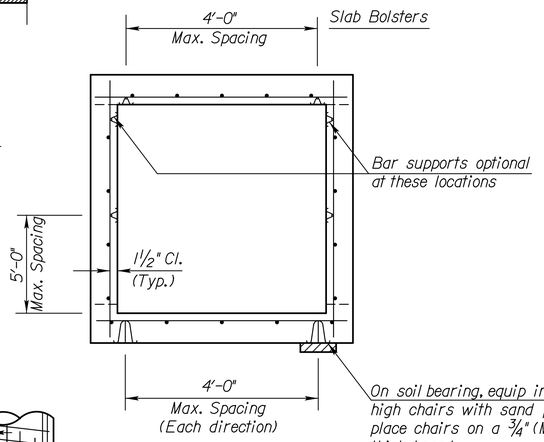
CONTINUOUS HAUNCHED SLAB



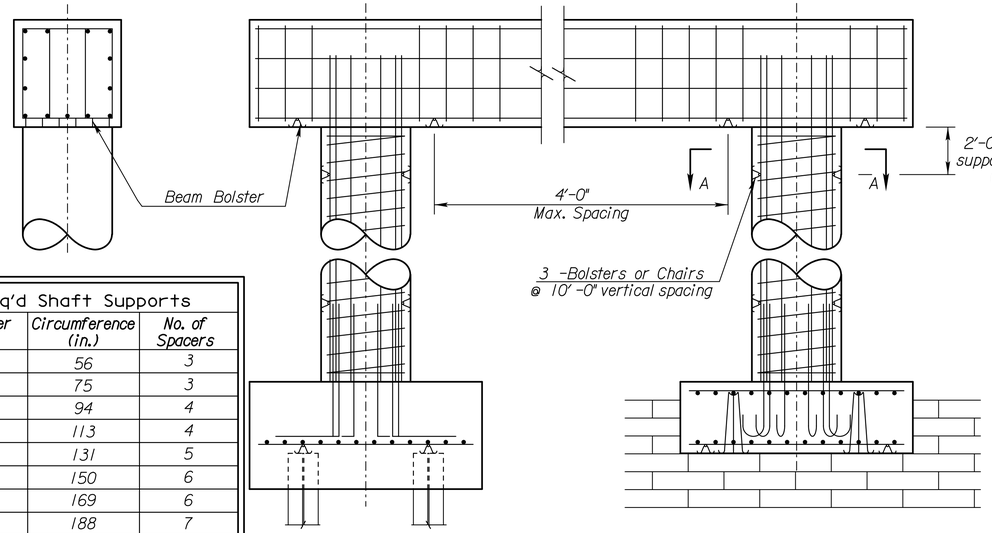
DECK GIRDERS



ABUTMENT

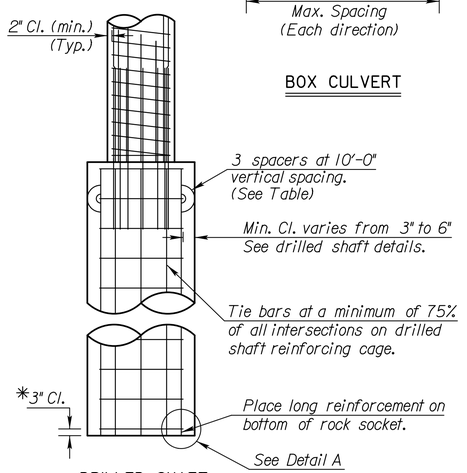


BOX CULVERT

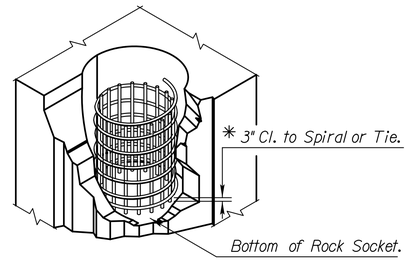


PIER

Req'd Shaft Supports		
Diameter (in.)	Circumference (in.)	No. of Spacers
18	56	3
24	75	3
30	94	4
36	113	4
42	131	5
48	150	6
54	169	6
60	188	7
66	207	7
72	226	8
78	244	9
84	263	9
90	282	10
96	301	11
102	320	11
108	339	12

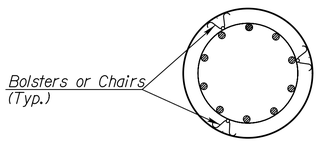


DRILLED SHAFT



DETAIL A

* Note: Longitudinal reinforcing steel is placed on the bottom of the rock socket. Maintain 3" clearance from the bottom of rock socket to the first spiral or tie bar.



SECTION A-A

TYPICAL SECTION

Reference is made to the latest edition of the CRSI "Manual of Standard Practice" for recommended industry practices concerning reinforcing steel.

Use only the following types of bar supports:

1) Wire Bar Supports:

a) Epoxy coated reinforcing: Class 1 Protection
b) Non-epoxy coated reinforcing: Class 1, 2, or 3 Protection

2) Plastic Bar Supports

3) Supplementary bars

When securing epoxy coated reinforcement, use tie wires or metal clips that are epoxy or plastic coated.

Do not weld reinforcing steel to bar supports or to other reinforcing steel. Shop weld spacer frames for haunched slabs.

Tie bars at all intersections around the perimeter of each mat and at not less than 2'-0" centers or at every intersection, whichever is greater.

Where more than one length of bar support is required, lap the end legs so they are locked or tied together.

Use proper height supports to maintain the distance between the reinforcing and the formed surface or the top surface of deck slabs within 1/4" of that indicated on the plans.

Spacings shown are maximums. Use sufficient supports, as determined by the Engineer, to retain the reinforcing steel in position.

Construct any platforms, required for the support of workers and/or equipment during concrete placement, directly on the forms and not on the reinforcing steel.

Designs and arrangements of Supports or Spacers other than as shown on this sheet, may be used with the permission of the Engineer.

NO.	DATE	REVISIONS	BY	APP'D
5	11-10-10	Column Bar Supports Req'd	JPJ	TLF
4	12-01-05	Drilled Shaft Spiral Steel Placement	JPJ	KFH
3	8-21-00	Added Pre-Cast Panel Detail	RAM	KFH
2	12-20-99	Added Haunched Slab Bolsters	RAM	KFH
1	12-09-99	Revised Drilled Shaft Clearance	RAM	KFH

KANSAS DEPARTMENT OF TRANSPORTATION

SUPPORTS AND SPACERS FOR REINFORCING STEEL

BRI20

FHWA APPROVAL		11-17-10	APP'D	Terry L. Fleck	
DESIGNED	RAM	DETAILED	RAA	QUANTITIES	CADD RAA
DESIGN CK.	LRR	DETAIL CK.	RAM	QUAN. CK.	CADD CK. RAM

Std. Base File: bri20.dgn
Plotted By: cluedke
File: bri20.dgn
Plot Date: 9/13/2013
Plot Location:

CADconform Certify This File

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	40	99

CURB AND GUTTER, COMBINED (AE)

LOCATION	STATION TO STATION		SIDE	(Lin. Ft.)		REMARKS
Johnson Drive	52+42.65	52+53.37	Rt.	25.7		SW Corner
Johnson Drive	52+46.73	52+52.66	Lt.	6.3		NW Corner
TOTAL				32.0		

RECAPITULATION OF BRIDGE QUANTITIES

BRIDGE NUMBER	STATION	SEE SHEET NO.
51860046T.519.5	53+38.14	13

RECAPITULATION OF ROAD QUANTITIES

ITEM	QUANTITY	UNIT
Contractor Construction Staking	1	Lump Sum
Field Office and Laboratory (Type A)	1	Each
Foundation Stabilization (Set Price)	1	Cu.Yd.
Mobilization	1	Lump Sum
Mobilization (DBE)	1	Lump Sum
Removal of Existing Structures	1	Lump Sum
Concrete for Seal Course (Set Price)	1	Cu.Yd.
Clearing and Grubbing	1	Lump Sum
Common Excavation	17	Cu.Yd.
Common Excavation (Contractor Furnished)	22	Cu.Yd.
Rock Excavation	99	Cu.Yd.
Compaction of Earthwork (Type A) (MR-5-5)	31	Cu.Yd.
Water (Grading)(Set Price)	1	M.Gal.
• Concrete (Grade 4.0)(AE)	35.9	Cu.Yd.
Reinforcing Steel (Grade 60)	4270	Lb.
• Curb and Gutter, Combined (AE)	32	Lin. Ft.
• Sidewalk Construction (4")(AE)	61	Sq. Yd.
• Sidwalk Ramp	3	Sq. Yd.
• Concrete Pavement (10" Uniform)(AE)	139	Sq. Yd.
Temporary Surfacing Material (Aggregate)(Set Price)	1	Cu.Yd.
Curing Environment	Lump Sum	Lump Sum

For Surfacing Quantities, See Sh. No. 72
For Temporary Project Water Pollution Control Quantities, See Sh. No. 41
For Seeding and Sodding Quantities, See Sh. No. 49
For Lighting Quantities, See Sh. No. 55
For Signing Quantities, See Sh. No. 63
For Pavement Marking Quantities, See Sh. No.71
For Traffic Control Quantities, See Sh. No. 86

•Concrete Shall be KCMMB-4K

REMOVAL OF EXISTING STRUCTURES (FOR INFORMATION ONLY)

LOCATION	STATION	SIDE	ITEM	QUANTITY	UNITS	REMARKS
Johnson Drive	53+38.14	℄	Bridge Superstructure	1	Each	
Johnson Drive	52+50	Rt.	Existing Guardrail	48	Lin. Ft.	

Note: The listing shown may not be complete. Payment for structures or obstructions not listed but whose removal is required by the construction, as determined by the Engineer, should not be paid for directly, but should be included in the bid item "Removal of Existing Structures".

SIDEWALK CONSTRUCTION (4")(AE)

STATION TO STATION		LOCATION	SIDE	AREA (SQ. YD.)	REMARKS
52+42.65	52+62.66	Johnson Drive	Rt.	23.8	SW Corner
52+46.73	52+62.66	Johnson Drive	Lt.	13.4	NW Corner
54+13.63	54+24.19	Johnson Drive	Lt.	10.0	NE Corner
54+13.63	54+25.17	Johnson Drive	Rt.	13.7	SE Corner
TOTAL				60.9	

SIDEWALK RAMP

STATION TO STATION		LOCATION	SIDE	AREA (SQ. YD.)	REMARKS
52+44.24	52+47.56	Johnson Drive	Rt.	3.3	SW Corner
TOTAL				3.3	

RETAINING WALL

LOCATION	STATION	SIDE	•CONCRETE (Grade 4.0) (AE) (Cu. Yd.)	REINFORCING STEEL (Gr. 60) (lb.)	REMARKS
Johnson Drive	52+65.16	Rt.	14.6	1710	Southwest Corner
Johnson Drive	54+11.13	Lt.	11.6	1350	Northeast Corner
Johnson Drive	54+11.13	Rt.	9.7	1210	Southeast Corner
TOTAL			35.9	4270	

•Concrete Shall be KCMMB-4K

CONCRETE PAVEMENT (10" UNIFORM) (AE) (BR APP)

STATION	LOCATION	QUANTITY (SQ. YDS.)	REMARKS
52+52.66	Johnson Drive	69.8	Abutment No. 1
54+23.63	Johnson Drive	69.6	Abutment No. 2
TOTAL		139.4	

EARTHWORK

STATION to STATION	EXCAVATION					COMPACTION			THROUGH CUTS NOT SUBGRADED			✳EMBANKMENT (CU.YDS.)		◭PLACE. SELECT SOIL CU.YDS.
	COMMON		ROCK		CONTR. FURN. CU.YDS.	TYPE AA MR-5-5 CU.YDS.	TYPE A MR-5-5 CU.YDS.		COMM. CU.YDS.	TYPE AA CU.YDS.		INITIAL CONSOL.	SETTLE-MENT	
	CU.YDS.	VMF	CU.YDS.	VMF										
52+52.66 - 54+23.63	17	0.80	99	1.00	22		31							
TOTALS	17		99		22		31							

1. Assumed VMF for Contractor Furnished Excavation is 0.80.
2. Rock Excavation is existing pavement, curb and gutter, sidewalk, and concrete slope protection to be wasted.

NO.	DATE	REVISIONS	BY	APP'D
2	1-14-08	Rem. Drainage Structure summary	S.W.K.	J.O.B.
1	1-9-91	Detailed on CADD	R.J.S	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

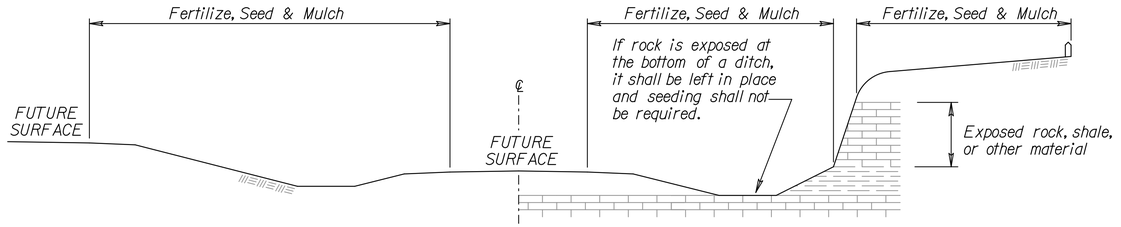
SUMMARY OF QUANTITIES

RD050

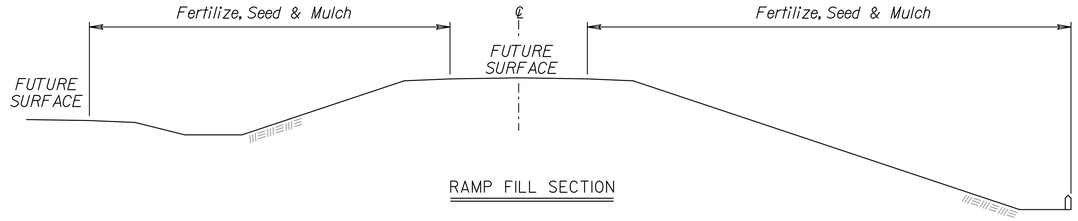
FHWA APPROVAL	5-28-08	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED B.N.B.
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. S.W.K.

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\Micro\Highways\Client Standards\rd050.dgn

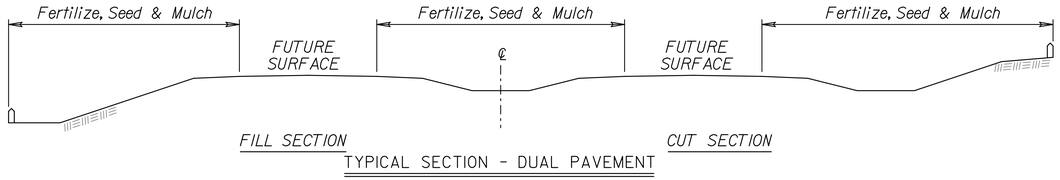
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	41	99



RAMP CUT SECTION



RAMP FILL SECTION



TYPICAL SECTION - DUAL PAVEMENT

FERTILIZER: A ratio and application rate that equals or exceeds the required minimum rate per acre of N, P_2O_5, K_2O listed in Summary of Quantities will be acceptable.

- *x - N = Nitrogen Rate of Application*
- *xx - P_2O_5 = Phosphorous Rate of Application*
- *xxx - K_2O = Potassium Rate of Application*

The Contractor will be required to finish areas of excavation, borrow and embankment in accordance with the specifications. Areas that require installation or construction of temporary water pollution control items will be finished in reasonable close conformity to the alignment, grade and cross section shown on the plans or as established by the Engineer.

CLT = Construction Limit Tract. This area is defined by the entire disturbed area of the project that requires seeding and erosion control measures to be placed. Any impervious areas (i.e. pavement, gravel, riprap, etc.) shall not be included in this measurement.

Slope = Defined by the area of the project that requires Class 1 erosion control material to be placed. This area shall be seeded using the Soil Erosion Mix prior to placement of the material. Broadcasting seed is preferred, however, drilling is acceptable.

Channel = Defined by the area of the project that requires Class 2 erosion control material to be placed. This area shall be seeded using the Soil Erosion Mix prior to placement of the material. Broadcasting seed is preferred, however, drilling is acceptable.

GENERAL NOTES

The entire disturbed area, excepting the paved or surfaced areas, steep rocky slopes and areas of undisturbed native sod or other desirable vegetation shall be fertilized (limed when required), seeded, and mulched. Soil preparation shall conform to the Standard Specifications.

Temporary seeding shall be done during any time of the year that the soil can be cultivated. After the temporary seeding has been completed on the entire project, a permanent seeding shall be done by another project during the normal seeding season.

MULCHING: Mulch shall be spread uniformly over all disturbed areas and punched in the soil, unless otherwise noted on the plans. The rate of application per acre, thickness in place, for the mulching materials is as follows:

1¾ - 2¼ Tons per Acre = 1½" loose depth spread uniformly over acre.

Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.
Other vegetative mulches acceptable only with the Engineer's concurrance.

The above rate is a guide. It will be at the discretion of the Engineer to determine what rate is sufficient for adequate protection of newly seeded areas.
The amount of mulch and mulch tacking slurry in the bid quantities is estimated. The total mulch and mulch tacking slurry required shall be determined in the field. The bid item for mulching and mulch tacking slurry shall be paid for by one of the following ways: A) Plan quantity as shown on Summary of Quantities, Seeding Sheet or Water Pollution Control Sheet, B) Slope measurement as measured in field, or C) Drill measurement less 5% as measured at the time of seeding.

SUMMARY OF SEEDING QUANTITIES: PROJECTS OF 1 ACRE OR MORE

P.L.S. RATE/ ACRE		ACRES		BID ITEM	QUANTITY	UNIT
CLT	SL/CH	CLT	SL/CH			
				Temporary Fertilizer (x - xx - xxx)		Lb
				Temporary Seed (Canada Wildrye Grass Seed)		Lb
				Temporary Seed (Sterile Wheatgrass)		Lb
				Soil Erosion Mix		Lb
				Agricultural Limestone		Tons
				Temporary Berm		LF
				Temporary Ditch Check (Rock) (Set Price)	1	Cu Yd
				Temporary Ditch Check		LF
				Temporary Inlet Sediment Barrier		Each
				Curb Inlet Protection		LF
				Temporary Sediment Basin		Cu Yd
				Temporary Slope Barrier (Set Price)	1	LF
				Temp Slope Barrier Biodegradable Log 9" Low Flow		LF
				Temp Slope Barrier Biodegradable Log 9" High Flow		LF
				Temp Slope Barrier Biodegradable Log 12" Low Flow		LF
				Temp Slope Barrier Biodegradable Log 12" High Flow		LF
				Temp Slope Barrier Biodegradable Log 20" Low Flow		LF
				Temp Slope Barrier Biodegradable Log 20" High Flow		LF
				Temporary Slope Drain		LF
				Temporary Stream Crossing		Each
				Sediment Removal (Set Price)	1	Cu Yd
				Mobilization, Emergency Erosion Control (Set Price)	1	Each
				Erosion Control (Class X, Type Y)		Sq Yd
				Erosion Control (Class X, Type Y)		Ac
				Mulching (Temporary)		Ac

SUMMARY OF SEEDING QUANTITIES: PROJECTS LESS THAN 1 ACRE

P.L.S. RATE/ ACRE		ACRES		BID ITEM	QUANTITY	UNIT
CLT	SL/CH	CLT	SL/CH			
150		0.2		*Temporary Fertilizer (15-30-15)*		Lb
20		0.2		Temporary Seed (Canada Wildrye Grass Seed)		Lb
45		0.2		Temporary Seed (Sterile Wheatgrass)		Lb
				Soil Erosion Mix		Lb
				Agricultural Limestone		Tons
				Temporary Seeding	*Lump Sum*	LS
				Temporary Ditch Check (Rock) (Set Price)	*1*	Cu Yd
				Temporary Ditch Check	*14*	LF
				Curb Inlet Protection	*390*	LF
				Temporary Slope Barrier (Set Price)	*1*	LF
				Sediment Removal (Set Price)	*1*	Cu Yd
				Mobilization, Emergency Erosion Control (Set Price)	*1*	Each
				Erosion Control (Class X, Type Y)		Sq Yd
				Mulch Tacking Slurry		Ac
				Mulching (Temporary)		Ac

NOTE: Projects less than 1 acre shall be bid as "Seeding" by the lump sum. All disturbed areas shall be seeded, fertilized and mulched at the listed rate per acre. The acres are estimated.

Regreen and Quick Guard are the approved sterile wheatgrass products.

SOIL EROSION MIX

PLS RATE	NAME	QTY (lb)
	Total (lb)	

NO.	DATE	REVISIONS	BY	APP'D
3	3/01/13	Revised Standard	MRM	SHS
2	2/01/13	Revised Standard	MRM	SHS
1	3/01/10	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

LA852A

FHWA APPROVAL		03/15/2013	APP'D	Scott H. Shields	
DESIGNED	MRM	DETAILED	MRM	QUANTITIES	CADD MRM
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	CADD CK. SHS

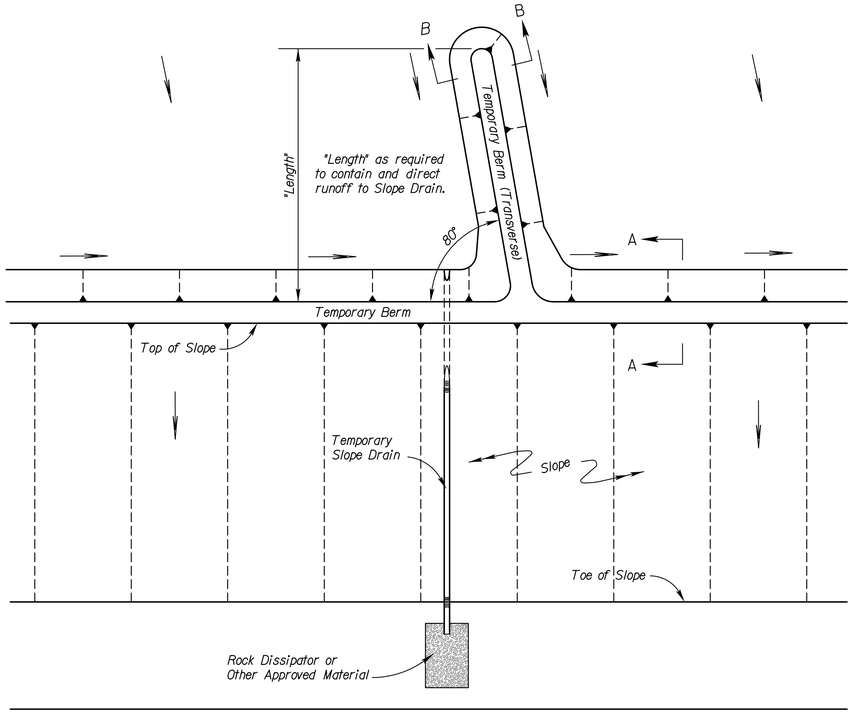
CADconform Certify This File

Sheet No.

Std. Base File:
Plotted By: *dluedke* | Plot Location:
File: *la852a.dgn*
Plot Date: *9/13/2013*

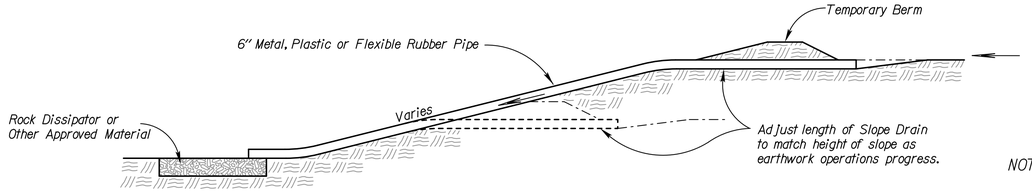
CADconform Certify This File

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	42	99



TYPICAL PLAN VIEW OF TEMPORARY BERM AND TEMPORARY SLOPE DRAIN

NO SCALE



TYPICAL PROFILE OF TEMPORARY SLOPE DRAIN

NO SCALE

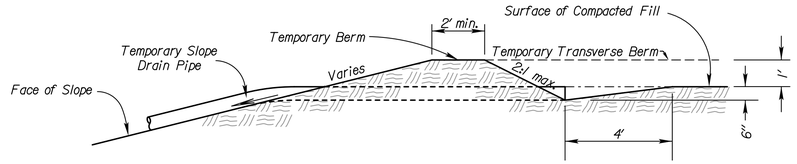
NOTES:

1) Temporary Slope Drain and Temporary Berm may be used on either project foreslopes or project backslopes.

2) Discharge of Slope Drains shall be into stabilized ditch or area, or into Sediment Basin.

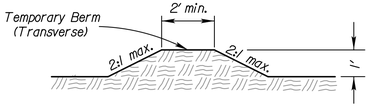
3) Pipe shall be secured in place as approved by Engineer.

4) Temporary Berms under 2,000 feet shall be bid by Set Price.



SECTION A-A

NO SCALE

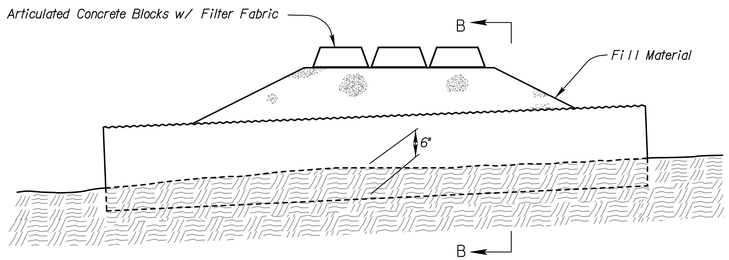


SECTION B-B

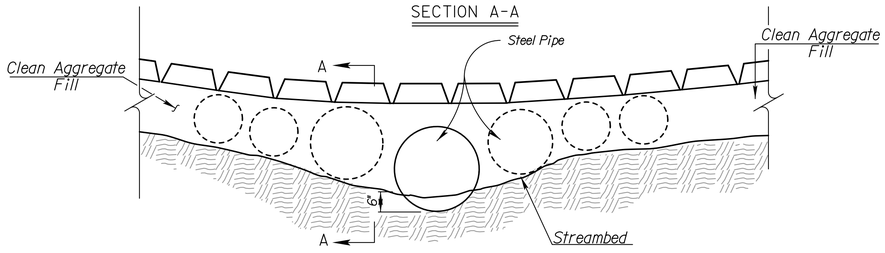
NO SCALE

TYPICAL PROFILE OF TEMPORARY BERM

NO SCALE



SECTION A-A



SECTION B-B

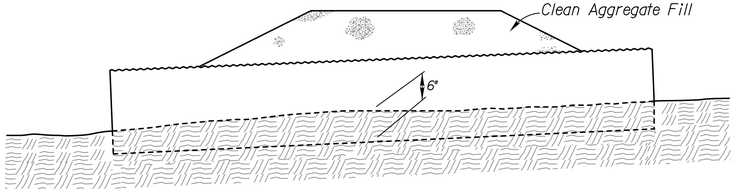
TEMPORARY STREAM CROSSING (ARTICULATED CONCRETE BLOCKS)

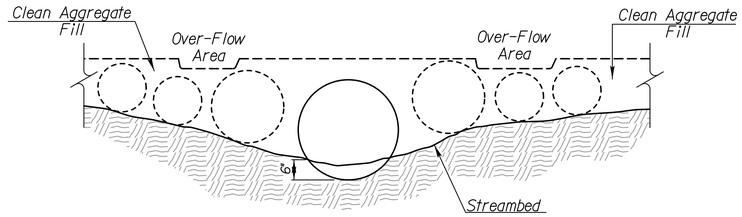
NO SCALE

Pipe size may vary

Place 1 pipe buried 6" into stream bottom, in the lowest point of the channel to allow the passage of aquatic organisms, with additional pipes placed along the remainder of the stream channel bottom such that ordinary high water (OHW) flows designated in the Contract Documents shall flow through the pipes without overtopping the crossing.

See KDOT Specifications for more information





SECTION B-B

TEMPORARY STREAM CROSSING (AGGREGATE)

NO SCALE

Pipe size may vary

Place 1 pipe buried 6" into stream bottom, in the lowest point of the channel to allow the passage of aquatic organisms, with additional pipes placed along the remainder of the stream channel bottom such that ordinary high water (OHW) flows designated in the Contract Documents shall flow through the pipes without overtopping the crossing.

See KDOT Specifications for more information

NO.	DATE	REVISIONS	BY	APP'D
3	6/11/13	Revised Standard	MRM	SHS
2	11/01/10	Revised Standard	MRM	SHS
1	10/15/10	Revised Standard	WCL	RDR

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

TEMPORARY SLOPE DRAIN

TEMPORARY STREAM CROSSING (AGGREGATE)

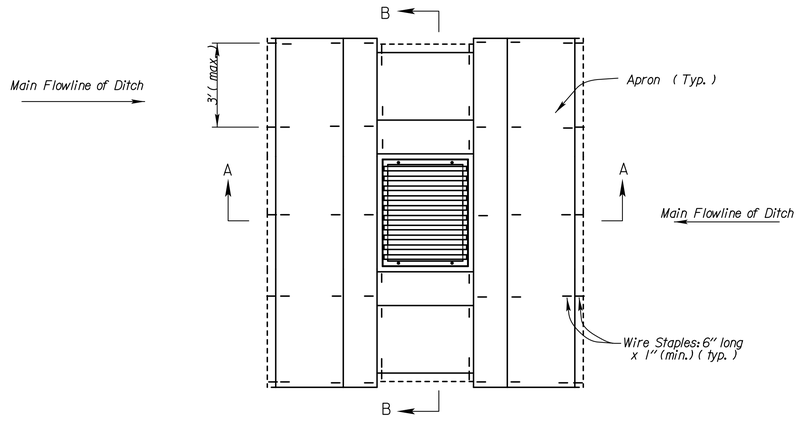
TEMP. STREAM CROSS. (ARTC. CONC. BLOCKS)

LA852B

FHWA APPROVAL	11/08/2010	APP'D	Scott H. Shields
DESIGNED MRM	DETAILED	QUANTITIES	CADD
DESIGN CK. SHS	DETAIL CK.	QUAN.CK.	CADD CK.

Std. Base File:
Plotted By: dluedke | Plot Location:
File: la852b.dgn
Plot Date: 9/13/2013

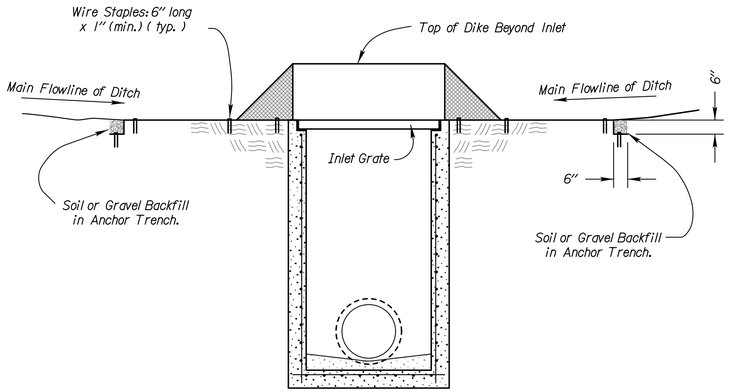
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	43	99



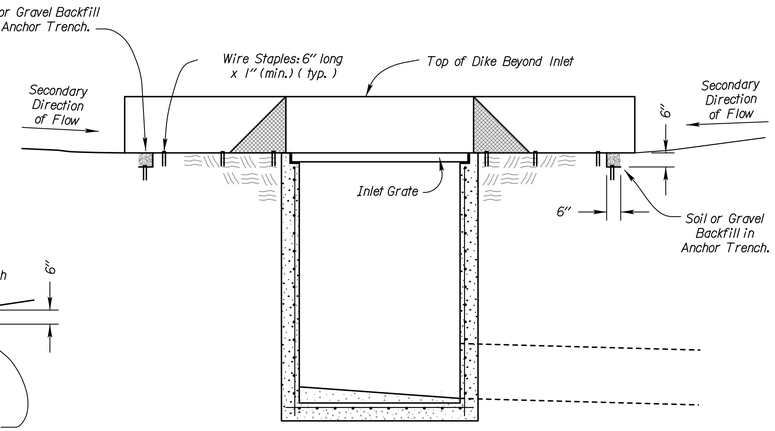
PLAN

TEMPORARY INLET SEDIMENT BARRIER
(TRIANGULAR SILT DIKE METHOD)

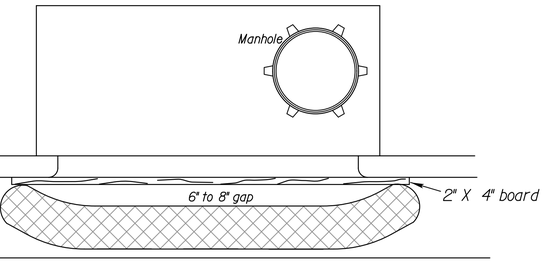
NO SCALE



SECTION A - A



SECTION B - B

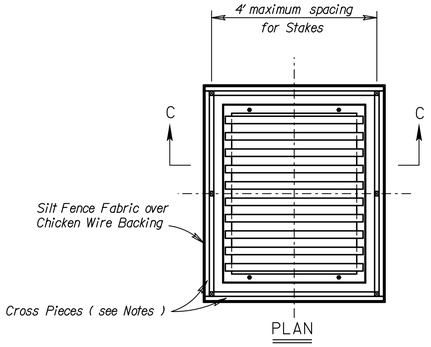


Bags = synthetic net (3mm mesh) or burlap bags

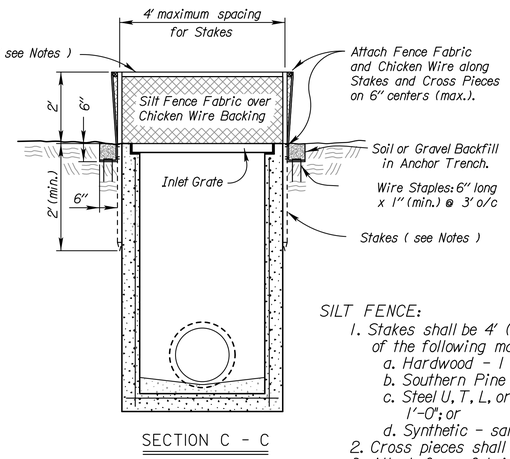
Rock = approximately 1" to 2" diameter

CURB INLET PROTECTION

1. If multiple gravel bags are required, place them in such a way that no gaps are evident.
2. Height of bags (8" minimum diameter) must not be above top of curb.
3. Alternative products may be used other than gravel bags such as the "Gutter Buddy". Products must be approved by the Engineer.
4. Curb inlet protection will be measured and paid for as Filter Sock.



PLAN



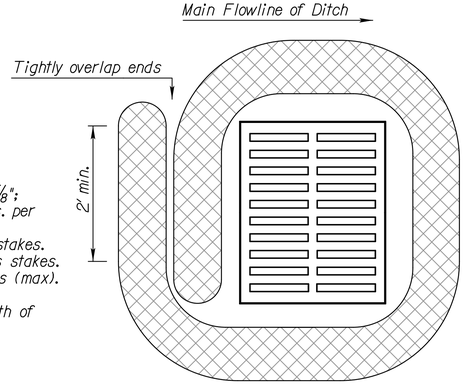
SECTION C - C

TEMPORARY INLET SEDIMENT BARRIER
(SILT FENCE METHOD)

NO SCALE

SILT FENCE:

1. Stakes shall be 4' (min.) long and of one of the following materials:
   a. Hardwood - 1 3/16" x 1 3/16";
   b. Southern Pine (No. 2) - 2 5/8" x 2 5/8";
   c. Steel U, T, L, or C Section - 1.25 lbs. per 1'-0"; or
   d. Synthetic - same strength as wood stakes.
2. Cross pieces shall be of same material as stakes.
3. Attach fence fabric securely on 6" centers (max).
4. Use of high flow material is acceptable.
5. Refer to plan sheets to estimate the length of silt fence required.



Drop inlet use
1'-6" TO 1'-8" diameter log

BIODEGRADABLE LOG/FILTER SOCK
DROP INLET PROTECTION

Note: 25% of log shall be keyed into ground during installation.

Stake every 4'

Material Requirements
Use 100% shredded mulch or other non-compost biodegradable material as fill for logs.
No compost or fines.
No hay or straw.
Do not use material which prohibits water infiltration.
Log Mesh: Use mesh with 1/4" openings or larger. Mesh must allow water infiltration but also hold fill materal in place.

NO.	DATE	REVISIONS	BY	APP'D
3	6/01/13	Revised Standard	MRM	SHS
2	3/01/13	Revised Standard	MRM	SHS
1	8/01/08	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

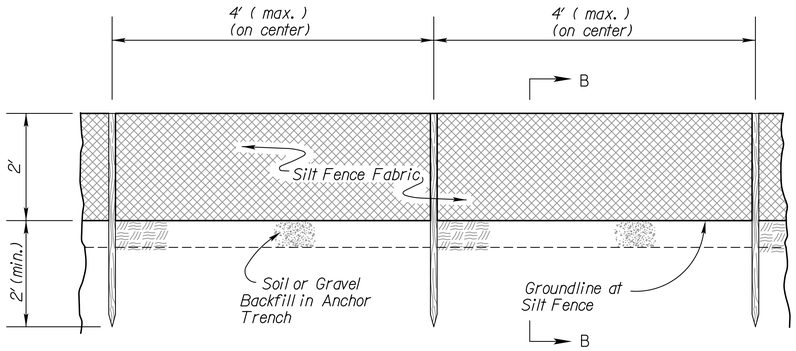
TEMP. INLET SEDIMENT BARRIER (SILT FENCE)
TEMP. INLET SEDIMENT BARRIER (T.S.D.)
CURB INLET PROTECTION
DROP INLET PROTECTION

LA852C

FHWA APPROVAL	5/14/2013	APP'D	Scott H. Shields
DESIGNED MRM	DETAILED MRM	QUANTITIES	CADD
DESIGN CK. SHS	DETAIL CK. SHS	QUAN.CK.	CADD CK.

Std. Base File:
Plotted By: cluedke
File: la852c.dgn
Plot Date: 9/13/2013
Plot Location:

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	44	99



TYPICAL ELEVATION

SILT FENCE SLOPE BARRIER

NO SCALE

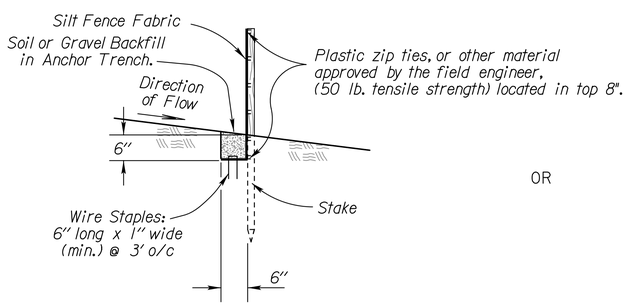
INSTALLATION NOTES

SILT FENCE:

1. Stakes shall be 4' (min.) long and of one of the following materials:
   a. Hardwood - 1 3/16" x 1 3/16";
   b. Southern Pine (No. 2) - 2 5/8" x 2 5/8";
   c. Steel U, T, L, or C Section - 1.25 lbs. per 1'-0"; or
   d. Synthetic - same strength as wood stakes.
2. Cross pieces shall be of same material as stakes.
3. Attach fence fabric securely on 6" centers (max.).
4. Use of high flow material is acceptable.
5. Refer to plan sheets to estimate the length of silt fence required.

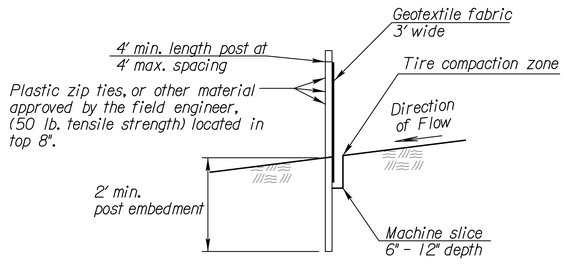
BIODEGRADABLE LOG BARRIERS

1. Place biodegradable logs tightly together.
2. Wood stakes shall be 2" x 2" (nom.).
3. Wire staples shall be 6" long x 1" wide (min.) and placed on 4' (max.) centers.
4. Refer to plan sheets to estimate length of biodegradable log barriers required.
5. Logs should be keyed into the ground at a minimum of 25% of its height.
6. Length of stakes should be 2 times the height of the log at a minimum.



SECTION B-B

OR



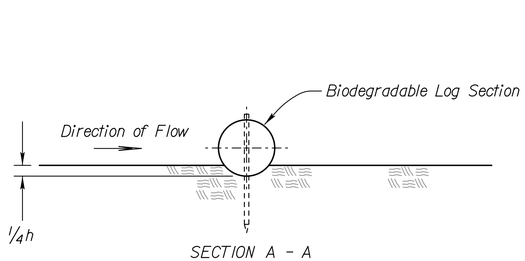
SECTION B-B

Biodegradable Logs, Straw Wattles & Sediment Logs

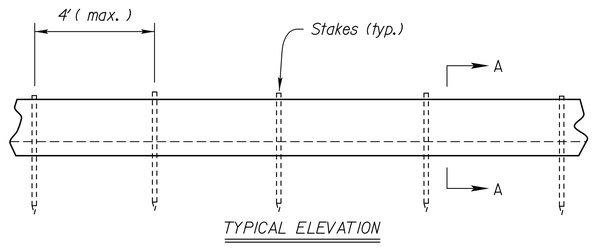
Slope Gradient	PRODUCT: 9" Sediment Log & 9" Straw Wattle (ft)	12" Sediment Log & 12" Straw Wattle (ft)	20" Sediment Log & 20" Straw Wattle (ft)
≤4H:1V	40	60	80
3H:1V	30	45	60
2H:1V	20	30	40
1H:1V	10	15	20

9" and 12" material should only be used in areas which have been seeded and mulched. 20" material should be used in all other areas. Deviations should be approved by the Field Engineer.

	BIODEGRADABLE LOG MATERIAL	
	LOW FLOW	HIGH FLOW
9"	Straw/Compost	Excelsior / Wood Chips / Coconut Fiber
12"	Straw/Compost	Excelsior / Wood Chips / Coconut Fiber
18"-20"	Straw/Compost	Excelsior / Wood Chips / Coconut Fiber



SECTION A - A



TYPICAL ELEVATION

BIODEGRADABLE LOG SLOPE BARRIER

NO SCALE

GENERAL NOTES

1) The slope barriers shall be placed along contour lines, with a short section turned upgrade at each end of the barrier. The maximum length of the slope barrier shall not exceed 250 feet, and the barrier ends need to be staggered.
2) At culverts, the Silt Fence shall be placed over the culvert, not through the streambed flowline.
3) Barriers damaged by Contractor's negligence, including improper maintenance or lack of maintenance, shall be repaired immediately by Contractor at no additional cost to KDOT.
4) Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.

NO.	DATE	REVISIONS	BY	APP'D
1	6/01/13	Revised Standard	MRM	SHS
2	3/01/13	Revised Standard	MRM	SHS
1	9/01/10	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

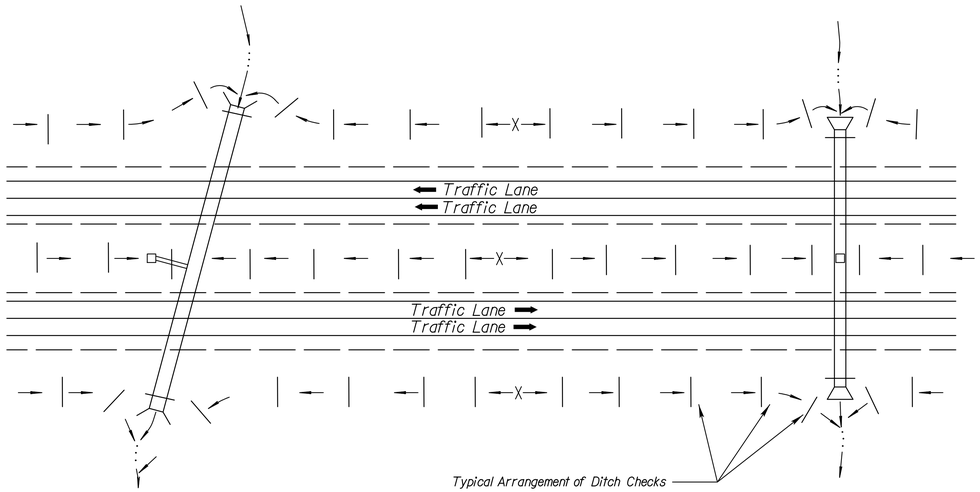
SILT FENCE SLOPE BARRIERS
BIODEGRADABLE LOG SLOPE BARRIERS

LA852D

FHWA APPROVAL		5/14/2013	APP'D	Scott H. Shields	
DESIGNED	MRM	DETAILED MRM	QUANTITIES	CADD	
DESIGN CK.	SHS	DETAIL CK.	QUAN.CK.	CADD CK.	

Std. Base File:	
Plotted By: dluedke	Plot Location:
File: la852d.dgn	
Plot Date: 9/13/2013	

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	45	99



TYPICAL DITCH CHECK LAYOUT PLAN

NO SCALE

TEMPORARY DITCH CHECK SPACING	
DITCH ℄ SLOPE (%)	SPACING INTERVAL (FEET)
1.0	*200*
2.0	*100*
3.0	*65*
4.0	*50*
5.0	*40*
6.0	*33*
NOTE: Use this spacing for all except Rock Ditch Checks.	

GENERAL NOTES

1) *The choice of ditch check methods is at the option of the Contractor.*
2) *Use only rock checks in situations where the ditch slope exceeds 6 percent.*
3) *Ditch checks damaged by Contractor's negligence, including improper maintenance or lack of maintenance, shall be repaired by Contractor at no extra cost to KDOT.*

Std. Base File:	
Plotted By: *dwadke*	Plot Location:
File: *la852e.dgn*	
Plot Date: *9/13/2013*	

NO.	DATE	REVISIONS	BY	APP'D
3	6/01/13	Revised Standard	MRM	SHS
2	9/10/07	Revised Standard	MRM	SHS
1	6/16/05	Revised Standard	WCL	RDR

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL

DITCH CHECKS

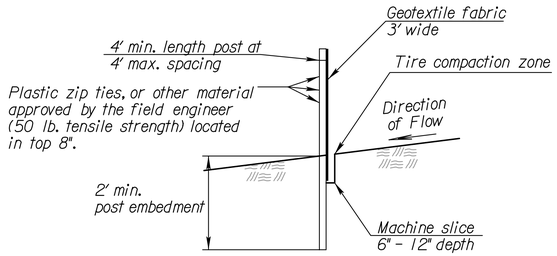
LA852E

FHWA APPROVAL	5/14/2013	APP'D	Scott H. Shields		
DESIGNED	MRM	DETAILED	MRM	QUANTITIES	CADD MRM
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	CADD CK. SHS

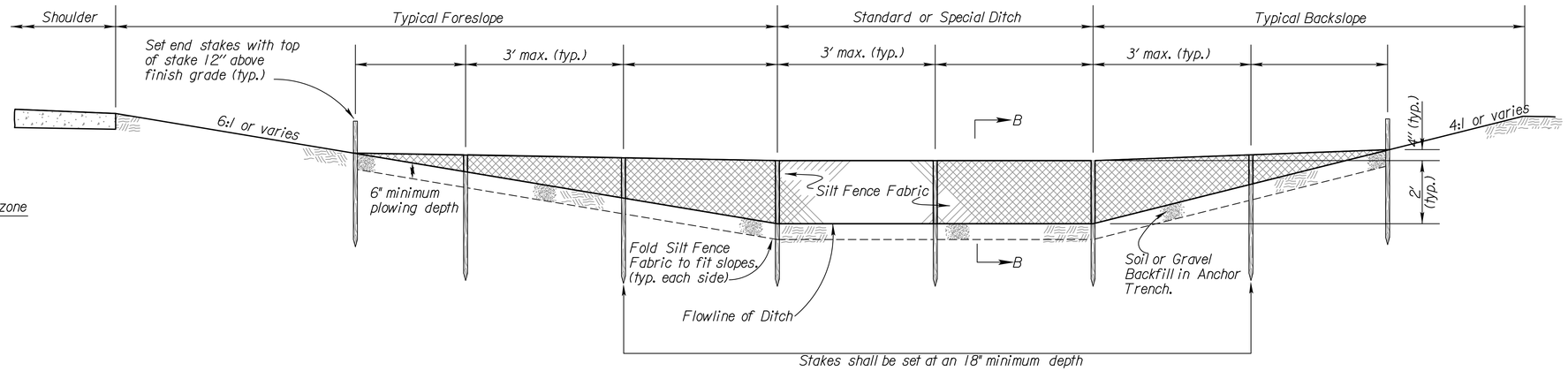
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	46	99

SILT FENCE:

1. Stakes shall be 4' (min.) long and of one of the following materials:
   a. Hardwood - 1 3/16" x 1 3/16";
   b. Southern Pine (No. 2) - 2 5/8" x 2 5/8";
   c. Steel U, T, L, or C Section - 1.25 lbs. per 1'-0"; or
   d. Synthetic - same strength as wood stakes.
2. Cross pieces shall be of same material as stakes.
3. Attach fence fabric securely on 6" centers (max.).
4. Use of high flow material is acceptable.
5. Refer to plan sheets to estimate the length of silt fence required.
6. Use support fencing when tributary area is greater than 2.4 acres or when ditch gradient is greater than 2 percent.
7. Silt fence plowing is acceptable at a 6" minimum depth. Trenching is acceptable in certain cases.

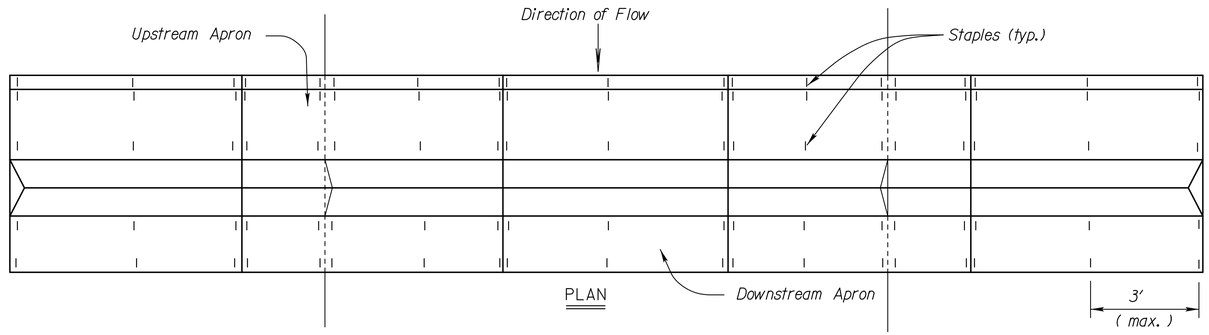


SILT FENCE INSTALLATION
SECTION B-B

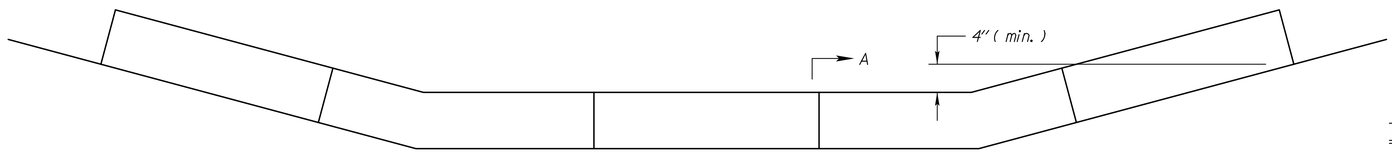


SILT FENCE DITCH CHECK
NO SCALE

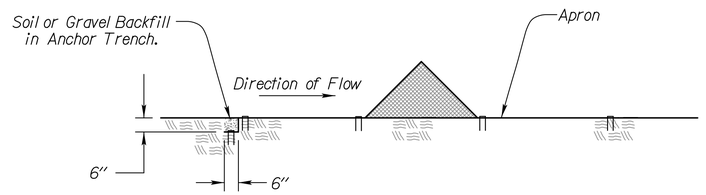
TYPICAL ELEVATION



PLAN



TYPICAL ELEVATION



SECTION A - A

TRIANGULAR SILT DIKE NOTES

1. Place Triangular Silt Dike sections tightly together, with apron material overlapping end-to-end by 6".
2. Wire Staples shall be 6" long by 1" wide, minimum.
3. Use as many Triangular Silt Dike sections as necessary to insure water does not flow around end of ditch check.

TRIANGULAR SILT DIKE DITCH CHECK
NO SCALE

NO.	DATE	REVISIONS	BY	APP'D
	6/01/13	Revised Standard	MRM	SHS
2	3/01/13	Revised Standard	MRM	SHS
1	12/31/09	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

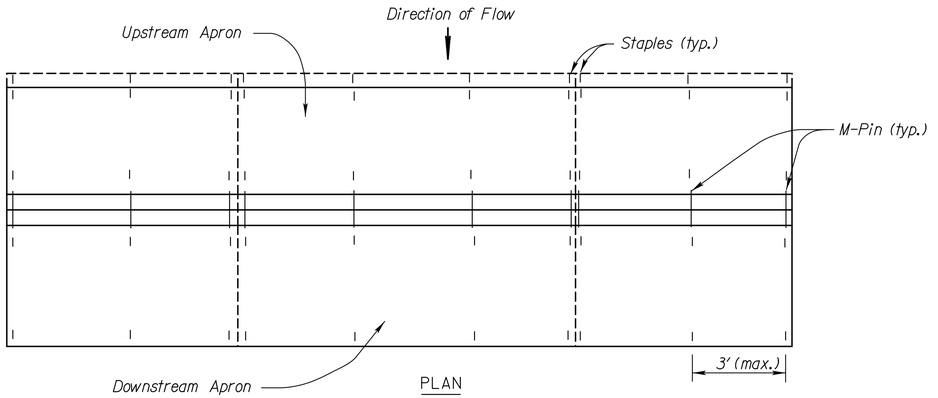
TEMPORARY EROSION AND POLLUTION CONTROL
SILT FENCE DITCH CHECKS
TRIANGULAR SILT DIKE DITCH CHECKS

LA852F

FHWA APPROVAL			5/14/2013	APP'D		Scott H. Shields	
DESIGNED	MRM	DETAILED	MRM	QUANTITIES		CADD	
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.		CADD CK.	

Std. Base File: _____
Plotted By: dluedke | Plot Location:
File: la852f.dgn
Plot Date: 9/13/2013

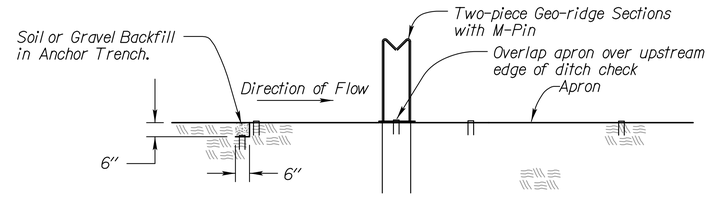
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	47	99



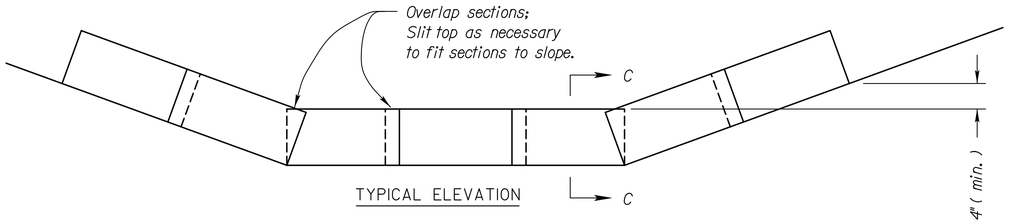
PLAN

GEO-RIDGE PERMEABLE BERM NOTES

1. Overlap Geo-ridge Berm sections and apron material by 6".
2. Use M-Pins supplied by manufacturer to secure geo-ridge Berm sections.
3. Use as many Geo-ridge Berm sections as necessary to insure water does not flow around end of ditch check.
4. Use silt fence material as the apron to prevent scour above and below the ditch check.
5. Wire Staples shall be 6" long by 1" wide, minimum.



SECTION C - C



TYPICAL ELEVATION

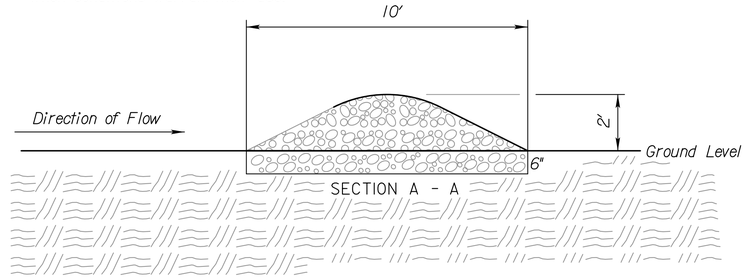
GEO-RIDGE PERMEABLE BERM DITCH CHECK

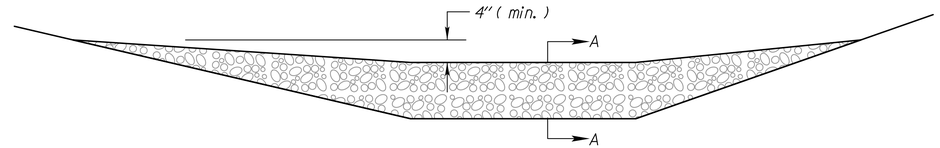
NO SCALE

ROCK DITCH CHECK NOTES

1. Rock shall be clean aggregate, D50 = 6".
2. Place rock in such manner that water will flow over, not around ditch check.
3. Do not use rock ditch checks in clear zone.
4. Excavation: The ditch area shall be reshaped to fill any eroded areas. Prior to placement of the rock, the ditch shall be excavated to the dimensions of the Rock Ditch Check and to a minimum depth of 6" (150mm). After placement of the rock, backfill and compact any over excavated soil to ditch grade. This work shall be subsidiary to the bid item Temporary Ditch Check (Rock) (Set Price).
5. Aggregate excavated on site may be used as an alternate to the 6" rock, if approved by the Engineer.
6. The Engineer may approve the use of larger aggregates when conditions warrant their use.

TEMPORARY ROCK DITCH CHECK SPACING	
DITCH ℄ SLOPE (%)	SPACING INTERVAL (FEET)
5.0	60
6.0	50
7.0	43
8.0	36
9.0	33
10.0	29
NOTE: Use this spacing only for Rock Ditch Checks.	

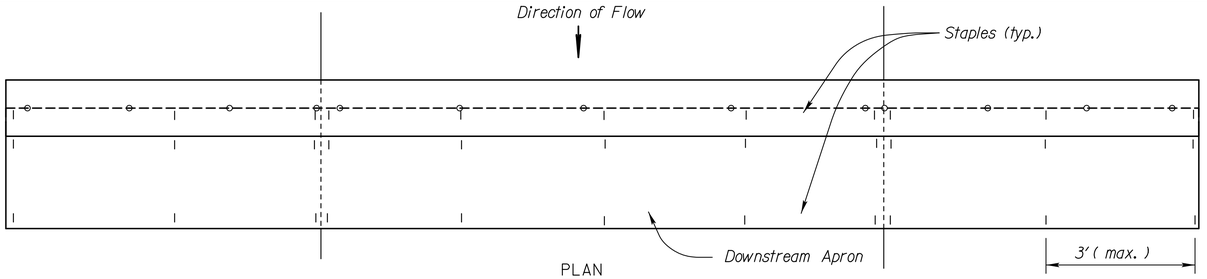




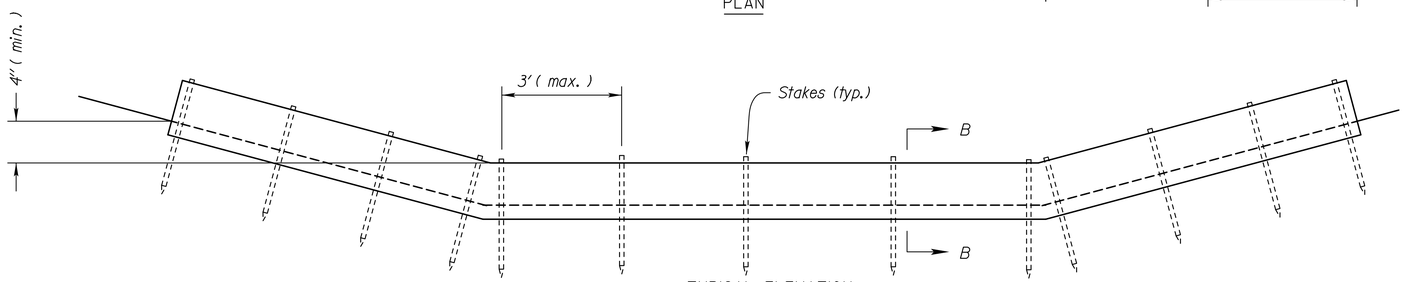
TYPICAL ELEVATION

ROCK DITCH CHECK

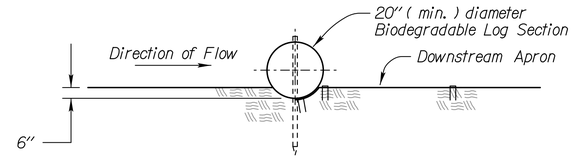
NO SCALE



PLAN



TYPICAL ELEVATION



SECTION B - B

BIODEGRADABLE LOG DIKE NOTES

1. Place biodegradable logs tightly together, with apron material overlapping end-to-end by 6".
2. Wire staples shall be 6" long by 1" wide, minimum.
3. Use as many biodegradable log sections as necessary to insure water does not flow around end of ditch check.
4. Wood stakes shall be 2" x 2" (nom.) x 4' (min.) long.
5. Use silt fence material as the downstream apron to prevent scour below the ditch check.

BIODEGRADABLE LOG DITCH CHECK

NO SCALE

NO.	DATE	REVISIONS	BY	APP'D
	9/01/10	Revised Standard	MRM	SHS
2	12/31/09	Revised Standard	MRM	SHS
1	5/03/06	Revised Standard	MRM	SHS

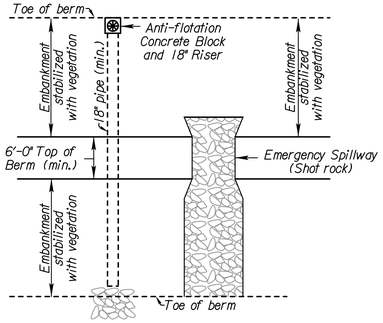
KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY EROSION AND POLLUTION CONTROL
ROCK DITCH CHECKS
BIODEGRADABLE LOG DITCH CHECKS
GEO-RIDGE PERMEABLE BERM DITCH CHECKS
LA852G

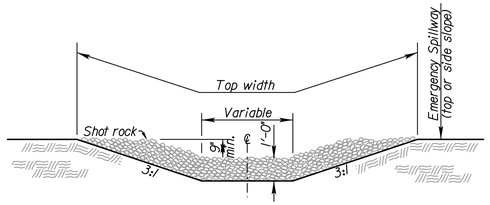
FHWA APPROVAL		9/21/2010	APP'D	Scott H. Shields	
DESIGNED	MRM	DETAILED	MRM	QUANTITIES	CADD
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	CADD CK.

Std. Base File:	la852g.dgn
Plotted By:	cluedke
File:	la852g.dgn
Plot Date:	9/13/2013
Plot Location:	

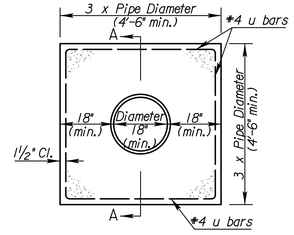
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	48	99



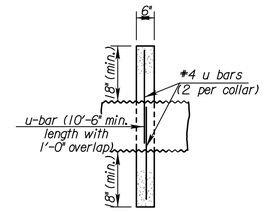
SEDIMENT STORAGE BASIN (PLAN)



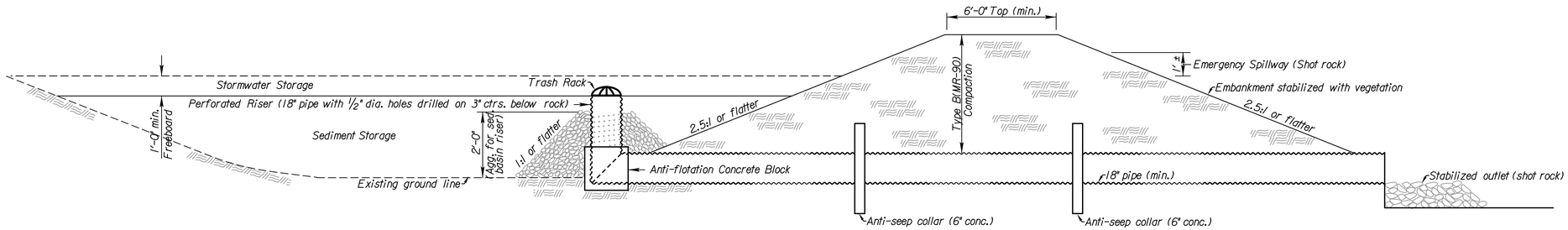
CROSS SECTION (EMERGENCY SPILLWAY)



CONCRETE ANTI-SEEP COLLAR



SECTION A-A



SEDIMENT STORAGE BASIN (ELEVATION)

NOTES:

1) Temporary Sediment Basins shall be constructed at locations as directed by the Engineer or as approved in the SWPPP Schedule. All work and materials necessary, including but not limited to, the fill material, compaction, drainage pipes, aggregates and all other incidentals necessary to construct the basin, shall be paid as "Temporary Sediment Basin".

2) Lengths and top dimensions shall be determined in the field by the Engineer.

SEDIMENT STORAGE BASIN LOCATIONS

STATION TO STATION	SIDE	REQUIRED STORAGE CAPACITY

Std. Base File:
Plotted By: dluedke | Plot Location:
File: la852h.dgn
Plot Date: 9/13/2013

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1	7/17/13	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

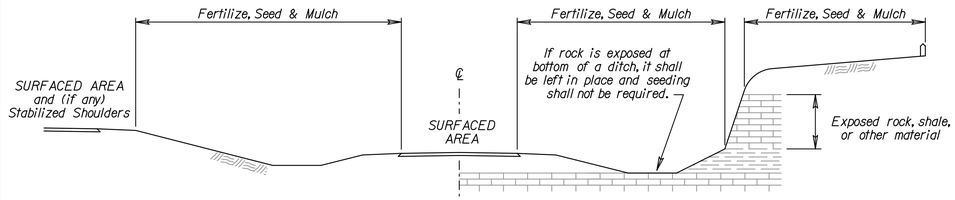
TEMPORARY EROSION AND POLLUTION CONTROL

SEDIMENT STORAGE BASIN

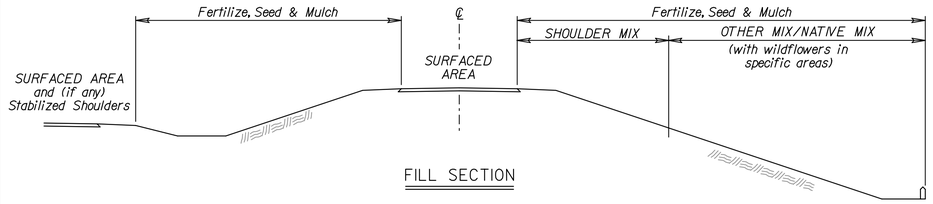
LA852H

FHWA APPROVAL		03/15/2013	APP'D	Scott H. Shields	
DESIGNED	BB	DETAILED	BB	QUANTITIES	BB
DESIGN CK.	SHS	DETAIL CK.	SHS	QUAN.CK.	SHS
				CADD	BB
				CADD CK.	SHS

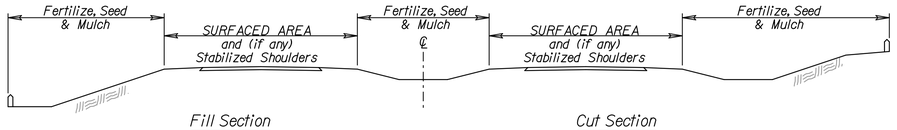
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	49	99



CUT SECTION



FILL SECTION



TYPICAL SECTION - DUAL PAVEMENT

SEEDING PERIODS

COOL SEASON February 15 to April 20 and August 15 to Sept. 30	WARM SEASON November 15 to June 1
SPECIES	SPECIES
Bluegrasses	Big Bluestem
Bromegrasses	Blue Grama
Canada Wildrye	Buffalograss
Fescues	Eastern Gamagrass
Prairie Junegrass	Indiangrass
Reed Canarygrass	Little Bluestem
Ryegrasses	Sand Bluestem
Sterile Wheatgrass	Sand Dropseed
Tall Dropseed	Sand Lovegrass
Western Wheatgrass	Side Oats Grama
	Switchgrass
	Wildflower Mixes

When "Cool Season" species are mixed with "Warm Season" species, in areas of 1 acre or more, the mixture shall be seeded during the "Warm Season". In areas of less than 1 acre, the mixture of "Cool Season" and "Warm Season" species may be seeded during the "Warm or Cool Seasons".

SODDING PERIODS
March 1 to April 15 and September 1 to November 15

GENERAL NOTES

The entire disturbed area, excepting the paved or surfaced areas, steep rocky slopes and areas of undisturbed native sod or other desirable vegetation shall be fertilized (limed when required), seeded, and mulched. Soil preparation shall conform to the Standard Specifications except as noted below.

All borrow areas shown on the plans are to be fertilized, seeded, and mulched. However, operation in borrow areas where crops are growing may be omitted when requested by the owner.

It shall not be required to till the area to bare ground prior to permanent seeding. If temporary cover has provided stable slopes with no erosion, seed the permanent grasses into the existing cover. If there has been erosion that requires repair prior to seeding, then it may be necessary to regrade the area, resulting in bare ground.

FERTILIZER: A ratio and application rate that equals or exceeds the required minimum rate per acre of N, P_2O_5, K_2O listed in Summary of Seeding Quantities will be acceptable.

MULCHING: Mulch shall be spread uniformly over all disturbed areas and punched in the soil, unless otherwise noted on the plans. The rate of application per acre, thickness in place, for the mulching material is as follows:

1¾ - 2¼ Tons per Acre = 1½" loose depth spread uniformly over acre.

Agricultural products, such as native prairie hay, used for mulching and erosion control practices, excluding wood based mulch, shall meet the North American Weed Free Forage Standards.

Other vegetative mulches are acceptable only with the Engineer's concurrence.

The above rate is a guide. It will be at the discretion of the Engineer to determine what rate is sufficient for adequate protection of newly seeded areas.

The amount of mulch in the quantities is estimated. The total mulch required shall be determined in the field. The bid item for mulching shall be paid for according to Standard Specification Section 904.

NATIVE WILDFLOWER MIX 1

PLS RATE	NAME	QTY (lb)
0.1	Black Eyed Susan	
1.8	Illinois Bundleflower	
0.15	Maximilian Sunflower	
0.4	Purple Prairie Clover	
2.9	Showy Partridge Pea	
0.1	Upright Prairie Coneflower	
0.3	Butterfly Milkweed	
0.1	Stiff Goldenrod	
0.05	Pinnate Prairie Coneflower	
0.1	Lance-leaf Coreopsis	
0.05	New England Aster	
0.2	Pale Purple Coneflower	
0.05	Plains Coreopsis	
0.05	Hoary Verbena	
0.3	Roundhead Lespedeza	
0.4	Thickspike Gayfeather	
0.05	Wild Bergamot	
0.2	Smooth Oxeye	
0.05	Lemon Mint	
7.35	Total (lb)	

NATIVE WILDFLOWER MIX 2

PLS RATE	NAME	QTY (lb)
0.1	Black Eyed Susan	
1.8	Illinois Bundleflower	
0.15	Maximilian Sunflower	
0.4	Purple Prairie Clover	
2.9	Showy Partridge Pea	
0.1	Upright Prairie Coneflower	
0.3	Butterfly Milkweed	
0.4	Dotted Blazing Star	
0.4	Annual Gaillardia	
0.05	Stiff Goldenrod	
0.05	New England Aster	
0.3	Missouri Evening Primrose	
0.05	Plains Coreopsis	
0.15	White Prairie Clover	
0.3	Roundhead Lespedeza	
0.05	Lemon Mint	
0.15	Pitcher Sage	
7.65	Total (lb)	

NATIVE WILDFLOWER MIX 3

PLS RATE	NAME	QTY (lb)
0.15	Black Eyed Susan	
1.9	Illinois Bundleflower	
0.15	Maximilian Sunflower	
0.05	Western Yarrow	
0.5	Black Sampson Echinacea	
0.05	Upright Prairie Coneflower	
0.3	Butterfly Milkweed	
0.4	Dotted Blazing Star	
0.75	Annual Gaillardia	
0.05	Stiff Goldenrod	
0.05	New England Aster	
0.4	Pitcher Sage	
0.01	Plains Coreopsis	
0.15	White Prairie Clover	
0.2	Purple Prairie Clover	
0.4	Leadplant	
0.02	White Heath Aster	
1	Blue Wild Indigo	
0.05	Lemon Mint	
6.58	Total (lb)	

NATIVE WILDFLOWER MIX 4

PLS RATE	NAME	QTY (lb)
1.9	Illinois Bundleflower	
0.4	Maximilian Sunflower	
0.05	Western Yarrow	
1	Black Sampson Echinacea	
0.1	Upright Prairie Coneflower	
0.1	Scarlet Globemallow	
0.4	Dotted Blazing Star	
1.1	Annual Gaillardia (Firewheel)	
0.1	Hoary Vervain	
0.3	White Prairie Clover	
0.4	Purple Prairie Clover	
0.4	Perennial Gaillardia (Blanket Flower)	
0.02	White Heath Aster	
0.05	Lemon Mint	
6.32	Total (lb)	

Package and deliver the wildflower seed separately from the grass seed mix. Package and deliver the Tall Drop Seed separately from the grass seed and the wildflower mix. Place the grass seed (except Tall Drop Seed) in the large seed box and drill (cover) seed 1/8" - 1/4". Place the wildflower seed in a separate seed box and drill (cover) seed 1/16" maximum. Place the Tall Drop Seed in a separate (third) seed box and place the seed (using the seed drill) on the soil surface.
OPTION: Broadcast Tall Drop Seed on the soil surface.

SUMMARY OF SEEDING QUANTITIES

P.L.S. RATE/ACRE URBAN	P.L.S. RATE/ACRE OTHER	ACRES URBAN	ACRES OTHER	BID ITEM	QUANTITY	UNIT
180		0.2		Seed (Turf-type Tall Fescue Seed Blend)		lb.
100		0.2		Fertilizer (15-30-15)		lb.
				Sod (Fine-Bladed Fescue Blend)(Strips)	66	Sq.Yd.
				Seeding	Lump Sum	L.S.
				Mulching (Permanent) (Set Price)		TON

SHLDR = Shoulder Turf Mix: Includes a 30 foot wide strip along the stabilized shoulder on each side of each traveled way, plus all median areas less than 60 feet wide.

OTHER = All other turf areas except Shoulder, Guardrail, and Native areas usually include the Native Wildflower Mix.

NOTE: Projects of less than 1 acre shall be bid as "Seeding" by the lump sum. All disturbed areas shall be seeded, fertilized and mulched at the listed rate per acre. The acres are estimated.

Sod Locations

Location	Side	Area (Sq. Yd.)	Remarks
Johnson Drive	Rt.	41.1	S.W. Corner
Johnson Drive	Lt.	7.7	N.W. Corner
Johnson Drive	Lt.	11.6	N.E. Corner
Johnson Drive	Rt.	5.7	S.E. Corner
	Total	66.1	

Note: The Contractor shall contact the City of Merriam to finalize the sod placement limits for the Project. Landscaping plantings at the corners of the bridge will be provided and installed by the City of Merriam.

Std. Base File:
Plotted By: cluedke
File: la850.dgn
Plot Date: 9/13/2013
Plot Location:

NO.	DATE	REVISIONS	BY	APP'D
4	6/01/13	Revised Standard	MRM	SHS
3	3/01/13	Revised Standard	MRM	SHS
2	2/24/12	Revised Standard	MRM	SHS
1	6/01/10	Revised Standard	MRM	SHS

KANSAS DEPARTMENT OF TRANSPORTATION

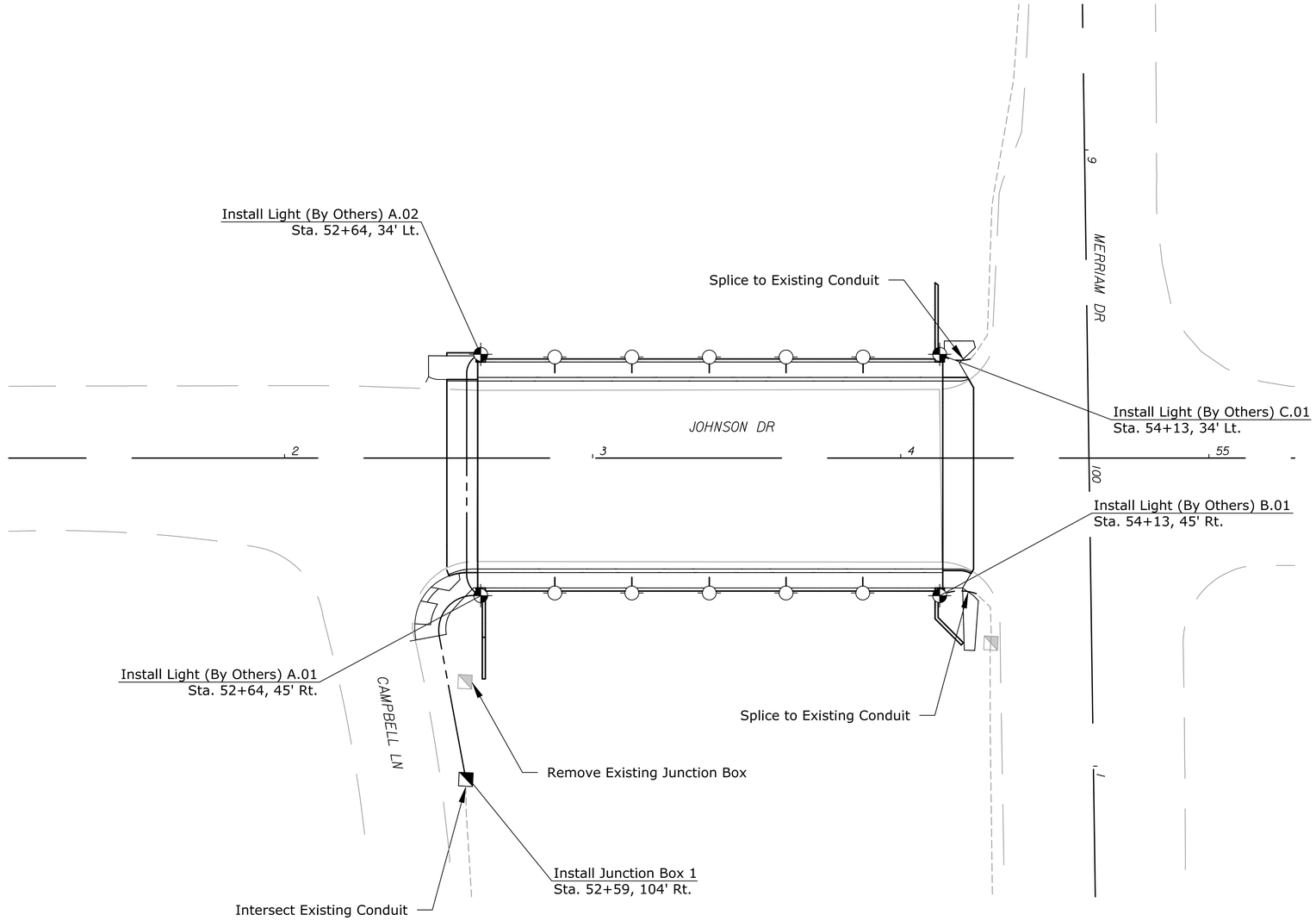
PERMANENT SEEDING
SUMMARY OF SEEDING QUANTITIES

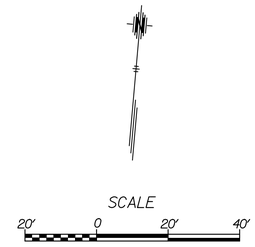
~~LA850~~

FHWA APPROVAL	5/14/2013	APP'D	Scott H. Shields
DESIGNED MRM	DETAILED MRM	QUANTITIES	CADD
DESIGN CK.	DETAIL CK.	QUAN.CK.	CADD CK.

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	50	99

DATE	BY	
		REFERENCES NOTED
		REFERENCES CHECKED





LEGEND

- Existing Conduit
- Proposed Conduit
- Existing Junction Box
- Proposed Junction Box
- Post Top Luminaire (By Others)
- Banner/Planter Post

NOTE: Luminaires to be installed by others after completion of project. Lighting foundations, conduit, and wire to be installed by the Contractor on this project.

Drawn By : cluedke
Plotted : 9/13/2013
File : G:\12664\MicroTraffic\12664_5101.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

LIGHTING PLAN

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	51	99

DATE	BY	
		REFERENCES NOTED
		REFERENCES CHECKED



N.T.S

LEGEND

- Existing Conduit
- Proposed Conduit
- Existing Junction Box
- 01 Proposed Junction Box
- Junction Box #
- A 01 Circuit
- Pole Number

KANSAS DEPARTMENT OF TRANSPORTATION

STREET LIGHTING

WIRING DIAGRAM

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\Micro\Traffic\12664_5102.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	52	99

THE GENERAL NOTES:

THE GENERAL NOTES, REMOVAL NOTES AND NOTES ON THE PLAN SHEETS HAVE BEEN GENERALLY WRITTEN IN THE IMPERATIVE MOOD. THE SUBJECT ' THE CONTRACTOR' IS IMPLIED. ALSO IMPLIED IN THIS LANGUAGE ARE 'SHALL', 'SHALL BE', OR SIMILAR WORDS OR PHRASES. THE WORD 'WILL' GENERALLY PERTAINS TO DECISIONS OR ACTIONS OF THE KANSAS DEPARTMENT OF TRANSPORTATION (KDOT).

ALL WIRING AND WIRING METHODS SHALL COMPLY WITH THE MOST CURRENT NATIONAL ELECTRIC CODE STANDARDS, ANY CHANGES TO THESE STANDARDS SHALL BE APPROVED BY THE AUTHORITY HAVING JURISDICTION.

LOCATE ALL EXISTING UTILITES PRIOR TO BEGINNING WORK.

GROUND ALL METAL PARTS OF SWITCHES, CASES, POLES AND CONDUIT USED ABOVE GROUND WITH A NO. 6 BARE WIRE.

PROVIDE INSPECTION OF EACH TOWER BY A FACTORY REPRESENTATIVE OF THE RAISE AND LOWER DEVICE PRIOR TO FINAL ACCEPTANCE OF THE PROJECT BY THE KANSAS DEPARTMENT OF TRANSPORTATION.

MAKE SPLICES IN THE INDUCT SECONDARY CABLE INSIDE OF TOWER BASES, POLE BASES AND JUNCTION BOXES. CONTACT THE BUREAU OF TRANSPORTATION SAFETY & TECHNOLOGY FOR APPROVAL OF OTHER SPLICE LOCATIONS.

A REPRESENTATIVE FROM THE KANSAS DEPARTMENT OF TRANSPORTATION'S REGIONAL MATERIALS LAB WILL VISUALLY INSPECT ALL TOWERS AND COMPLETE A TEST REPORT PRIOR TO ASSEMBLY.

PROVIDE WELD PROCEDURE NUMBERS FOR THE DIFFERENT TYPES OF WELDS ON THE SHOP DRAWINGS FOR LIGHTING TOWERS. INCLUDE THE KDOT LAB NUMBER UNDER WHICH THESE WELD PROCEDURES HAVE BEEN APPROVED.

OZ EXPANSION JOINTS SHALL BE INSTALLED IN THE CONDUIT RUNS AT ALL BRIDGE EXPANSION JOINTS.

FURNISH AND INSTALL LIGHTING EQUIPMENT, INCLUDING TOWERS, CONCRETE BASES FOR TOWERS, ANCHOR BOLTS, LUMINAIRES, LAMPS, WIRING, CONDUIT, CONDUIT FITTINGS, SUPPORTS, INTERCONNECTIONS, GROUND RODS, AND ALL INCIDENTIALS NECCESSARY FOR THE COMPLETE INSTALLATION WITHIN THE LIMITS SHOWN ON THE PLANS. FURNISH AND INSTALL ALL EQUIPMENT NECESSARY FOR SATISFACTORY OPERATION, WHETHER SPECIFICALLY MENTIONED OR NOT.

PLOW OR TRENCH AND BACKFILL UNDERGROUND CONDUCTOR INDUCT APPROXIMATELY 2 FT. BELOW GROUNDLINE. BACKFILL IN ALL AREAS SHALL BE FREE OF RUBBLE AND ROCK.

FINAL LOCATION FOR ENCLOSURE INSTALLATION(S) IS/ARE TO BE DETERMINED BY THE ENGINEER IN CHARGE. POSITION THE ENCLOSURE(S) SO THAT THE PHOTOCELL(S) IS/ARE POINTED DUE NORTH.

THE FOLLOWING COMPANIES HAVE A RAISE AND LOWERING DEVICE THAT HAS BEEN ACCEPTED BY THE KANSAS DEPARTMENT OF TRANSPORTATION: AMERICAN HIGH MAST, EAGLE HIGH MAST LIGHTING CO., CAROLINA HIGH MAST, HOLOPHANE, METROLUX AND STRATUS. ADDITIONAL COMPANIES DESIRING APPROVAL BY THE KDOT MUST INSTALL ONE RAISE AND LOWERING DEVICE ON A POLE PROVIDED BY THE KDOT IN TOPEKA, OR ON ANOTHER POLE AS APPROVED. THE INSTALLATION WILL BE REVIEWED AND TESTED BY KDOT PRIOR TO APPROVAL ON THIS PROJECT. ALL ASSOCIATED COSTS WILL BE THE RESPONSIBILITY OF THE MANUFACTURERS/CONTRACTORS.

THE FOLLOWING COMPANIES HAVE SCREW-IN LIGHT POLE FOUNDATIONS THAT HAVE BEEN ACCEPTED BY THE KANSAS DEPARTMENT OF TRANSPORTATION : A.B. CHANCE COMPANY AND J.H. BOTTS, INC. ADDITIONAL COMPANIES DESIRING APPROVAL BY KDOT MUST SUBMIT DESIGN COMPUTATIONS AND CATALOG CUTS TO THE KANSAS DEPARTMENT OF TRANSPORTATION, BUREAU OF TRANSPORTATION SAFETY & TECHNOLOGY, EISENHOWER BUILDING, 700 SW HARRISON, 6TH FLOOR, TOPEKA, KS 66603-3754.

DUE TO SOIL CONDITIONS IN ALL OR PART OF THE FOLLOWING COUNTIES, SCREW-IN LIGHT POLE FOUNDATIONS MAY ONLY BE USED FOR ROADWAY LIGHTING UPON APPROVAL BY THE KANSAS DEPARTMENT OF TRANSPORTATION, BUREAU OF TRANSPORTATION SAFETY & TECHNOLOGY: BOURBON, CRAWFORD, CHEROKEE, LABETTE, MEADE, CLARK, COMANCHE, KIOWA, BARBER, HARPER, AND KINGMAN. SCREW-IN FOUNDATIONS SHALL NOT BE INSTALLED IN DISTRICT ONE.

ALL ATTACHMENTS TO BRIDGES SHALL HAVE A HIGHWAY RIGHT OF WAY PERMIT (DOT304) AND UTILITY ATTACHMENT PERMIT (DOT 310) APPROVED BEFORE WORK BEGINS, IN ACCORDANCE WITH CURRENT KANSAS DEPARTMENT OF TRANSPORTATION UTILITY ACCOMMODATION POLICY.

NO.	DATE	REVISIONS	BY	APP'D
3	5/7/13	Fixed Typo's	C.E.	B.D.G.
2	12/28/11	Fixed Typo	C.E.	B.D.G.
1	6/14/11	Changed to 3" Schedule 80 PVC	V.V.H.	V.V.H.

KANSAS DEPARTMENT OF TRANSPORTATION

GENERAL NOTES

~~TE200b~~

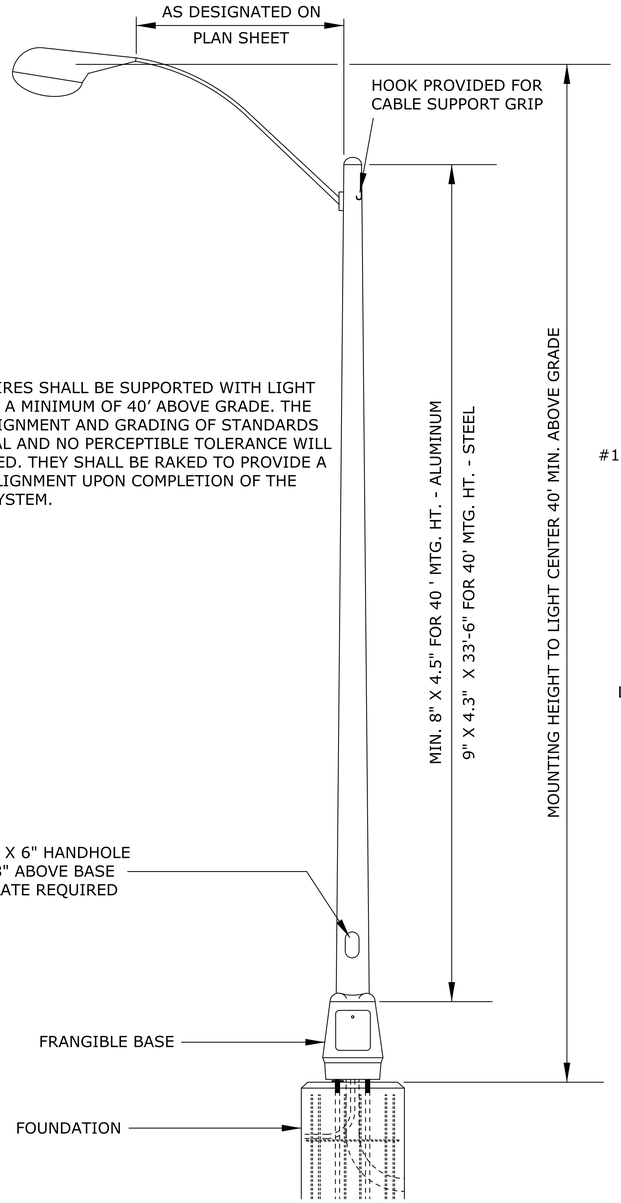
FHWA APPROVAL			APP'D		
DESIGNED	C.E.	DETAILED	C.E.	QUANTITIES	TRACED
DESIGN CK.	B.D.G.	DETAIL CK.	B.D.G.	QUAN. CK.	TRACE CK.

Plotted : 13-SEP-2013 10:53

Drawn By : cluedke
File : te200b.dgn

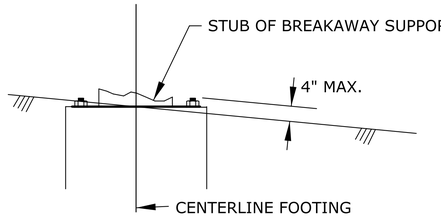
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	53	99

GROUNDING NOTE:
LIGHT STANDARDS SHALL BE GROUNDED BY MEANS OF A NO.6 AWG SOLID BARE COPPER GROUND WIRE BOLTED TO THE INSIDE OF THE LIGHT STANDARD SHAFT WITH A 1½" GROUNDING NUT. GROUND WIRE SHALL BE CONNECTED TO THE COPPERCLAD GROUND ROD BY MEANS OF A GROUND ROD CLAMP WITH AN EXOTHERMIC CONNECTION.



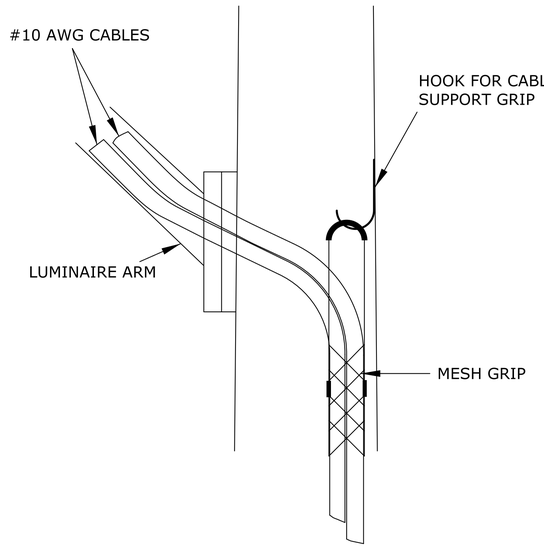
NOTE:
ALL LUMINAIRES SHALL BE SUPPORTED WITH LIGHT CENTERS AT A MINIMUM OF 40' ABOVE GRADE. THE CAREFUL ALIGNMENT AND GRADING OF STANDARDS IS ESSENTIAL AND NO PERCEPTIBLE TOLERANCE WILL BE PERMITTED. THEY SHALL BE RAKED TO PROVIDE A VERTICAL ALIGNMENT UPON COMPLETION OF THE LIGHTING SYSTEM.

THE 4" MAX. IS A REQUIREMENT OF THE BREAKAWAY CRITERIA IN THE LATEST EDITION OF AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS (NO. 12.5.3).



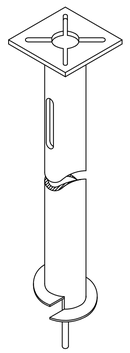
BREAKAWAY SUPPORT STUB CLEARANCE DIAGRAM

NOTE:
THE FOUNDATION SHALL BE INSTALLED TO MEET THE BREAKAWAY SUPPORT STUB CLEARANCE OF 4" MAXIMUM, TO INCLUDE ANCHOR BOLTS.



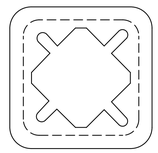
CABLE SUPPORT GRIP

NOTE:
THE CABLE SUPPORT GRIP SHALL BE INSTALLED ON THE HOOK SUPPLIED WITHIN THE LIGHT POLE AS SHOWN ABOVE.

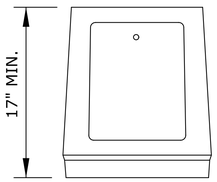


SCREW-IN BASE

NOTE:
DUE TO SOIL CONDITIONS IN ALL OR PART OF THE FOLLOWING COUNTIES, SCREW-IN LIGHT POLE FOUNDATIONS MAY ONLY BE USED FOR ROADWAY LIGHTING UPON APPROVAL BY THE KANSAS DEPARTMENT OF TRANSPORTATION BUREAU OF TRANSPORTATION SAFETY & TECHNOLOGY: BOURBON, CRAWFORD, CHEROKEE, LABETTE, MEADE, CLARK, COMANCHE, KIOWA, BARBER, HARPER, AND KINGMAN. SCREW-IN FOUNDATIONS SHALL NOT BE INSTALLED IN DISTRICT ONE.



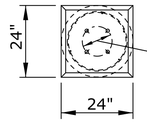
TOP AND BOTTOM VIEW



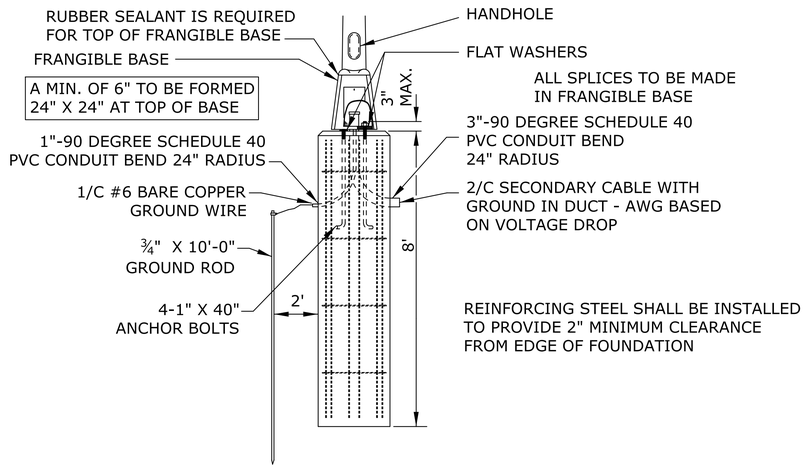
SIDE VIEW

FRANGIBLE BASE

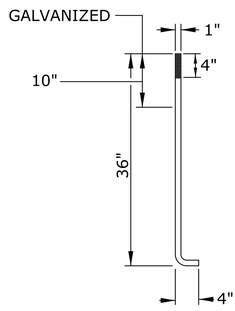
* NOTE:
THE FRANGIBLE BASE MUST ACCOMMODATE UP TO A 15" BOTTOM BOLT CIRCLE.



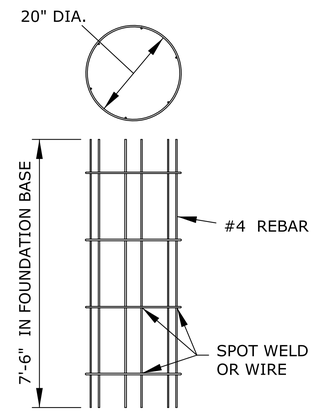
TOP VIEW FOUNDATION



FOUNDATION DETAIL



ANCHOR BOLT DETAIL



REBAR DETAIL

NO.	DATE	REVISIONS	BY	APP'D
3	1/12/12	Chg'd Foundation Detail to show 1" PVC Cond	C.E.	B.D.G.
2	2/25/11	Modify Bolt Circle Note	V.H.	B.D.G.
1	2/4/11	Chg'd Bolt Circle Diameter	V.H.	J.F.F.

KANSAS DEPARTMENT OF TRANSPORTATION

ROADWAY
LIGHTING DETAILS

TE201

FHWA APPROVAL		12/15/2011	APP'D	Brian D. Gower	
DESIGNED	C.E.	DETAILED	C.E.	QUANTITIES	TRACED
DESIGN CK.	B.D.G.	DETAIL CK.	B.D.G.	QUAN. CK.	TRACE CK.

KDOT Graphics Certified 01-12-2012

Sh. No. 53

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:53
Drawn By : cluedke
File : te201.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	54	99

NOTE:
WHENEVER THE PLAN SPECIFICATIONS CONFLICT WITH THE STANDARD KANSAS DEPARTMENT OF TRANSPORTATION SPECIFICATIONS, LATEST EDITION, THE PLAN SPECIFICATIONS SHALL GOVERN.

CONSTRUCTION:
LIGHT STANDARDS SHALL BE DESIGNED TO COMPLY WITH THE 2009 EDITION OF THE AASHTO PUBLICATION, STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR THE HIGHWAY SIGNS, LUMINAIRES, TRAFFIC SIGNALS, AND CURRENT INTERIMS. USE FATIGUE CATEGORY II. SEE STANDARD SPECIFICATIONS, LATEST EDITION, SECTION 814.

LIGHT STANDARDS:
(A) STEEL STANDARDS: THE LIGHT STANDARDS SHALL COMPLY WITH THE REQUIREMENTS OF ASTM 607 WITH THE EXCEPTIONS AND/OR ADDITIONS AS LISTED IN THE STANDARD KANSAS DEPARTMENT OF TRANSPORTATION SPECIFICATIONS, LATEST EDITION, SECTION 1608.2. THE LIGHT STANDARDS WITH 40' MOUNTING SHALL BE ONE (1) SECTION. THE DIAMETER OF THE STANDARDS SHALL BE SHOWN ON THE DETAIL SHEETS.

(a) BASE: THE BASE SHALL BE EITHER A ONE (1) PIECE STEEL BASE COMPLYING WITH THE REQUIREMENTS OF ASTM A-27, GRADE 65-35, A STEEL FORGED BASE COMPLYING WITH THE REQUIREMENTS OF ASTM A-668, CLASS C OR A PLATE BASE COMPLYING WITH THE REQUIREMENTS OF ASTM A 36-77A. THE BASE SHALL BE SECURED TO THE BOTTOM SECTION BY TWO (2) CONTINUOUS ELECTRIC ARC WELDS.

(b) MAST ARMS: THE MAST ARMS SHALL BE FABRICATED FROM 2" STANDARD PIPE MEETING THE REQUIREMENTS OF ASTM A-53 OR ASTM A-35.

(c) FINISH: THE BASE, MAST ARMS, AND LIGHT STANDARDS SHALL BE GALVANIZED ACCORDING TO ASTM A-123.

(B) ALUMINUM STANDARDS: STANDARDS SHALL CONFORM TO THE PLAN DRAWINGS AND SPECIFICATIONS. THE SHAFT SHALL BE SPUN FROM ONE PIECE OF SEAMLESS TUBING (ASTM B-429) ALUMINUM ASSOCIATION ALLOY 6063-T6, WITH MINIMUM NOMINAL WALL THICKNESS OF 0.312 INCH FOR 40' MOUNTING HEIGHT. AFTER FABRICATION IT SHALL HAVE MECHANICAL STRENGTH OF NOT LESS THAN T6 TEMPER. THE SHAFT SHALL HAVE NO LONGITUDINAL NOR CIRCUMFERENTIAL WELDS EXCEPT AT THE LOWER END JOINING THE SHAFT TO THE BASE.

(a) BASE: A ONE-PIECE CAST ALUMINUM (ASTM B-108 ALLOY S.G. 70A-T6) ANCHOR BASE OF ADEQUATE STRENGTH, SHAPE AND SIZE BE SECURED TO THE LOWER END OF THE SHAFT BY TWO (2) CONTINUOUS WELDS. THE BASE SHALL TELESCOPE THE SHAFT AND THE ONE WELD SHALL BE ON THE INSIDE OF THE BASE AT THE END OF THE SHAFT WHILE THE OTHER WELD SHALL BE ON THE OUTSIDE AT THE TOP OF THE BASE. (THE TWO (2) WELDS SHALL DEVELOP THE DESIGN STRENGTH OF THE POLE ASSEMBLY.)

(b) MAST ARMS: SINGLE MEMBER TYPE: THE BRACKET ARM ASSEMBLY SHALL CONSIST OF A TAPERED ALUMINUM ARM, ELLIPTICAL IN SHAPE AT THE POLE END AND TAPERING TO A MINIMUM OF 2 1/4" O.D. AT THE LUMINAIRE END, MADE OF (ASTM B-429) ALUMINUM ASSOCIATION ALLOY 6063-T6 TUBING.

TRUSS TYPE MEMBER: THE UPPER AND LOWER MEMBERS SHALL BE 2" IPS ALUMINUM PIPE, (ASTM B-429) ALUMINUM ASSOCIATION ALLOY 6063-T6.

(c) FINISH: ALL HARDWARE (BOLTS, NUTS, AND WASHERS - BUT NOT INCLUDING ANCHOR BOLTS) NOT OTHERWISE SPECIFICALLY DESIGNATED IN THE SPECIFICATIONS, SHALL BE ALUMINUM OR STAINLESS STEEL. ALL MATERIALS SHALL BE FURNISHED IN NATURAL ALUMINUM COLOR, POLE SHAFTS SHALL BE FURNISHED WITH A POLISHED SURFACE.

ELECTRICAL MATERIAL:

(A) PHOTO CELL: PHOTO-ELECTRIC CONTROL SHALL BE SOLID STATE TYPE, 1000 W/1800 VA, SINGLE POLE, SINGLE THROW, TWIST LOCK MOUNTING, 105 TO 300 VOLT OPERATION. THE OPERATING LEVELS SHALL BE 1.6 FT. C. ON AND 0.4 FT. C. OFF, WITH AN ALLOWABLE 0.5 FT. C. VARIANCE ON OR OFF. THE PHOTO-ELECTRIC CONTROL SHALL HAVE A MINIMUM OF 30 SECOND TIME DELAY AND FAIL IN THE ON MODE.

(B) DUCT: THE DUCT FOR SECONDARY CABLE UNDERGROUND SHALL BE POLYETHYLENE DUCT WITH MINIMUM TENSILE STRENGTH OF 3100 PSI DUCT TO PROVIDE FOR 40% MAXIMUM FILL. THE DUCT SIZED SHALL MEET ASTM D3485 (LATEST REVISION).

(C) CONDUCTOR: CONDUCTOR SHALL BE STRANDED ANNEALED COPPER MEETING THE REQUIREMENTS OF ASTM B-8 AND ASTM B-33.

(a) SECONDARY CABLE (INDUCT): SECONDARY CABLE SHALL BE COPPER SINGLE CONDUCTOR CABLE FOR OPERATION AT 600 VOLTS MAXIMUM. MATERIAL SHALL MEET THE APPLICABLE REQUIREMENTS OF ICEA STANDARD S-105-692 AND LISTED BY UL AS TYPE USE-2 FOR DIRECT BURIAL.

(D) LUMINAIRES: ALL LUMINAIRES SHALL BE SUPPLIED WITH HIGH POWER FACTOR BALLAST.

150 WATT LUMINAIRES: 150 WATT LUMINAIRES SHALL HAVE A HOUSING OF ALUMINUM ALLOY CASTING WITH AN INTEGRAL SLIPFITTER FOR A 2 INCH BRACKET MOUNTING WITH A NATURAL UNPAINTED ALZAK ALUMINUM REFLECTOR. THE BALLASTS SHALL BE A VOLTAGE AUTO-REGULATOR TYPE FOR HIGH PRESSURE SODIUM AT A VOLTAGE OF 480 VOLTS. THE REFRACTORS SHALL BE ACRYLIC WITH TYPE III LIGHT DISTRIBUTION.

(E) SODIUM LUMINAIRE IGNITORS: ALL HPS LUMINAIRES SHALL BE EQUIPPED WITH THE FOLLOWING:

HIGH PRESSURE SODIUM IGNITORS: ELECTRONIC IGNITOR FOR THE IGNITION OF HIGH PRESSURE SODIUM (HPS) VAPOR LAMPS. THE IGNITOR SHALL BE DESIGNED TO DIRECT THE HIGH VOLTAGE SPIKE DIRECTLY TO THE LAMP WITHOUT BEING DIRECTED TO THE LAMP THROUGH THE BALLAST WINDINGS. THE IGNITOR SHALL BE CAPABLE OF BEING USED WITH ALL BRANDS AND TYPES OF 60 HERTZ HPS BRANDS. THE IGNITOR SHALL BE DESIGNED SO THAT CYCLING OR EXTINGUISHED LAMP SHALL NOT ADVERSELY AFFECT THE IGNITOR OR BALLAST. THE IGNITOR SHALL BE BE TOTALLY EPOXY ENCAPSULATED IN A METAL CAN AND SHALL BE OPEN CIRCUIT TESTED FOR A PERIOD OF 48 HOURS WITH POWER APPLIED AT ELEVATED TEMPERATURES TO 100°C WITH CONSTANT MONITORING OF CASE TEMPERATURES. THE IGNITOR SHALL HAVE A TWO (2) MINUTE TIME DELAY. IF THE LAMP DOES NOT START, THE IGNITOR WILL BE SHUT OFF UNTIL THE POWER TO THE LUMINAIRES IS RESET BY THE PHOTO CONTROL. THE IGNITOR SHALL BE WARRANTED AGAINST DEFECTIVE MATERIALS AND WORKMANSHIP FOR A PERIOD OF NOT LESS THAN FIVE (5) YEARS FROM THE SHIPPING DATE.

(F) LAMPS: (a) 150 WATT LAMPS: 150 WATT LAMPS SHALL BE 16,000 LUMEN, HIGH PRESSURE SODIUM.

(G) ENCLOSURE: THE ENCLOSURE CABINET SHALL BE CONSTRUCTED OF 5052 ALLOY ALUMINUM 0.125" THICK. THE CABINET SHALL BE OF CLEAN CUT DESIGN HAVING NO SHARP EDGES, CORNERS OR PROJECTIONS. THE CIRCUITING SHALL BE SERVICEABLE BY MEANS OF A FULL LENGTH HINGED DOOR WITH PADLOCK PROVISIONS. THE ENCLOSURE SHALL HAVE 600 VOLT RATED MOLDED CASE MAIN AND SECONDARY BREAKERS, TWIST-LOCK PHOTO-CELL SOCKET, AND ELECTRICALLY HELD CONTACTORS. THE EQUIPMENT WITHIN THE ENCLOSURE SHALL BE WIRED PRIOR TO DELIVERY. THE ENCLOSURE SHALL HAVE A METER VIEW WINDOW AND A GLASS TO ALLOW LIGHT TO THE PHOTO-CELL.

(a) MAIN AND SECONDARY BREAKERS: THE MAIN AND SECONDARY BREAKERS SHALL HAVE A MOUNTING DIMENSIONS OF 1 3/8" WIDE X 4 1/2" HIGH MOUNTING HOLES SHALL BE POSITIONED TO ACCOMMODATE A BREAKER 2 3/4" WIDE SIDE BY SIDE. SEE ENCLOSURE DETAIL SHEET.

GENERAL MATERIALS AND NOTES:

(A) MISCELLANEOUS HARDWARE: MISCELLANEOUS HARDWARE THAT REQUIRES GALVANIZING OR ELECTROPLATING SHALL CONFORM TO THE STANDARD KANSAS DEPARTMENT OF TRANSPORTATION SPECIFICATIONS, LATEST EDITION, SECTION 1703.2 (c).

(B) METALLIC CONDUIT: METALLIC CONDUIT SHALL BE RIGID STEEL CONDUIT MEETING THE REQUIREMENTS OF AMERICAN STANDARD SPECIFICATION C-80.1. TRENCHING FOR CONDUIT WILL NOT BE PERMITTED THROUGH EXISTING PAVEMENT. JACKING WILL NOT BE PERMITTED IN DISTRICT ONE UNLESS APPROVED BY THE ENGINEER IN CHARGE OF CONSTRUCTION.

(C) METALLIC CONDUIT FITTINGS: METALLIC CONDUIT FITTINGS SHALL BE ZINC COATED AND SHALL MEET THE REQUIREMENTS OF AMERICAN STANDARD SPECIFICATION C-80.1.

(D) NON-METALLIC CONDUIT: D.1 RIGID POLYVINYL CHLORIDE MEETING THE REQUIREMENTS OF FEDERAL SPECIFICATIONS NO. LP 1036A, TYPE II, ELECTRICAL CLASS 2, GRADE C. EACH LENGTH SHALL BEAR THE UNDERWRITERS, INC. LABEL. NON-METALLIC CONDUIT FITTINGS SHALL BE FABRICATED FROM POLYVINYL CHLORIDE HAVING THE SAME CHEMICAL AND PHYSICAL PROPERITIES AS THE CONDUIT WITH WHICH IT IS TO BE USED. EACH SHALL BEAR THE UNDERWRITERS, INC. LABEL. THE JOINTS SHALL BE MADE IN ACCORDANCE WITH THE MANUFACTURERS RECOMMENDATIONS.

D.2. HIGH DENSITY POLYETHYLENE (HDPE) MEETING THE REQUIREMENTS OF ASTM F 2160 AND ASTM F 2176. A LETTER OF CERTIFICATION (LOC) WILL BE REQUIRED FROM THE CONDUIT PRODUCER AND/OR RESIN PRODUCER. THE CONDUIT WILL NEED TO BE MARKED WITH ASTM F 2160 DESIGNATION ON THE PRINT LINE.

(E) GROUND: GROUND WIRE SHALL BE A #6 AWG SOLID BARE COPPER WIRE AND ARRANGEMENT SHALL BE AS NOTED ON PLANS.

(F) ANCHOR BOLTS: ANCHOR BOLTS SHALL CONFORM TO THE STANDARD KANSAS DEPARTMENT OF TRANSPORTATION SPECIFICATIONS, LATEST EDITION, SECTION 1600 (GRADE 55), WITH THE FOLLOWING EXCEPTION. DO NOT USE CUT THREADS, USE ROLLED THREADS.

(G) BREAK-AWAY CABLE CONNECTORS: THE CONNECTORS SHALL PROVIDE A FUSED WATERPROOF WIRING CONNECTION THAT WHEN SUBJECTED TO STRAIN CONSISTANT WITH A KNOCKDOWN WILL SEPARATE WITHOUT DAMAGE TO WIRING. WHEN SEPARATION OCCURS, THE CONNECTORS SHALL HAVE NO CONTACTS EXPOSED TO PRESENT A SHOCK HAZARD. THE CABLE CONNECTORS SHALL MEET AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS (LATEST EDITION), ARTICLE 12.5.3 (C). CONNECTOR INSTALLATION SHALL BE AS DIRECTED BY THE MANUFACTURER.

(H) FRANGIBLE BASE: FRANGIBLE BASE SHALL BE CAST ALUMINUM TO MEET THE STANDARDS OF THE LATEST AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, AND TRAFFIC SIGNALS. THE FRANGIBLE BASE SHALL ACCOMMODATE UP TO A 15" BOTTOM BOLT CIRCLE. THE FRANGIBLE BASE SHALL HAVE HIGH DENSITY PLASTIC DOORS. THE COLOR SHALL BE PIGMENTED THROUGHOUT TO GIVE THE APPEARANCE OF ALUMINUM. THE DOORS SHALL WITHSTAND TEMPERATURE EXTREMES OF -70° TO 150° F. AND ARE COMPOUNDED WITH U.V. INHIBITORS.

(I) JUNCTION BOXES (IN-GROUND): THE JUNCTION BOX SHALL BE OF SUFFICIENT SIZE TO FACILITATE THE CONDUIT AND WIRING AS INDICATED IN THE PLANS. JUNCTION BOXES SHALL HAVE THE MINIMUM NOMINAL DIMENSIONS OF 12 INCHES DEEP WITH A MINIMUM PHYSICAL OPENING OF 130 SQUARE INCHES. IN-GROUND JUNCTION BOXES MAY BE CONSTRUCTED OF ONE OF THE FOLLOWING METHODS:

PRE-CAST CONCRETE WITH A CAST IRON COVER; POLYMER CONCRETE WITH A POLYMER CONCRETE COVER; FIBERGLASS REINFORCED POLYMER BODY WITH A POLYMER CONCRETE RING AND COVER; HIGH DENSITY POLYETHYLENE BODY WITH POLYMER CONCRETE RING AND COVER. THE RING SHALL BE SECURELY ATTACHED TO THE BODY.

ENCLOSURES, BOXES AND COVERS ARE REQUIRED TO CONFORM TO ALL TEST PROVISIONS OF ANSI/SCTE 77 2002 "SPECIFICATION FOR UNDERGROUND INTEGRITY" FOR TIER 15 APPLICATIONS.

THE COVER SHALL BEAR THE LOGO "LIGHTING" CLEARLY AND PERMANENTLY MOLDED OR ETCHED INTO THE COVER.

JUNCTION BOXES (ABOVE-GROUND): ABOVE GROUND JUNCTION BOXES SHALL HAVE THE NOMINAL DIMENSIONS OF 12 INCHES BY 12 INCHES BY 6 INCHES. THE JUNCTION BOX SHALL BE MADE OF 14 GAUGE SHEET METAL(STEEL) WITH WELDED SEAMS, KNOCKOUTS AND WEATHER PROOF SCREW COVER. BOXES SHALL BE HOT DIPPED GALVANIZED IN ACCORDANCE WITH ASTM A-123 AFTER FABRICATION.

(J) WEDGE TYPE STUD BOLT ANCHORS: THE CONTRACTOR SHALL INSTALL TWO 3/8" X 3" WEDGE TYPE ANCHORS FOR CONDUIT CLAMPS. THE ANCHORS SHALL BE WEDGE TYPE MADE FROM CARBON STEEL MEETING AISI 12L14 STEEL. THE MINIMUM EMBEDDED DEPTH SHALL BE 1 3/4".

(K) CONDUIT CLAMPS WITH CLAMP BACKS: THE CONTRACTOR SHALL INSTALL 2" CONDUIT CLAMPS WITH A COMPATIBLE CLAMP BACK. CLAMPS SHALL BE HEAVY DUTY STEEL TO SECURE THE 2" RIGID CONDUIT TO STRUCTURE. CONDUIT CLAMPS ARE TO BE SPACED AT 6' INTERVALS.

(L) CABLE GRIP SUPPORTS: THE CONTRACTOR SHALL INSTALL ONE (1) CABLE SUPPORT GRIP IN EACH ROADWAY LIGHTING POLE. THE CABLE SUPPORT GRIP SHALL BE MADE OF HIGH GRADE, NON-MAGNETIC TIN COATED BRONZE STRAND. THE CABLE SUPPORT GRIP SHALL BE CAPABLE OF SECURING TWO (2) #10 AWG TYPE USE-2 CABLES IN A VERTICAL POSITION HOLDING THE WEIGHT OF THE CABLES AND CABLE CONNECTORS OFF THE LUMINAIRE ASSEMBLY. SEE ROADWAY LIGHTING DETAILS SHEET.

(M) ALL WELDS SHALL BE SMOOTH CLEAN DENSE DEPOSIT THAT WILL EXCLUDE MOISTURE AND CONFORM TO A.W.S. SPECIFICATION D1.1 (LATEST REVISION). FIELD WELDS WILL NOT BE ALLOWED.

(N) GROUND RODS SHALL BE COPPER CLAD STEEL, 3/4" DIAMETER BY 10' LONG.

NO.	DATE	REVISIONS	BY	APP'D
3	5/7/13	Fixed Typo	C.E.	B.D.G.
2	2/27/13	Modified LGT STD & (F), Add (D) D.2, Typos	C.E.	B.D.G.
1	7/14/11	Modify Note (F) chg 250 to 150 watt	V.H.	B.D.G.

KANSAS DEPARTMENT OF TRANSPORTATION

CONSTRUCTION AND MATERIAL REQUIREMENTS FOR HIGHWAY LIGHTING (CONVENTIONAL)

TE207

FHWA APPROVAL		04/29/2013	APP'D	Brian D. Gower	
DESIGNED	V.H.	DETAILED	V.H.	QUANTITIES	TRACED
DESIGN CK.	C.A.L.	DETAIL CK.	C.A.L.	QUAN. CK.	TRACE CK.

KDOT Graphics Certified 05-07-2013

Sh. No. 54

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:53

Drawn By : cluedke
File : te207.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	55	99

SUMMARY OF ELECTRIC LIGHTING SYSTEM QUANTITIES (HARDWARE)

ITEM	UNIT	QUANTITY
SOLID STATE PHOTO CELL	EACH	
ENCLOSURE	EACH	
IN-GROUND JUNCTION BOX (12" X 12" X 6")	EACH	1
CABLE CONNECTORS	EACH	
OZ EXPANSION JOINT	EACH	
CABLE SUPPORT GRIP	EACH	
600 VOLT 2/C SECONDARY CABLE WITH GROUND WIRE (IN DUCT) #4 A.W.G.	LIN FT	
600 VOLT 2/C SECONDARY CABLE WITH GROUND WIRE (IN DUCT) #6 A.W.G.	LIN FT	
600 VOLT 1/C SECONDARY CABLE WITH GROUND WIRE #2 A.W.G.	LIN FT	525
600 VOLT 1/C SECONDARY CABLE WITH GROUND WIRE #8 A.W.G.	LIN FT	
600 VOLT 1/C SECONDARY CABLE #10 A.W.G.	LIN FT	
GROUNDWIRE, #6 A.W.G.SOLID BARE COPPER	LIN FT	
1" METALLIC CONDUIT	LIN FT	
2" METALLIC CONDUIT	LIN FT	
3" METALLIC CONDUIT	LIN FT	
3" 90 DEGREE PVC CONDUIT BENDS 24" RADIUS	EACH	
2" SCHEDULE 40 PVC	LIN FT	160
3" SCHEDULE 40 PVC	LIN FT	
3" SCHEDULE 80 PVC	LIN FT	
3" 90 DEGREE SCHEDULE 40 PVC CONDUIT BENDS 36" RADIUS	EACH	
2" 90 DEGREE SCHEDULE 40 PVC CONDUIT BENDS 24" RADIUS	EACH	5
3/8" x 3" WEDGE TYPE ANCHOR BOLTS	EACH	
ALUMINUM TRIPLEX #4 A.W.G. (AERIAL)	LIN FT	
2" CONDUIT CLAMPS W/ CLAMP BACKS	EACH	
GROUND RODS WITH CLAMPS 3/4" X 10'	EACH	

SUMMARY OF ELECTRIC LIGHTING SYSTEM QUANTITIES (LIGHT STANDARDS)

ITEM	UNIT	QUANTITY
◔ 11' Banner/Planter Post	EACH	10
40' MOUNTING HEIGHT LIGHT STANDARD WITH 10' MASTARM	EACH	
40' MOUNTING HEIGHT LIGHT STANDARD WITH 12' MASTARM	EACH	
40' MOUNTING HEIGHT LIGHT STANDARD WITH 15' MASTARM	EACH	
150 WATT LUMINAIRE, AUTO REGULATOR BALLAST, HIGH PRESSURE SODIUM	EACH	
150 WATT, 16,000 LUMEN, HIGH PRESSURE SODIUM, 24,000 LAMP HOUR	EACH	
FRANGIBLE BASE	EACH	
24" CONCRETE BASE FOR LIGHT STANDARD	EACH	
30" CONCRETE BASE FOR LIGHT STANDARD	EACH	

◔ See Sh. No. 34 for planter/hanger post details.

SUMMARY OF ELECTRIC LIGHTING SYSTEM QUANTITIES (MISCELLANEOUS ITEMS)

ITEM	UNIT	QUANTITY
TRENCH AND BACKFILL	LIN FT	
REMOVAL OF CONCRETE BASE	EACH	

THE FOLLOWING COMPANIES HAVE A RAISE AND LOWERING DEVICE THAT HAS BEEN ACCEPTED BY THE KANSAS DEPARTMENT OF TRANSPORTATION : AMERICAN HIGH MAST, EAGLE HIGH MAST LIGHTING CO., CAROLINA HIGH MAST, METROLUX AND HALOPHANE. ADDITIONAL COMPANIES DESIRING APPROVAL BY THE KDOT MUST INSTALL ONE RAISE AND LOWERING DEVISE ON A POLE PROVIDED BY THE KDOT IN TOPEKA, OR ON ANOTHER POLE AS APPROVED. THE INSTALLATION WILL BE REVIEWED AND TESTED BY KDOT PRIOR TO APPROVAL ON THIS PROJECT. ALL ASSOCIATED COSTS WILL BE THE RESPONSIBILITY OF THE MANUFACTURERS/CONTRACTORS.

✳ EACH TOWER SHALL BE INSPECTED BY A FACTORY REPRESENTATIVE OF THE RAISE AND LOWER DEVICE MANUFACTURER PRIOR TO FINAL ACCEPTANCE OF THE PROJECT BY THE KANSAS DEPARTMENT OF TRANSPORTATION.

NOTE: LINEAR QUANTITIES ARE APPROXIMATE AND INCLUDED FOR INFORMATION ONLY. THE CONTRACTOR SHALL CONDUCT ALL WORK, AND FURNISH AND INSTALL ALL EQUIPMENT NECESSARY, TO INSURE COMPLETE AND SATISFACTORY OPERATION OF THE ELECTRIC LIGHTING SYSTEM WHETHER SPECIFICALLY MENTIONED OR NOT.

SUMMARY OF ELECTRIC LIGHTING SYSTEM QUANTITIES (TOWER)

	1000 WATT LUMINAIRE, PEAK LEAD AUTO TRANSFORMER BALLAST, METAL HALIDE, ASYMMETRIC	EACH	
	1000 WATT LUMINAIRE, PEAK LEAD AUTO TRANSFORMER BALLAST, METAL HALIDE, SYMMETRIC	EACH	
	1000 WATT, 110,00 LUMEN, METAL HALIDE, VERTICAL BURNING, 12,000 HOUR LAMP	EACH	
✳	RAISE AND LOWER DEVICE	EACH	
	1/2" HEAVY DUTY REVERSIBLE DRILL	EACH	
	STEP DOWN TRANSFORMER	EACH	

BID ITEM

ITEM	UNIT	QUANTITY
ELECTRIC LIGHTING SYSTEM	L.S.	Lump Sum
HIGH MAST LIGHT TOWER (100')(36" CONCRETE BASE)	EACH	
HIGH MAST LIGHT TOWER (110')(36" CONCRETE BASE)	EACH	
HIGH MAST LIGHT TOWER (120')(42" CONCRETE BASE)	EACH	
HIGH MAST LIGHT TOWER (100')	EACH	
HIGH MAST LIGHT TOWER (110')	EACH	
HIGH MAST LIGHT TOWER (120')	EACH	

NOTE: FOR ADDITIONAL CONSTRUCTION QUANTITES, SEE THE FOLLOWING SHEETS, AS APPLICABLE:
1. CONSTUCTION AND MATERIAL REQUIREMENTS AND QUANTITIES FOR SIGN LIGHTING
2. UNDERDECK LIGHTING DETAIL.
3. FOOTING DETAILS & ENGINEERING GEOLOGY FOR FOOTING LENGTHS.

THE BID ITEM "ELECTRIC LIGHTING SYSTEM" INCLUDES ALL WORK PERSONS, TOOLS, EQUIPMENT AND NEW HARDWARE NECESSARY TO COMPLETE THE WORK.

ITEMS NOT SPECIFICALLY CALLED OUT ARE SUBSIDIARY TO OTHER BID ITEMS.

THE FOUR STREET LIGHT POLES AND LUMINAIRES WILL BE PROVIDED AND INSTALLED BY THE CITY OF MERRIAM.

THE TEN PLANTER/HANGER POSTS AND ALL NECESSARY HARDWARE WILL BE PROVIDED AND INSTALLED BY THE CONTRACTOR AND IS SUBSIDIARY TO THE BID ITEM "ELECTRIC LIGHTING SYSTEM".

NO.	DATE	REVISIONS	BY	APP'D
3	1/12/12	Chg'd Bid Items & Add 1" PVC Conduit	C.E.	B.D.G.
2	7/14/11	Change 250 Watts to 150 Watts	V.H.	B.D.G.
1	6/14/11	Changed 150 to 250 Watts	V.H.	B.D.G.

KANSAS DEPARTMENT OF TRANSPORTATION

CONSTRUCTION QUANTITIES FOR ELECTRIC LIGHTING SYSTEM

~~TE209~~

FHWA APPROVAL			12/15/2011	APP'D	Brian D. Gower
DESIGNED	V.H.	DETAILED	V.H.	QUANTITIES	TRACED
DESIGN CK.	B.D.G.	DETAIL CK.	B.D.G.	QUAN. CK.	TRACE CK.

Plotted : 13-SEP-2013 10:53

Drawn By : cluedke
File : te209.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	56	99

SYMBOL KEY

- REMOVE SIGN
- REMOVE POST
- REMOVE FOOTING
- REMOVE SIGN & POST
- REMOVE POST & FOOTING
- REMOVE SIGN, POST, & FOOTING
- MOUNT ON WOOD POST IN CONCRETE FOOTING
- MOUNT ON WOOD POST IN SOIL
- MOUNT ON STEEL BEAM BREAKAWAY POST
- MOUNT ON STEEL U-POST
- MOUNT ON PSST POST
- MOUNT ON PSST POST WITH COUPLER
- MOUNT ON PSST POST WITH COUPLER AND FOOTING
- MOUNT ON EXISTING POST
- MOUNT ON VERTICAL SUPPORT
- SHOULDER MOUNTED INSTALLATION
- OFFSET MOUNTED INSTALLATION
- EXISTING SIGN
- EXISTING SIGN TO BE OVERLAID
- SIGN IS NOT PART OF PROJECT
- TYPE 'A' DELINEATOR (RIGID)
- TYPE 'B' DELINEATOR (RIGID)
- TYPE 'A' DELINEATOR (FLEXIBLE)
- TYPE 'B' DELINEATOR (FLEXIBLE)
- TYPE 2 OBJECT MARKER
- TYPE 3 OBJECT MARKER

GENERAL NOTES

IN ORDER TO EXPEDITE THE COMPLETION OF THE PROJECT FOR TRAFFIC SERVICE, THE SIGNING AND DELINEATOR WORK SHALL BE SEQUENCED WITH ANY OTHER CONTRACT WORK SUCH THAT THE PHASES OF CONSTRUCTION MAY PROCEED AND BE COMPLETED AT THE SAME TIME.

NEW SIGNS ERECTED ON THE PROJECT WHICH ARE IN CONFLICT WITH EXISTING SIGNING ARE TO BE COMPLETELY COVERED UNTIL THE EXISTING SIGNS ARE REMOVED OR THE NEW SIGNING IS APPLICABLE. THE EXISTING SIGNS THAT ARE BEING REPLACED, REMOVED, OR DO NOT FOLLOW THE CURRENT MUTCD SIGNING STANDARDS ARE TO BE REMOVED WHEN THE PROJECT IS COMPLETED OR AS DETERMINED BY THE ENGINEER.

THE CONTRACTOR SHALL EXERCISE CAUTION AT ALL TIMES WHEN INSTALLING SIGN SUPPORTS IN AND AROUND AREAS WHERE UTILITIES EXIST, EITHER UNDERGROUND OR OVERHEAD, AND WILL BE HELD RESPONSIBLE FOR ANY DAMAGE INCURRED TO THE SYSTEM. THE INSTALLATION OF SIGN SUPPORTS SHALL INCLUDE THE EXCAVATION, DRILLING, OR DRIVING THE SUPPORT FOOTING AND THE ERECTION OF THE SIGN SUPPORT. THE CONTRACTOR SHALL EXERCISE CAUTION WHEN WORKING AROUND ANY EXISTING SIGNS THAT ARE TO REMAIN AND WILL BE HELD RESPONSIBLE FOR ANY DAMAGE TO THE SIGNS, SUPPORTS, OR FOOTINGS. THE CONTRACTOR SHALL EXERCISE CARE WHEN WORKING AROUND SHRUBBERY WHILE REMOVING OR INSTALLING SIGNS OR SIGN SUPPORTS.

AN EXISTING SIGN POST INSTALLATION SHALL BE PLUMB AND THE COMPACTION OF THE BACKFILL SOIL SHALL COMPLY WITH THE SPECIFICATIONS AFTER THE REMOVAL AND RESETTING OF A SIGN, THE REMOVAL AND REPLACEMENT OF A SIGN, OR THE INSTALLATION OF A NEW SIGN.

THE CONTRACTOR SHALL PROVIDE MOUNTING BOLTS THAT ARE OF A LENGTH THAT DOES NOT EXTEND MORE THAN A NOMINAL 1 INCH BEYOND THE SIGN POST. THE CONTRACTOR SHALL NOT MAKE ANY FIELD MODIFICATIONS TO THE MOUNTING BOLT PRIOR TO OR AFTER THE SIGN IS INSTALLED.

SPECIFIC SERVICE, LOGO, SIGNS THAT ARE TO BE REMOVED SHALL HAVE THE BUSINESS LOGO PLAQUES REMOVED AND TRANSPORTED TO LOCATION DETERMINED BY KDOT, AT WHICH TIME THE PLAQUES BECOME THE PROPERTY OF KDOT. THE CONTRACTOR WILL BE ASSESSED A REPLACEMENT COST FOR ANY DAMAGE TO A BUSINESS LOGO PLAQUE PRIOR TO THE PLAQUE BECOMING THE PROPERTY OF KDOT.

THE MATERIALS AND FABRICATION FOR SIGNING AND DELINEATION WORK SHALL CONFORM TO THE STANDARD SPECIFICATIONS FOR STATE ROAD AND BRIDGE CONSTRUCTION (2007 EDITION) AND SPECIAL PROVISIONS.

INDEX OF SHEETS

56 SIGNING INDEX, SYMBOLS, & GENERAL NOTES
57 HEIGHT & LATERAL DISTANCE FOR ERECTION
58 PLAN SHEETS (INSTALLATIONS)
59 PLAN SHEETS (REMOVALS)
60 QUANTITIES SHEETS (INSTALLATIONS)
61 SUMMARY SHEET (INSTALLATIONS & REMOVALS)
62 SUMMARY SHEET (REMOVAL & RESET)
63 RECAPITULATION SHEET
64 STANDARD STRUCTURAL SIGN SUPPORTS (WOOD & STEEL POSTS)
65-66 DETAILS FOR FLAT SHEET SIGN BLANKS
67 DETAILED SIGN SPECIFICATIONS

NO.	DATE	REVISIONS	BY	APP'D
1	7/23/10	Changed General Notes and Spec Book Date	D.D.G.	D.B.

KANSAS DEPARTMENT OF TRANSPORTATION

SIGNING SYMBOL KEY
GENERAL NOTES
AND INDEX

TE402 7/1/03

FHWA APPROVAL			7/23/2010	APP'D	Steven A. Buckley		
DESIGNED	D.D.G.	DETAILED	W.S.B.	QUANTITIES		TRACED	
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.		TRACE CK.	

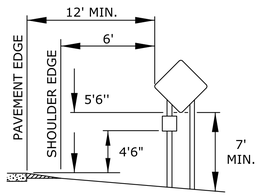
KDOT Graphics Certified 06-03-2013

Sh. No. 56

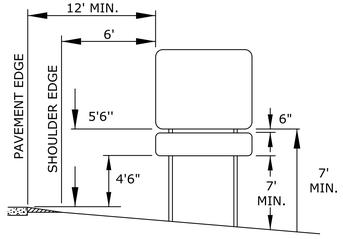
Plotted : 13-SEP-2013 10:53
Drawn By : cluedke
File : TE402.dgn

KDOT Graphics Certified

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	57	99

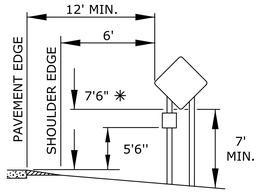


SHOULDER MOUNT



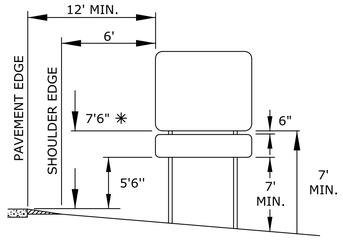
SHOULDER MOUNT

CONVENTIONAL ROADWAY



* - 8'6" WITH SECONDARY SIGN

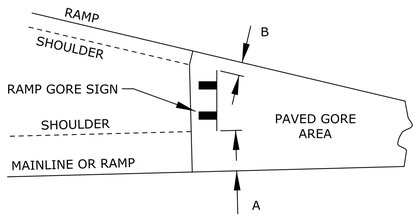
SHOULDER MOUNT



* - 8'6" WITH SECONDARY SIGN

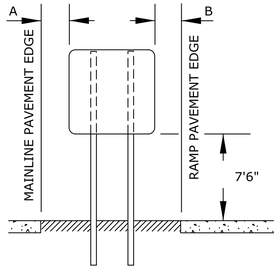
SHOULDER MOUNT

FREEWAY AND EXPRESSWAY ROADWAY
RAMPS AND SIDE ROADS

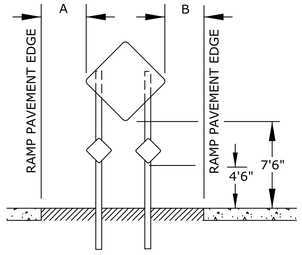


GORE TYPE	A (FT)	B (FT)
MAINLINE/ RAMP	6	3
RAMP/RAMP	3	3

(DIMENSIONS ARE MINIMUMS)



GORE INSTALLED SIGN
(MAINLINE/RAMP)

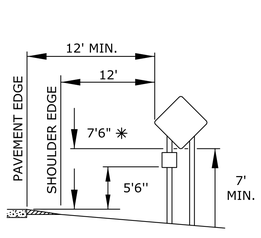


GORE INSTALLED SIGN
WITH TWO T1/OM'S
(RAMP/RAMP)

NOTES:
THE OUTER EDGE OF THE SIGN, ON EXPRESSWAYS AND FREEWAYS, SHALL BE A MINIMUM OF 10 FEET FROM THE RIGHT OF WAY LINE.
IN BUSINESS, COMMERCIAL, OR RESIDENTIAL DISTRICTS WHERE LATERAL OFFSETS ARE LIMITED, A MINIMUM LATERAL CLEARANCE OF 2 FEET WITH A 7'6" MINIMUM MOUNTING HEIGHT MAY BE USED.
WHEN SIGNS ARE MOUNTED BEHIND GUARD FENCE, THE NEAR EDGE OF THE SIGN SHALL NOT EXTEND BEYOND THE BACK SIDE OF THE GUARD FENCE AND THE NEAREST SIGN POST SHALL BE A MINIMUM OF 5 FEET FROM THE FACE OF THE GUARD FENCE. THERE SHALL NOT BE ANY SHOULDER MOUNTED SIGNS LOCATED BETWEEN 100 FEET IN ADVANCE OF AND 50 FEET BEYOND THE NOSE OF THE GUARD FENCE.
WHEN SIGNS ARE MOUNTED IN A MEDIAN, THE LATERAL PLACEMENT SHOULD BE THE SAME AS A SHOULDER MOUNT. IF THE MEDIAN IS TOO NARROW FOR THIS PLACEMENT THE SIGN MAY BE PLACED A MINIMUM OF 2 FEET FROM THE BACK OF THE CURB, BUT IN NO CASE SHALL THE SIGN EDGE EXTEND BEYOND THE BACK EDGE OF THE CURB. SIGNS LOCATED AT THE MEDIAN NOSE SHOULD BE SET THE SAME DISTANCE FROM THE BACK OF THE CURB AS THE RADIUS OF THE MEDIA NOSE, BUT SHOULD NOT EXCEED THE DISTANCE OF THE SHOULDER MOUNT OR BE CLOSER THAN 2 FEET FROM THE BACK OF THE CURB.

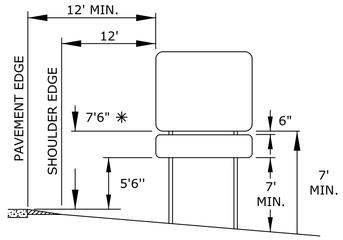
THE GORE SIGN SHALL BE INSTALLED IN THE FOOTING BLOCKOUT IN THE PAVED GORE AREA. IF NO BLOCKOUT IS PROVIDED, THEN LOCATE THE GORE SIGN AT THE PLAN STATION. THE EDGES OF THE GORE SIGN SHALL NOT EXTEND BEYOND THE SHOULDER EDGE. THE MINIMUM DISTANCE FROM THE POST CENTERLINE TO THE BACK EDGE OF THE PAVED GORE AREA IS 3 FEET.

ADJUSTMENTS:
SIGNS MAY BE MOVED LATERALLY OR LONGITUDINALLY IF IT WILL IMPROVE THE VISIBILITY OF THE SIGN OR OTHER SIGNS AND IF IT WILL PROTECT THE SIGN MORE.
THE MAXIMUM ALLOWABLE LONGITUDINAL ADJUSTMENTS OF SIGNS ARE:
ADVANCE GUIDE - 1320 FEET
SUPPLEMENTAL GUIDE - 1320 FEET
MOTORIST SERVICE - 1320 FEET
EXIT DIRECTION - 100 FEET
MILEAGE - 2640 FEET
MERGE OR ANY SIGNS IN AN INTERCHANGE - 50 FEET
MILEPOST - 50 FEET
IF ANY SIGN WITH A DISTANCE IS LONGITUDINALLY ADJUSTED, THE DISTANCE TO THE DESTINATION SHALL BE CHECKED AND MODIFIED AS NEEDED.
THE MINIMUM SPACING BETWEEN GUIDE SIGNS ON AN EXPRESSWAY OR FREEWAY IS 800 FEET.
THE MINIMUM SPACING BETWEEN SIGNS ON A RAMP OR CONVENTIONAL ROADWAY IS 100 FEET.



* - 8'6" WITH SECONDARY SIGN

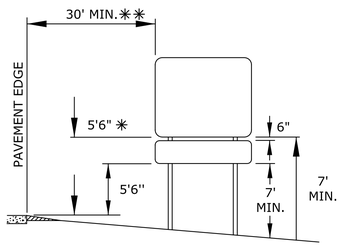
SHOULDER MOUNT



* - 8'6" WITH SECONDARY SIGN

SHOULDER MOUNT

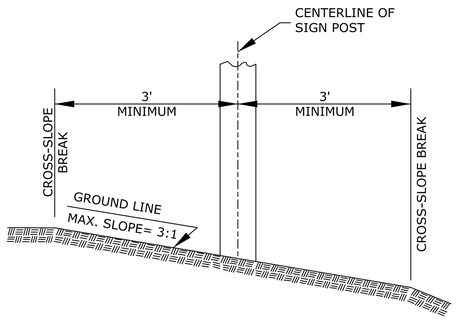
FREEWAY AND EXPRESSWAY ROADWAY



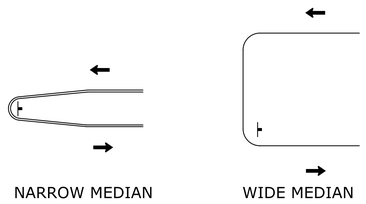
* - 8'6" WITH SECONDARY SIGN
** - 60' MAX.

OFFSET MOUNT

FREEWAY AND EXPRESSWAY ROADWAY

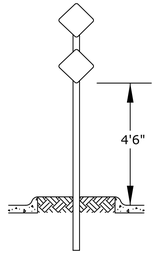


POST PLACEMENT CRITERIA
(CROSS-SLOPE BREAK)

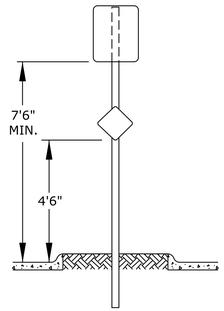


NARROW MEDIAN WIDE MEDIAN

TYPICAL MEDIAN SIGN LOCATION



ONE OR TWO OM1-3



PRIMARY SIGN WITH OM1-3

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

DETAILS FOR MOUNTING HEIGHTS LATERAL OFFSETS AND LONGITUDINAL ADJUSTMENTS

TE406 7/1/03

FHWA APPROVAL			7/22/2003	APP'D	Steven A. Buckley		
DESIGNED	D.D.G.	DETAILED	W.S.B.	QUANTITIES		TRACED	
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.		TRACE CK.	

KDOT Graphics Certified 01-18-2011

Sh. No. 57

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:53

Drawn By : cluedke
File : TE406.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	58	99

SCALE
20' 0 20' 40'

MERRIAM DR

JOHNSON DR

CAMPBELL LN

2 3 4 55 6 7

100

RR

Install R3-7L/D1-2
Sta. 52+94

Install D3/D3/R1-1
Sta. 52+45

Install SP-1/SP-2
Sta. 52+59

CAMPBELL LN D3
JOHNSON DR D3
STOP R1-1

D1-2

R3-7L

SP-2

SP-1

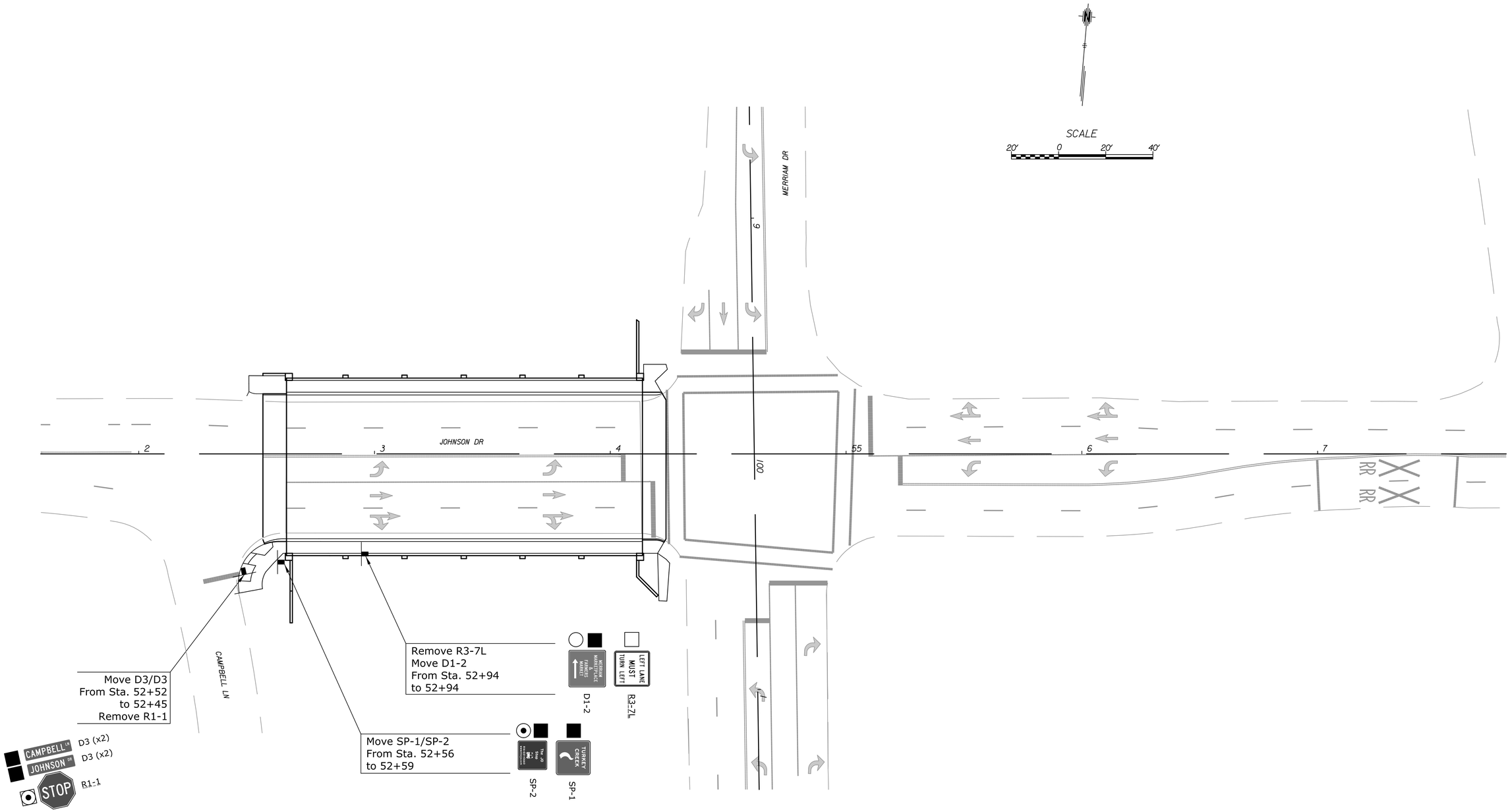
DATE	BY	REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke Plotted :9/13/2013
File : G:\12664\Micro\Traffic\12664_5201.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

SIGNING PLAN

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	59	99



REFERENCES NOTED	BY	DATE
REFERENCES CHECKED		

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\Micro\Traffic\12664_5202.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

EXISTING SIGNING PLAN

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	60	99

QUANTITIES SHEET

SIGNS TO BE INSTALLED ON PROJECT

QUANTITY	SIGN NO.	SIGN SIZE	TYPE OF SIGN			DESIGN DETAILS (AS REQUIRED)
			FLAT SHEET	REINFORCED PANEL	OVERLAY	
1	R1-1	36 X 36	X			
1	R3-7L	30 X 30	X			

QUANTITY	SIGN NO.	SIGN SIZE	TYPE OF SIGN			DESIGN DETAILS (AS REQUIRED)
			FLAT SHEET	REINFORCED PANEL	OVERLAY	

NOTE:
SEE STANDARD PLAN SHEET TE590
FOR DETAILED SPECIFICATIONS.

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

QUANTITIES SHEET
SIGN INSTALLATIONS

TE433 7/1/03

FHWA APPROVAL		7/22/2003	APP'D	Steven A. Buckley			
DESIGNED	D.D.G.	DETAILED	K.D.S.	QUANTITIES		TRACED	
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.		TRACE CK.	

KDOT Graphics Certified 06-03-2013

Sh. No. 60

KDOT Graphics Certified

Drawn By : cluedke
File : TE433.dgn

Plotted : 13-SEP-2013 10:54

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	61	99

SUMMARY OF QUANTITIES

SIGNS

TYPE	NUMBER	SQUARE FEET
FLAT SHEET	2	15.25
REINFORCED PANEL		
OVERLAY		

OBJECT MARKERS

TYPE	NUMBER
TYPE 2 ("U" POST)	
TYPE 3 ("U" POST)	

SIGN STRUCTURES

TYPE	NEW	MODIFIED
OVERHEAD STRUCTURE		
CANTILEVER STRUCTURE		
BUTTERFLY STRUCTURE		
BRIDGE MOUNT ATTACHMENT		
MAST ARM SIGN SUPPORT		

DELINEATORS

TYPE	FLEXIBLE DELINEATOR		RIGID DELINEATOR	
	TYPE I ANCHOR	TYPE III ANCHOR	"U" POST	BRACKET MOUNT
TYPE 'A' WHITE				
TYPE 'A' YELLOW				
TYPE 'B' WHITE				
TYPE 'B' YELLOW				
TYPE 'A' WHITE (BACK TO BACK)				
TYPE 'A' YELLOW (BACK TO BACK)				

REMOVALS

TYPE	NUMBER
SIGNS	2
POSTS	3
FOOTINGS	2
SIGN STRUCTURES	

POSTS AND ALUMINUM BEAMS

	4" x 6" POST				"U" POST		GALVANIZED STEEL BEAM POST						PERFORATED SQUARE STEEL TUBE (PSST)			
	WOOD		STEEL				W6x9		W10x12		W10x22					
	FLAT SHEET SIGN	REINFORCED PANEL SIGN	STRUCTURAL TUBING	3I2.25 ALUMINUM BEAM	2 LBS/FT	3 LBS/FT	A36 STEEL	A572 STEEL (ALT)	A36 STEEL	A572 STEEL (ALT)	A36 STEEL	A572 STEEL (ALT)	1-3/4"	2"	2-1/4"	2-1/2"
NUMBER																3
FEET																39

POST FOOTINGS AND PSST COUPLERS

	CONCRETE FOOTING (DIA.)					PERFORATED SQUARE STEEL TUBE (PSST)					
	WOOD	A36 STEEL		A572 STEEL (ALT)		PERFORATED SQUARE STEEL TUBE FOOTING				COUPLER	COUPLER WITH FOOTING
	18"	24"	30"	24"	30"	1-3/4"	2"	2-1/4"	2-1/2"		
NUMBER									2		
FEET											

BASE PLATES AND STUB POSTS

	W6x9		W10x12		W10x22	
	A36 STEEL	A572 STEEL (ALT)	A36 STEEL	A572 STEEL (ALT)	A36 STEEL	A572 STEEL (ALT)
BREAKAWAY BASES						
BASE PLATE (TOP)						
STUB POST WITH BASE PLATE						
NON-BREAKAWAY BASES						
BASE PLATE						

NUMBER & LENGTHS OF POSTS & ALUMINUM BEAMS (INFORMATION ONLY)

LENGTH OF POST OR BEAM	4" x 6" POST				"U" POST		GALVANIZED STEEL BEAM POST						PERFORATED SQUARE STEEL TUBE (PSST)			
	WOOD		STEEL				W6x9		W10x12		W10x22					
	FLAT SHEET SIGN	REINFORCED PANEL SIGN	STRUCTURAL TUBING	3I2.25 ALUMINUM BEAM	2 LBS/FT	3 LBS/FT	A36 STEEL	A572 STEEL (ALT)	A36 STEEL	A572 STEEL (ALT)	A36 STEEL	A572 STEEL (ALT)	1-3/4"	2"	2-1/4"	2-1/2"
2.1' - 4'																
4.1' - 6'																
6.1' - 8'																
8.1' - 10'																
10.1' - 12'																
12.1' - 14'																3
14.1' - 16'																
16.1' - 18'																
18.1' - 20'																
20.1' - 22'																
22.1' - 24'																
24.1' - 26'																
26.1' - 28'																
28.1' - 30'																
30.1' - 32'																

NO.	DATE	REVISIONS	BY	APP'D
1	7/23/10	Revised Tables	D.D.G.	D.B.

KANSAS DEPARTMENT OF TRANSPORTATION

SUMMARY OF QUANTITIES
FOR
INSTALLATIONS AND REMOVALS

TE439 7/1/03

FHWA APPROVAL		7/23/2010	APP'D	Steven A. Buckley	
DESIGNED	D.D.G.	DETAILED	K.D.S.	QUANTITIES	TRACED
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.	TRACE CK.

Plotted : 13-SEP-2013 10:54
Drawn By : cluedke
File : TE439.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	62	99

SUMMARY OF QUANTITIES

REMOVAL AND RESETTING OF SIGNS ON PROJECT

EXISTING PLAN STATION NUMBER	NEW PLAN STATION NUMBER	SIGN TYPE	SIGN SIZE
52+52	52+45	D3	42 X 9
52+52	52+45	D3	42 X 9
52+52	52+45	D3	42 X 9
52+52	52+45	D3	42 X 9
52+56	52+59	SP-1	18 X 18
52+56	52+59	SP-2	18 X 18
52+94	52+94	D2-1	24 X 24

EXISTING PLAN STATION NUMBER	NEW PLAN STATION NUMBER	SIGN TYPE	SIGN SIZE

EXISTING PLAN STATION NUMBER	NEW PLAN STATION NUMBER	SIGN TYPE	SIGN SIZE

EXISTING PLAN STATION NUMBER	NEW PLAN STATION NUMBER	SIGN TYPE	SIGN SIZE

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

SUMMARY OF QUANTITIES
FOR REMOVAL AND RESETTING
OF SIGNS

TE445 7/1/03

FHWA APPROVAL		7/22/2003	APP'D	Steven A. Buckley	
DESIGNED	D.D.G.	DETAILED	K.D.S.	QUANTITIES	TRACED
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.	TRACE CK.

Sh. No. 62

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : TE445.dgn

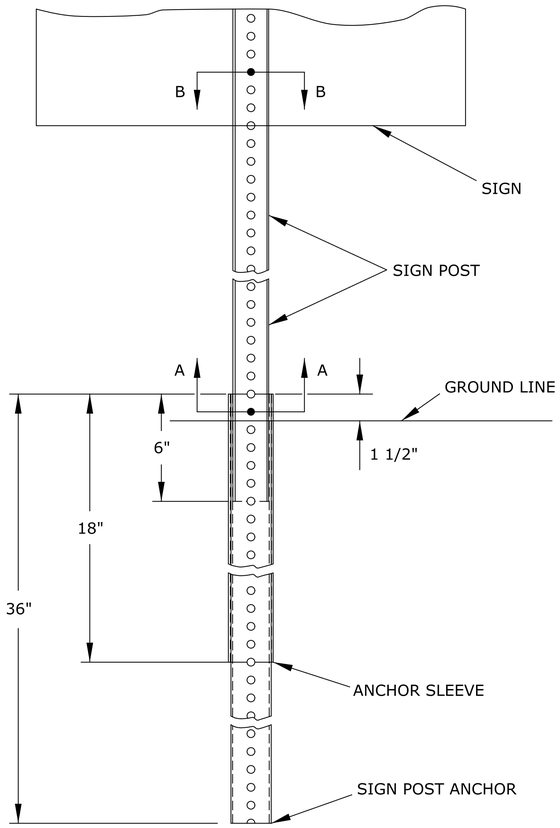
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	63	99

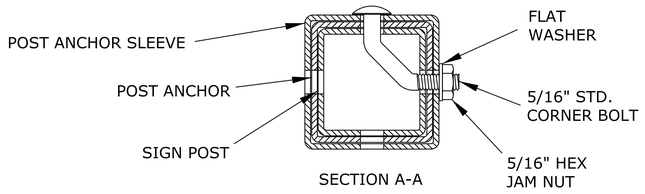
RECAPITULATION OF SIGNING & DELINEATION BID ITEMS

BID ITEMS	APPROXIMATE QUANTITIES		UNITS
SIGN (FLAT SHEET) (HIGH PERFORMANCE)	15.25		SQUARE FOOT
SIGN (REINFORCED PANEL) (HIGH PERFORMANCE)			SQUARE FOOT
SIGN (OVERLAY) (HIGH PERFORMANCE)			SQUARE FOOT
SIGN POST (4" x 6" WOOD) (FLAT SHEET SIGN)			LINEAR FOOT
SIGN POST (4" x 6" WOOD) (REINFORCED PANEL SIGN)			LINEAR FOOT
SIGN POST (2 LB/FT "U" STEEL)			LINEAR FOOT
SIGN POST (3 LB/FT "U" STEEL)			LINEAR FOOT
SIGN POST (1-3/4" PERFORATED SQUARE STEEL TUBE)			LINEAR FOOT
SIGN POST (2" PERFORATED SQUARE STEEL TUBE)			LINEAR FOOT
SIGN POST (2-1/4" PERFORATED SQUARE STEEL TUBE)			LINEAR FOOT
SIGN POST (2-1/2" PERFORATED SQUARE STEEL TUBE)	39		LINEAR FOOT
SIGN POST (4" X 6" STRUCTURAL STEEL)			LINEAR FOOT
SIGN POST (3 I 2.25 ALUMINUM)			LINEAR FOOT
	A36	A572(ALT)	
SIGN POST (W6X9 STEEL BEAM)			LINEAR FOOT
SIGN POST (W10X12 STEEL BEAM)			LINEAR FOOT
SIGN POST (W10X22 STEEL BEAM)			LINEAR FOOT
SIGN POST STUB WITH BREAKAWAY BASE PLATE (W6X9)			EACH
SIGN POST STUB WITH BREAKAWAY BASE PLATE (W10X12)			EACH
SIGN POST STUB WITH BREAKAWAY BASE PLATE (W10X22)			EACH
SIGN POST BREAKAWAY BASE PLATE (W6X9)			EACH
SIGN POST BREAKAWAY BASE PLATE (W10X12)			EACH
SIGN POST BREAKAWAY BASE PLATE (W10X22)			EACH
SIGN POST FOOTING (24" Dia. CONCRETE)(STEEL BEAM POST)			LINEAR FOOT
SIGN POST FOOTING (30" Dia. CONCRETE)(STEEL BEAM POST)			LINEAR FOOT
SIGN POST FOOTING (18" Dia. CONCRETE)(WOOD POST)			LINEAR FOOT
SIGN POST FOOTING (1-3/4" PERFORATED SQUARE STEEL TUBE)			EACH
SIGN POST FOOTING (2" PERFORATED SQUARE STEEL TUBE)			EACH
SIGN POST FOOTING (2-1/4" PERFORATED SQUARE STEEL TUBE)			EACH
SIGN POST FOOTING (2-1/2" PERFORATED SQUARE STEEL TUBE)	2		EACH
SIGN POST FOOTING (SIGN POST SQUARE COUPLER) (2-1/4")			EACH
SIGN POST SQUARE COUPLER (2-1/4")			
SIGNING OBJECT MARKER (TYPE 2)			EACH
SIGNING OBJECT MARKER (TYPE 3)			EACH
SIGNING DELINEATOR (TYPE A)(WHITE RIGID, "U" POST)			EACH
SIGNING DELINEATOR (TYPE A)(YELLOW RIGID, "U" POST)			EACH
SIGNING DELINEATOR (TYPE B)(WHITE RIGID, "U" POST)			EACH
SIGNING DELINEATOR (TYPE B)(YELLOW RIGID, "U" POST)			EACH
SIGNING DELINEATOR (TYPE A)(WHITE FLEXIBLE)(TYPE I ANCHOR)			EACH
SIGNING DELINEATOR (TYPE A)(YELLOW FLEXIBLE)(TYPE I ANCHOR)			EACH
SIGNING DELINEATOR (TYPE B)(WHITE FLEXIBLE)(TYPE I ANCHOR)			EACH
SIGNING DELINEATOR (TYPE B)(YELLOW FLEXIBLE)(TYPE I ANCHOR)			EACH
SIGNING DELINEATOR (TYPE A)(WHITE FLEXIBLE)(TYPE 3 ANCHOR)			EACH
SIGNING DELINEATOR (TYPE A)(YELLOW FLEXIBLE)(TYPE 3 ANCHOR)			EACH
SIGNING DELINEATOR (TYPE B)(WHITE FLEXIBLE)(TYPE 3 ANCHOR)			EACH
SIGNING DELINEATOR (TYPE B)(YELLOW FLEXIBLE)(TYPE 3 ANCHOR)			EACH

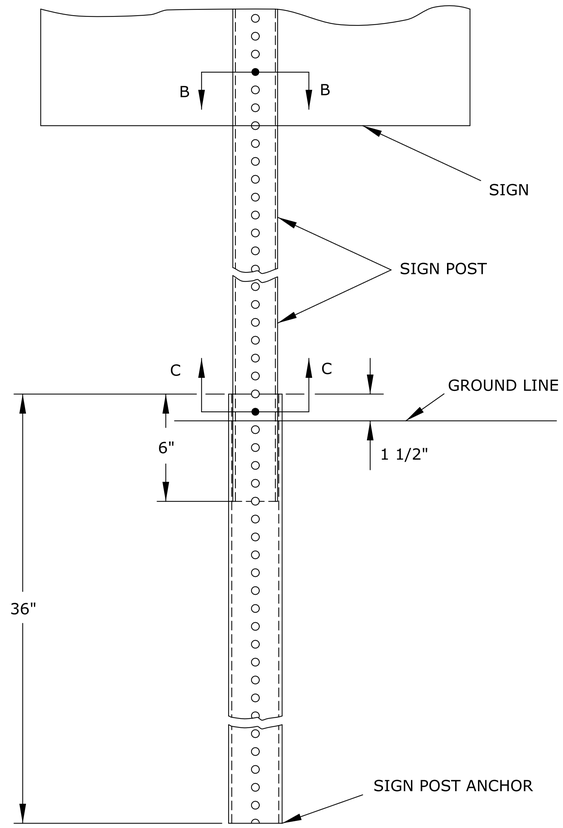
BID ITEMS	APPROXIMATE QUANTITIES	UNITS
SIGN (REMOVE AND RESET)	LUMP SUM	LUMP SUM

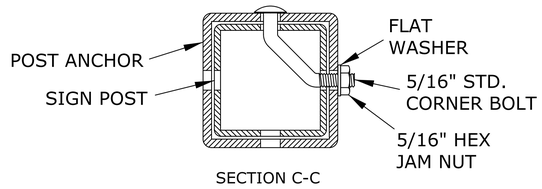
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	64	99





DETAILS FOR 1 3/4", 2", OR 2 1/4" SIGN POST





DETAILS FOR 2 1/2" SIGN POST

MATERIAL REQUIREMENTS

SIGN POSTS, POST ANCHORS, AND POST ANCHOR SLEEVES:
THE SIGN POSTS, POST ANCHORS, AND POST ANCHOR SLEEVES SHALL CONFORM TO (1) HOT ROLLED CARBON STEEL ASTM A-570 GRADE 33 OR ASTM STRUCTURAL QUALITY C 1021 WITH A HOT-DIPPED GALVANIZING CONFORMING TO AASHTO M-120 WITH AN INLINE SYSTEM FOLLOWED BY A CHROMATE CONVERSION AND A CROSS-LINKED POLYURETHANE ACRYLIC EXTERIOR COATING OR (2) A GALVANIZED FINISH COLD ROLLED CARBON STEEL ASTM A-446 GRADE A WITH A GALVANIZING CONFORMING TO ASTM A-525, DESIGNATION G-90.

THE SIGN POST SHALL HAVE 7/16" DIA. HOLES OR 7/16" DIA. EMBOSSED RING (DIE-CUT KNOCKOUTS) AT 1" CENTERS ON ALL FOUR SIDES FOR TOTAL LENGTH OF POST.

THE POST ANCHOR AND POST ANCHOR SLEEVE MAY BE FURNISHED WITHOUT THE BOLT HOLES ON ALL FOUR SIDES WITH THE EXCEPTION OF THE HOLES IMMEDIATELY ABOVE THE GROUND LINE.

ALL HOLES SHALL BE DRILLED OR PUNCHED AND ALL WELDS, CUTS, BURRS, AND SHARP EDGES ARE TO BE SMOOTHED OFF BEFORE APPLICATION OF FINISH.

FASTENING ACCESSORIES:
THE BOLTS, NUTS, AND WASHERS SHALL COMPLY WITH SUBSECTION 1614 OF THE STANDARD SPECIFICATIONS FOR STATE ROAD AND BRIDGE CONSTRUCTION (1990 EDITION).

THE BOLTS, NUTS, AND WASHERS SHALL BE BID SUBSIDIARY TO THE BID ITEM 'SQUARE STEEL POST'.

BASIS OF ACCEPTANCE:
ACCEPTANCE OF THE STEEL POSTS, ANCHORS, SLEEVES, AND FASTENERS FURNISHED TO COMPLY WITH THE ABOVE REQUIREMENTS WILL BE BASED ON RECEIPT AND APPROVAL OF A TYPE D CERTIFICATION AS DESCRIBED IN SECTION 2600 OF THE STANDARD SPECIFICATIONS FOR STATE ROAD AND BRIDGE CONSTRUCTION (1990 EDITION) AND VISUAL INSPECTION FOR CONDITION AND DIMENSIONAL REQUIREMENTS.

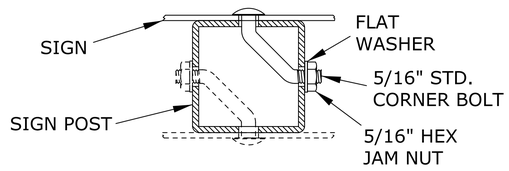
INSTALLATION SEQUENCE

1. DRIVE POST ANCHOR INTO SUBGRADE.
2. DRIVE POST ANCHOR SLEEVE (IF REQUIRED) INTO SUBGRADE OVER THE POST ANCHOR.
3. INSTALL SIGN POST INTO THE POST ANCHOR.

NOTE:
IN ALL INSTALLATIONS, THE FIRST HOLE ABOVE THE GROUND LINE ON THE SIGN POST, POST ANCHOR, AND POST ANCHOR SLEEVE (IF REQUIRED) MUST BE IN LINE FOR INSERTION OF THE CORNER BOLT.

MATERIALS TABLE FOR SIGN POST AND FOOTING		
SIGN POST 12 GA. OR 14 GA.	FOOTING	
	POST ANCHOR	POST ANCHOR SLEEVE
1 3/4 " X 1 3/4 "	2" X 2" X 12 GA.	2 1/4" X 2 1/4" X 12 GA.
2" X 2"	2 1/4" X 2 1/4" X 12 GA.	2 1/2" X 2 1/2" X 12 GA.
2 1/4" X 2 1/4"	2 1/2" X 2 1/2" X 12 GA.	3" X 3" X 7 GA.
2 1/2" X 2 1/2"	3" X 3" X 7 GA.	NOT REQUIRED

NOTE: 14 GA. POSTS MUST MEET A CERTIFIED MINIMUM YIELD STRENGTH OF 60,000 P.S.I.



NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

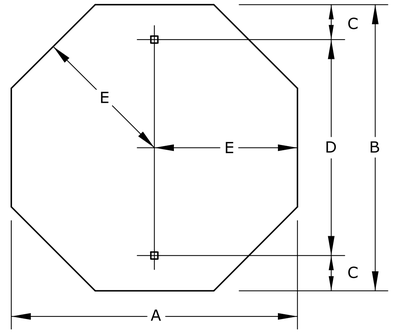
DETAILS FOR PERFORATED SQUARE STEEL TUBE POSTS (PSST)

TE466 7/1/03

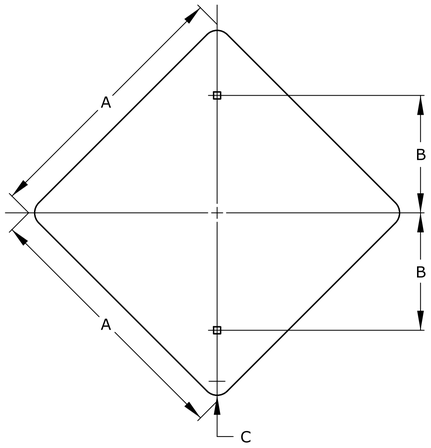
FHWA APPROVAL		7/22/2003	APP'D	Steven A. Buckley	
DESIGNED	D.D.G.	DETAILED	A.A.D.	QUANTITIES	TRACED
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.	TRACE CK.

Plotted : 13-SEP-2013 10:54
Drawn By : cluedke
File : TE466.dgn

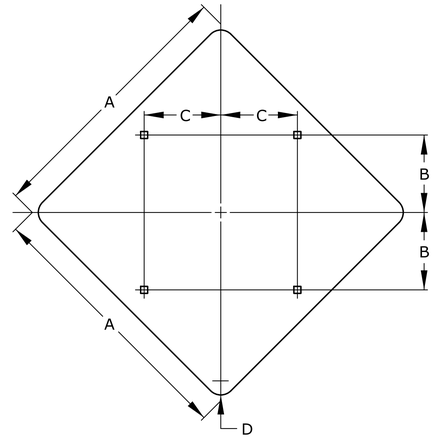
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	65	99



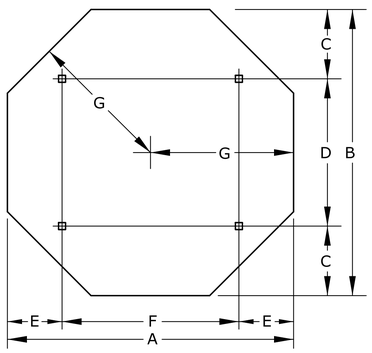
SIGN SIZE	A	B	C	D	E	T	AREA
30 X 30	30	30	3	24	15	0.080	5.18
36 X 36	36	36	6	24	18	0.080	7.46



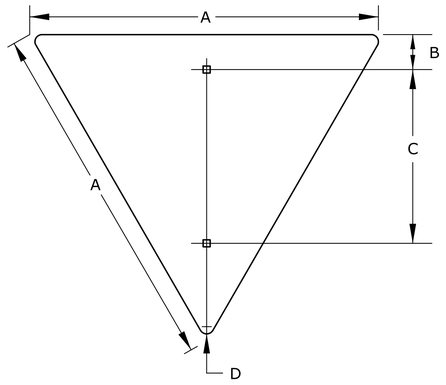
	SIGN SIZE	A	B	C	T	AREA
(1)	5 X 5	5	2 1/2	3/8	0.040	0.17
	18 X 18	18	6	1 1/2	0.080	2.25
	24 X 24	24	12	1 1/2	0.080	4.00
	30 X 30	30	12	1 7/8	0.080	6.25
(2)	36 X 36	36	18	2 1/4	0.080	9.00



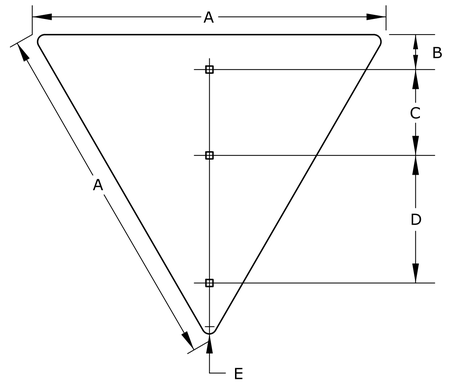
SIGN SIZE	A	B	C	D	T	AREA
48 X 48	48	12	15	3	2 1/2	16.00



SIGN SIZE	A	B	C	D	E	F	G	T	AREA
48 X 48	48	48	12	24	9	30	24	2 1/2	13.25



SIGN SIZE	A	B	C	D	T	AREA
36 X 36	36	3	18	2	0.080	3.90



SIGN SIZE	A	B	C	D	E	T	AREA
48 X 48	48	3	12	18	3	0.080	6.93
60 X 60	60	3	18	18	4	0.100	10.83

NOTE:
ALL HOLES ARE 3/8 " SQUARE UNLESS OTHERWISE NOTED.

THE DIMENSION "T" IS THE THICKNESS OF THE ALUMINUM BLANK.

ALL DIMENSIONS ARE IN INCHES.

(1) HOLES SHALL BE 5/16 " DIAMETER.

(2) CENTER HOLE IS REQUIRED.

ALL DIMENSIONS ARE IN INCHES.

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

SIGN BLANK DETAILS FOR FLAT SHEET SIGNS

TE503 7/1/03

FHWA APPROVAL			7/22/2003	APP'D	Steven A. Buckley		
DESIGNED	D.D.G.	DETAILED	A.A.D.	QUANTITIES		TRACED	
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.		TRACE CK.	

KDOT Graphics Certified 01-18-2011

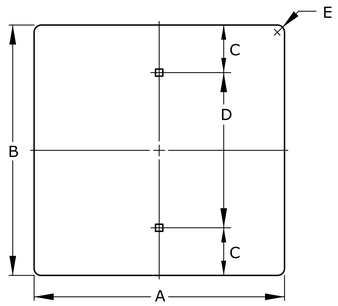
Sh. No. 65

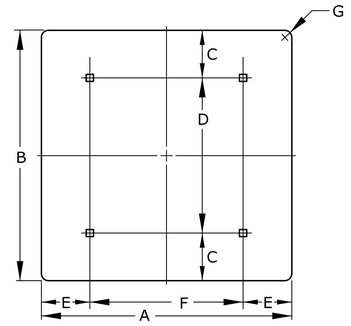
KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : TE503.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	66	99





	SIGN SIZE	A	B	C	D	E	T	AREA
(1)	3 X 8	3	8	1	6	3/8	0.040	0.17
	6 X 12	6	12	3	6	3/8	0.040	0.50
	12 X 6	12	6	1 1/2	3	3/8	0.063	0.50
	12 X 18	12	18	3	12	1 1/2	0.063	1.50
	12 X 24	12	24	3	18	1 1/2	0.080	2.00
	12 X 36	12	36	6	24	1 1/2	0.080	3.00
	12 X 48	12	48	6	36	1 1/2	0.080	4.00
	15 X 15	15	15	1 1/2	12	1 1/2	0.063	1.56
	18 X 6	18	6	1 1/2	3	1 1/2	0.063	0.75
	18 X 12	18	12	3	6	1 1/2	0.063	1.50
	18 X 18	18	18	3	12	1 1/2	0.063	2.25
	18 X 24	18	24	3	18	1 1/2	0.080	3.00
	21 X 15	21	15	1 1/2	12	1 1/2	0.080	2.19
	24 X 6	24	6	1 1/2	3	1 1/2	0.080	1.00
	24 X 12	24	12	3	6	1 1/2	0.080	2.00
	24 X 18	24	18	3	12	1 1/2	0.080	3.00
	24 X 24	24	24	3	18	1 1/2	0.080	4.00
	24 X 30	24	30	3	24	1 1/2	0.080	5.00
	24 X 36	24	36	6	24	1 1/2	0.080	6.00
	30 X 15	30	15	1 1/2	12	1 1/2	0.080	3.13
	30 X 18	30	18	3	12	1 1/2	0.080	3.75
	30 X 24	30	24	3	18	1 1/2	0.080	5.00
	30 X 30	30	30	3	24	1 1/2	0.080	6.25
	30 X 36	30	36	6	24	1 1/2	0.080	7.50
	36 X 12	36	12	3	6	1 1/2	0.080	3.00
	36 X 18	36	18	3	12	1 1/2	0.080	4.50
	36 X 24	36	24	3	18	1 1/2	0.080	6.00
	36 X 30	36	30	3	24	1 1/2	0.080	7.50
	36 X 36	36	36	6	24	2	0.080	9.00

	SIGN SIZE	A	B	C	D	E	F	G	T	AREA
	36 X 12	36	12	3	6	6	24	1 1/2	0.080	3.00
	36 X 18	36	18	6	12	6	30	1 1/2	0.080	4.50
	36 X 24	36	24	6	12	6	24	1 1/2	0.080	6.00
	36 X 30	36	30	6	18	6	30	1 1/2	0.080	7.50
	36 X 36	36	36	6	24	6	24	1 1/2	0.080	9.00
	36 X 42	36	42	6	30	6	30	1 1/2	0.080	10.50
	48 X 12	48	12	3	6	9	30	1 1/2	0.080	4.00
	48 X 18	48	18	3	12	9	30	1 1/2	0.080	6.00
	48 X 24	48	24	6	12	9	30	2	0.080	8.00
	48 X 30	48	30	6	18	9	30	0	0.100	10.00
	48 X 36	48	36	6	24	9	30	0	0.100	12.00
	48 X 42	48	42	6	30	9	30	0	0.100	14.00
	48 X 48	48	48	9	30	9	30	0	0.100	16.00
(2)	48 X 72	48	72	15	42	9	30	0	0.100	24.00
(2)	48 X 96	48	96	21	54	9	30	0	0.100	32.00
	54 X 36	54	36	6	30	12	30	0	0.100	13.50
	54 X 42	54	42	6	30	12	30	0	0.100	15.75
	60 X 12	60	12	3	6	12	36	0	0.100	5.00
	60 X 18	60	18	3	12	12	36	0	0.100	7.50

SIGN SIZE	A	B	C	D	E	F	G	T	AREA
60 X 24	60	24	6	12	12	36	0	0.100	10.00
60 X 30	60	30	6	18	12	36	0	0.100	12.50
60 X 36	60	36	6	24	12	36	0	0.100	15.00
60 X 42	60	42	6	30	12	36	0	0.100	17.50
60 X 48	60	48	9	30	12	36	0	0.100	20.00
60 X 60	60	60	12	36	12	36	0	0.100	25.00
72 X 12	72	12	3	6	15	42	0	0.100	6.00
72 X 18	72	18	3	12	15	42	0	0.100	9.00
72 X 24	72	24	6	12	15	42	0	0.100	12.00
72 X 30	72	30	6	18	15	36	0	0.100	15.00
72 X 36	72	36	6	24	15	42	0	0.100	18.00
72 X 42	72	42	6	30	15	42	0	0.100	21.00
72 X 48	72	48	9	30	15	42	0	0.100	24.00
84 X 18	84	18	6	12	15	48	0	0.100	10.50
84 X 30	84	30	6	18	18	48	0	0.100	17.50
84 X 36	84	36	6	24	18	48	0	0.100	21.00
84 X 42	84	42	6	30	18	48	0	0.100	24.50
84 X 48	84	48	9	30	18	48	0	0.100	28.00

NOTE:
ALL HOLES ARE 3/8 " SQUARE, UNLESS OTHERWISE NOTED.

THE DIMENSION "T" IS THE THICKNESS OF THE ALUMINUM BLANK.

(1) HOLES SHALL BE 5/16 " DIAMETER.

(2) DIMENSION "D" REQUIRES A CENTER HOLE.

ALL DIMENSIONS ARE IN INCHES.

NO.	DATE	REVISIONS	BY	APP'D

KANSAS DEPARTMENT OF TRANSPORTATION

SIGN BLANK DETAILS FOR FLAT SHEET SIGNS

TE506 7/1/03

FHWA APPROVAL		7/22/2003	APP'D	Steven A. Buckley	
DESIGNED	D.D.G.	DETAILED	A.A.D.	QUANTITIES	TRACED
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.	TRACE CK.

KDOT Graphics Certified 01-18-2011

KDOT Graphics Certified

Sh. No. 66

Plotted : 13-SEP-2013 10:54
Drawn By : cluedke
File : TE506.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	67	99

DETAILED SPECIFICATIONS FOR FLAT SHEET SIGNS

ALL NEW FLAT SHEET SIGN BLANKS SHALL BE OF THE ALUMINUM ALLOY AND THICKNESS SHOWN ON THE FLAT SHEET BLANK DETAIL SHEETS.

FLAT SHEET BLANKS SHALL BE USED FOR SIGNS THAT ARE LESS THAN OR EQUAL TO 7'-0" IN LENGTH AND/OR LESS THAN OR EQUAL TO 4'-0" IN HEIGHT. FLAT SHEET BLANKS SHALL ALSO BE USED FOR SIGNS THAT ARE 4'-0" IN LENGTH AND LESS THAN OR EQUAL TO 8'-0" IN HEIGHT.

THE DESIGN DETAILS FOR SIGNS (COLOR, LETTER HEIGHT, AND LETTER SERIES) SHALL BE AS SHOWN IN THE 'STANDARD HIGHWAY SIGNS' MANUAL (2004 EDITION), UNLESS OTHER DETAILS ARE SHOWN IN THE PLANS.

ALL SIGN FACES WITH BLUE, GREEN, RED, YELLOW, FLUORESCENT YELLOW GREEN, BROWN, OR WHITE BACKGROUND SHALL BE COVERED WITH TYPE IV HIGH INTENSITY RETROREFLECTIVE SHEETING.

THE TYPE OF ADHESIVE USED FOR RETROREFLECTIVE SHEETING OR LETTERING FILM SHALL BE HEAT ACTIVATED OR PRESSURE SENSITIVE.

THE SIGN FACES SHALL BE DIRECT SCREEN PROCESS, REVERSE SCREEN PROCESS, OR DIRECT APPLIED LEGEND.

DETAILED SPECIFICATIONS FOR STRUCTURAL EXTRUDED PANEL SIGNS

ALL NEW REINFORCED SIGN PANELS SHALL BE OF THE FABRICATION, ALUMINUM ALLOY, AND THICKNESS SHOWN ON THE REINFORCED PANEL DETAIL SHEETS. IF EXTRUSHEET FABRICATED SIGN PANELS ARE USED, THEY SHALL BE OF THE LENGTH, WIDTH AND IN THE POSITION SHOWN. IF EXTRUSHEET FABRICATED PANEL DIMENSIONS ARE NOT SHOWN, A LINE OF LEGEND SHOULD BE PLACED ENTIRELY ON ONE PANEL. IF EXTRUDED FABRICATED SIGN PANELS ARE USED, EITHER 1'-0" OR 6" PANELS SHALL BE USED. THE 1'-0" PANELS SHALL BE USED ONLY AT THE TOP OR BOTTOM OF SIGNS.

REINFORCED PANELS SHALL BE USED FOR SIGNS THAT ARE GREATER THAN 7'-0" IN LENGTH OR GREATER THAN 4'-0" IN HEIGHT.

ALL SIGN FACES SHALL BE COVERED WITH TYPE IV HIGH INTENSITY RETROREFLECTIVE SHEETING.

THE RETROREFLECTIVE SHEETING USED FOR THE DIRECT APPLIED LEGEND, AND DIRECT APPLIED BORDERS SHALL BE TYPE IV HIGH INTENSITY RETROREFLECTIVE SHEETING.

THE TYPE OF ADHESIVE USED FOR RETROREFLECTIVE SHEETING OR LETTERING FILM SHALL BE HEAT ACTIVATED OR PRESSURE SENSITIVE.

LETTERS AND NUMBERS ON REINFORCED PANEL SIGNS ARE MODIFIED SERIES "E" UNLESS OTHERWISE SHOWN.

SPACING TABLE DIMENSIONS ARE IN INCHES.

NO.	DATE	REVISIONS	BY	APP'D
1	7/23/10	Changed Notes and Sheeting Type	D.D.G.	D.B.

KANSAS DEPARTMENT OF TRANSPORTATION

DETAILS SPECIFICATIONS FOR REINFORCED SIGN PANELS AND FLAT SHEET SIGNS

TE590 7/1/03

FHWA APPROVAL		7/23/2010	APP'D	Steven A. Buckley	
DESIGNED	D.D.G.	DETAILED	K.D.S.	QUANTITIES	TRACED
DESIGN CK.	S.A.B.	DETAIL CK.	D.D.G.	QUAN. CK.	TRACE CK.

KDOT Graphics Certified 01-18-2011

KDOT Graphics Certified

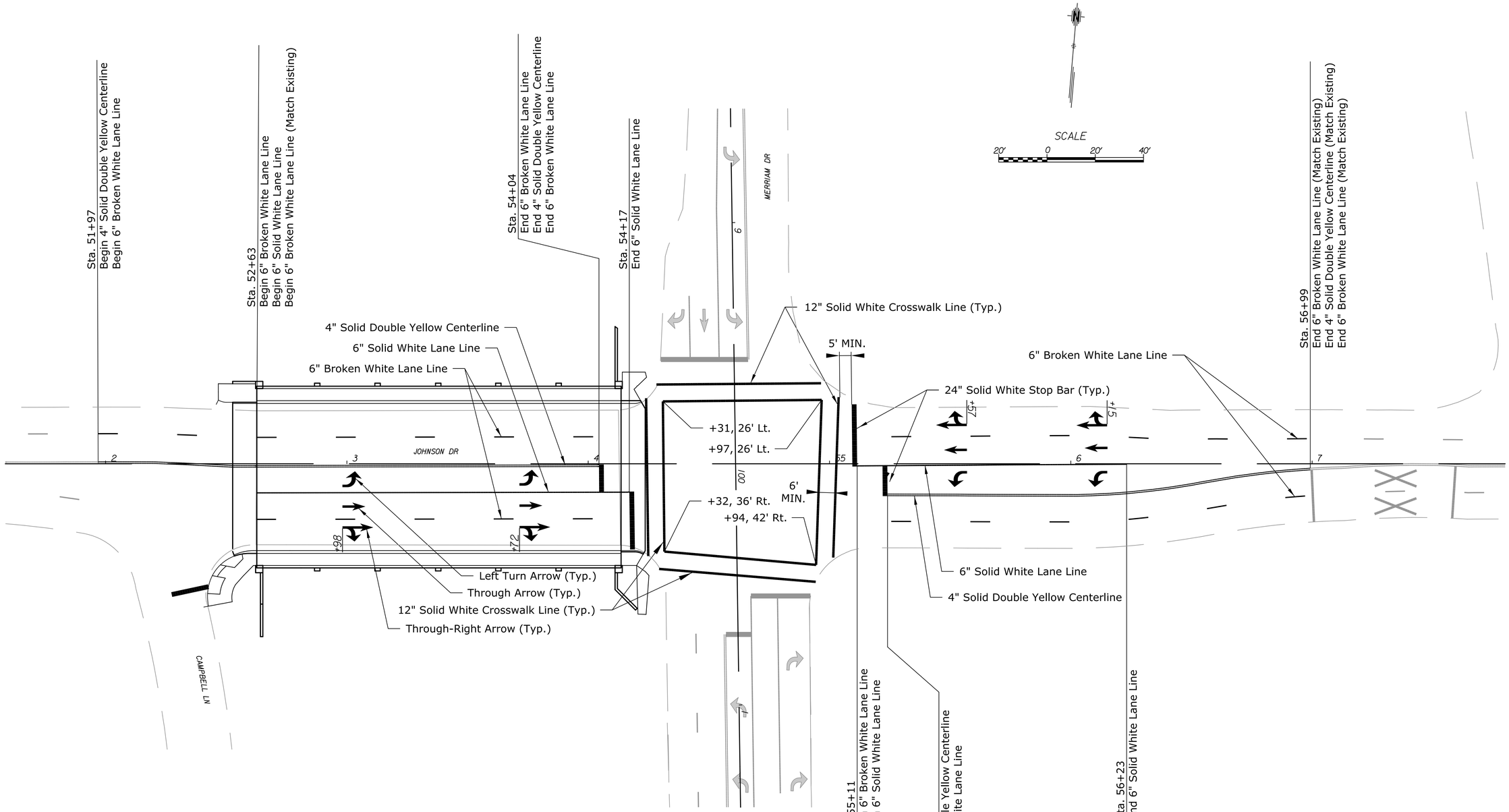
Sh. No. 67

Drawn By : cluedke
File : TE590.dgn

Plotted : 13-SEP-2013 10:54

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	68	99

BY	DATE
REFERENCES NOTED	
REFERENCES CHECKED	

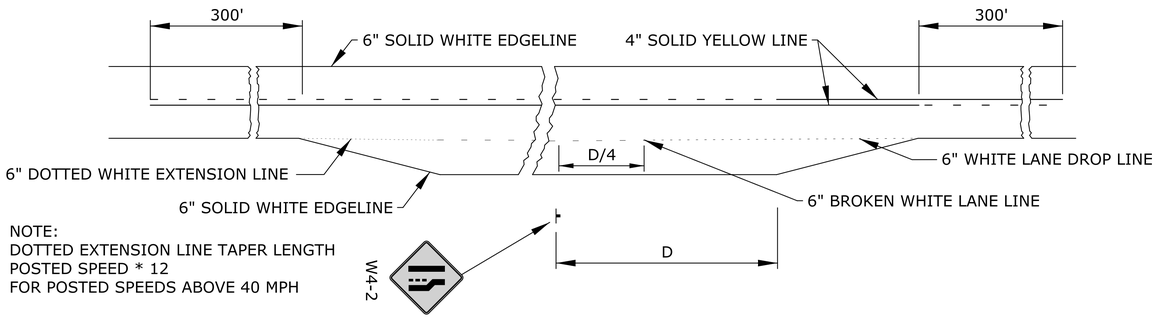


Drawn By : cluedke Plotted :9/13/2013
File : G:\12664\Micro\Traffic\12664_5203.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

PAVEMENT MARKING PLAN

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	69	99

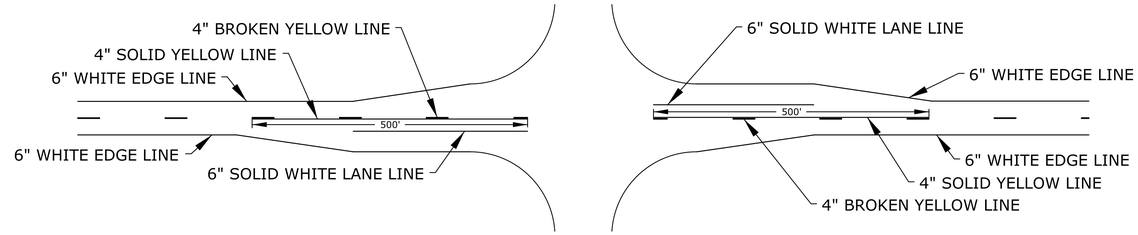


NOTE:
DOTTED EXTENSION LINE TAPER LENGTH
POSTED SPEED * 12
FOR POSTED SPEEDS ABOVE 40 MPH

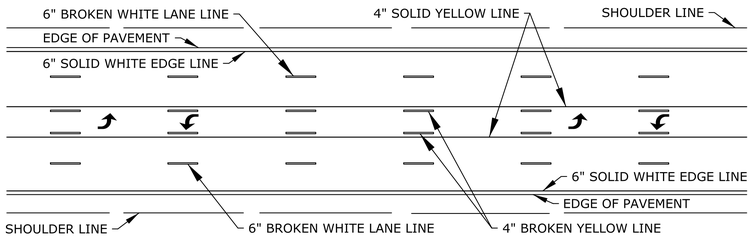
TYPICAL MARKING FOR AUXILIARY PASSING LANE

NOTE:
ALL PAVEMENT MARKINGS SHALL BE BROKEN AT CROSS ROADS.

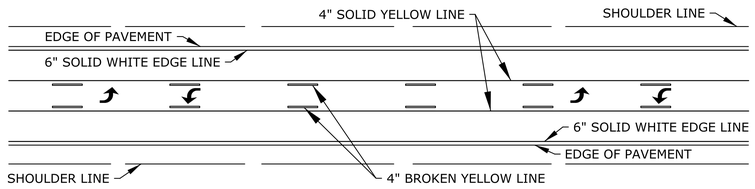
FOR HIGHWAY JUNCTIONS THE NO PASSING ZONE WILL EXTEND 1000' FROM INTERSECTION.



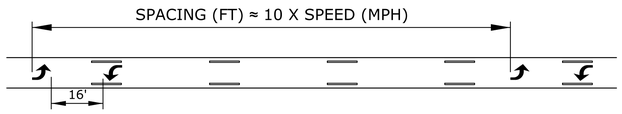
TYPICAL ROAD JUNCTION MARKINGS WITH BYPASS LANES



TWO-WAY LEFT TURN DETAIL FOR FIVE LANE ROADWAY

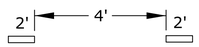


TWO-WAY LEFT TURN DETAIL FOR THREE LANE ROADWAY

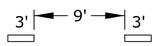


TWO-WAY LEFT TURN ARROW SPACING DETAIL

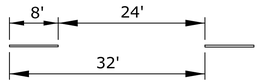
NOTE:
IF ARROWS ARE USED SPACE THE ARROWS AS SHOWN IN THE SPACING DETAIL.



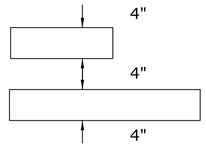
TYPICAL SPACING FOR DOTTED EXTENSION LINES, UNLESS OTHERWISE NOTED ON PLANS.



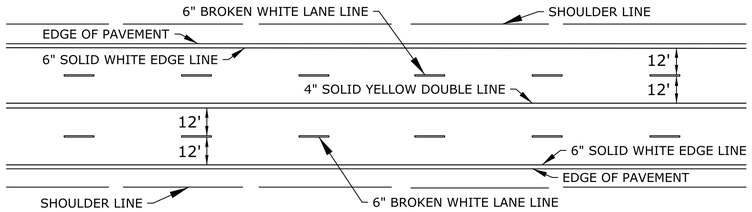
TYPICAL SPACING FOR LANE DROP. UNLESS OTHERWISE NOTED ON PLANS.



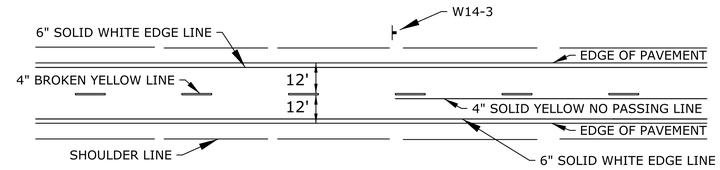
TYPICAL SPACING FOR BROKEN LINES UNLESS OTHERWISE NOTED ON PLANS



TYPICAL SPACING FOR NO PASSING LINES UNLESS OTHERWISE NOTED ON PLANS



TYPICAL MARKINGS FOR FOUR LANE ROADWAY



TYPICAL TWO LANE MARKINGS

NOTE:
LONGITUDINAL PAVEMENT MARKING LINES SHALL BE OFFSET A MINIMUM OF 2" FROM LONGITUDINAL PAVEMENT JOINTS.

NOTE:
ON NON I, US, AND K ROUTES, 4" EDGE LINES MAY BE INSTALLED.
6" EDGE LINES ARE NOT REQUIRED ON NON I, US, AND K ROUTES.

NO.	DATE	REVISIONS	BY	APP'D
3	5/25/12	Added Dotted Extension and Lane Drop Lines	B.A.H.	B.D.G.
2	9/20/05	Removed Aux. Passing Lane Dotted Ext. Line	J.F.F.	B.D.G.
1	7/26/05	New FHWA Approval Date	J.F.F.	B.D.G.

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL PAVEMENT MARKING DETAILS FOR UNDIVIDED ROADWAYS

TE308

FHWA APPROVAL		5/25/2012	APP'D	Brian D. Gower	
DESIGNED	J.F.F.	DETAILED	J.F.F.	QUANTITIES	TRACED
DESIGN CK.	B.D.G.	DETAIL CK.	B.D.G.	QUAN. CK.	TRACE CK.

KDOT Graphics Certified 06-20-2012

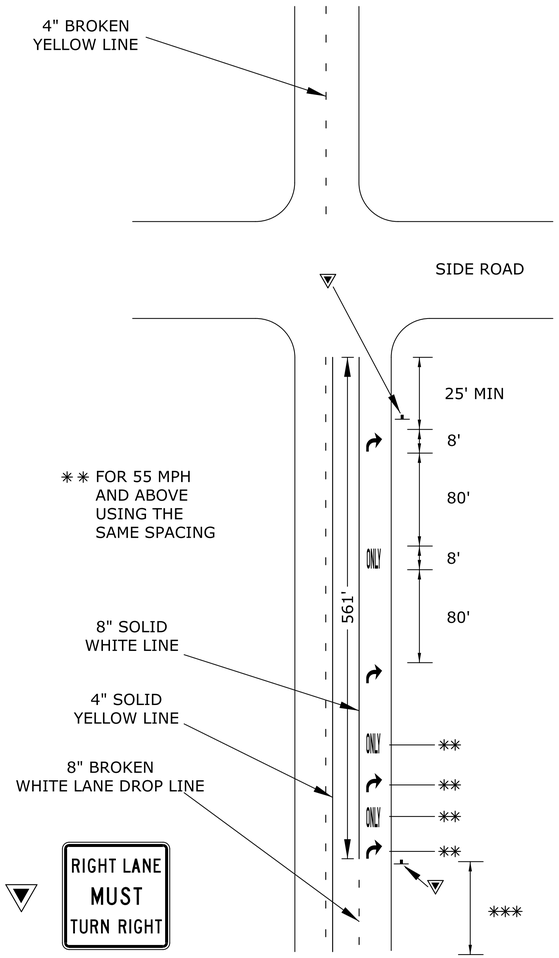
Sh. No. 69

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54
Drawn By : cluedke
File : te308.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	70	99

TYPICAL SIGNING AND MARKING FOR RIGHT LANE MUST TURN RIGHT



✳✳ FOR 55 MPH AND ABOVE USING THE SAME SPACING

✳✳✳ THE LANE DROP MARKINGS LENGTH IS A MINIMUM OF 100' AND A MAXIMUM OF 250' PRIOR TO THE 8"SOLID WHITE LINE

RAILROAD CROSSING MARKING

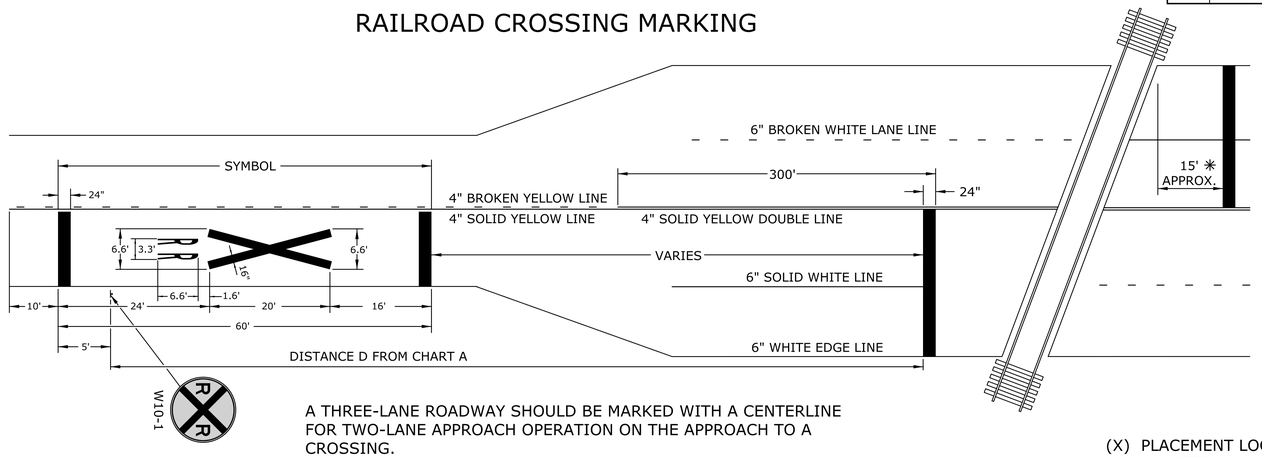


CHART "A"

SPEED MPH	DISTANCE D (feet)
75	850
70	750
65	650
60	550
55	450
50	375
45	300
40	225
35	150
30	(X)
25	(X)
20	(X)

ALL DISTANCES ARE MINIMUM.

A THREE-LANE ROADWAY SHOULD BE MARKED WITH A CENTERLINE FOR TWO-LANE APPROACH OPERATION ON THE APPROACH TO A CROSSING.
ON MULTI-LANE ROADS THE TRANSVERSE BANDS SHOULD EXTEND ACROSS ALL APPROACH LANES, AND INDIVIDUAL R X R SYMBOLS SHOULD BE USED IN EACH APPROACH LANE.
REFER TO STANDARD ALPHABET FOR HIGHWAY SIGNS AND MARKINGS FOR R X R SYMBOLS DETAILS.

*STOP LINE 8' FROM NEAR EDGE OF GATE OR CANTILEVER, IF PRESENT.

NOTE:
ON NON I, US, AND K ROUTES, 4" EDGE LINES MAY BE INSTALLED.
6" EDGE LINES ARE NOT REQUIRED ON NON I, US, AND K ROUTES.

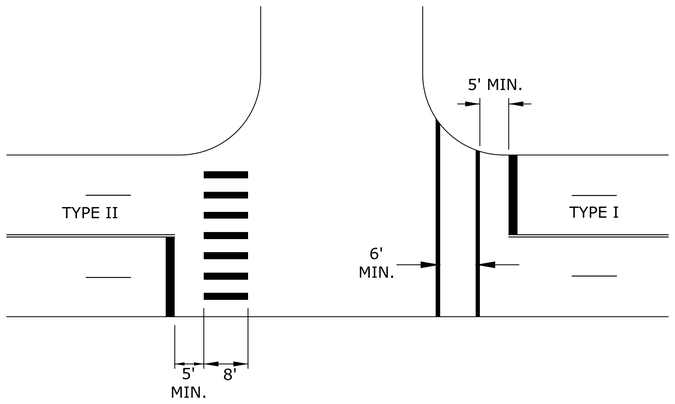
(X) PLACEMENT LOCATION IS DEPENDENT ON SITE CONDITIONS AND OTHER SIGNING TO PROVIDE ADEQUATE ADVANCE WARNING TO THE DRIVER

TYPICAL CROSSWALKS

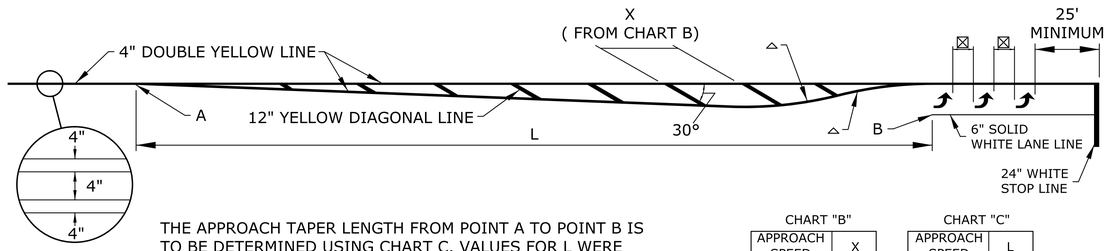
TYPE I: CROSSWALK LINES SHALL BE 12" SOLID WHITE LINES. THEY SHALL BE SPACED A MINIMUM OF 6' APART FROM INSIDE EDGE TO INSIDE EDGE.

TYPE II: THESE LINES SHOULD BE SOLID WHITE 24" WIDE PLACED PARALLEL TO THE DIRECTION OF TRAFFIC FLOW. THE LINE PLACEMENT IS DETERMINED BY LANE LINE,CENTER LINE, AND WHEEL PATH IN SUCH A MANNER AS TO MINIMIZE TRAFFIC WEAR. THE CROSSWALK WIDTH SHOULD BE NOT LESS THAN 8'. THE TRANSVERSE CROSSWALK LINES MAY BE ADDED.

WHEN REQUIRED, STOP LINES SHALL BE INSTALLED A MINIMUM OF 5' FROM CROSSWALKS.



TYPICAL APPROACH TAPER DETAIL



THE APPROACH TAPER LENGTH FROM POINT A TO POINT B IS TO BE DETERMINED USING CHART C. VALUES FOR L WERE CALCULATED USING THE EQUATIONS BELOW AND INCREASED TO THE NEXT HIGHER 5 MPH INCREMENT.

- SPEEDS < 45 MPH $L = \frac{W*S^2}{60}$
- SPEEDS = 45 MPH $L = W*S$

IF ARROWS ARE USED AND UNLESS OTHERWISE SPECIFIED THE SPACE BETWEEN LINES SHOULD BE AT LEAST FOUR TIMES THE HEIGHT OF THE CHARACTERS FOR LOW SPEED ROADS BUT NOT MORE THAN TEN TIMES THE HEIGHT OF THE CHARACTERS, UNDER ANY CONDITIONS.

FOR SPEEDS LESS THAN OR EQUAL TO 40 MPH, R=150'.
FOR SPEEDS GREATER THAN OR EQUAL TO 45 MPH, R=300'.

CHART "B"

APPROACH SPEED	X
20 MPH	20'
25 MPH	25'
30 MPH	30'
35 MPH	35'
40 MPH	40'
45 MPH	45'
50 MPH	50'
55 MPH	55'
60 MPH	60'
65 MPH	65'
70 MPH	70'

CHART "C"

APPROACH SPEED	L
20 MPH	80'
25 MPH	125'
30 MPH	180'
35 MPH	245'
40 MPH	320'
45 MPH	540'
50 MPH	600'
55 MPH	660'
60 MPH	720'
65 MPH	780'
70 MPH	840'

NO.	DATE	REVISIONS	BY	APP'D
3	5/25/12	Updated Chart B and Lane Drop Lines	B.A.H.	B.D.G.
2	10/20/06	RR Xing Symbol Changed from 18' to 16'	T.L.H.	B.D.G.
1	9/20/05	Added 4" Solid Yellow Double Line to RRxing	J.F.F.	B.D.G.

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL MISCELLANEOUS PAVEMENT MARKING DETAIL SHEET

TE309

FHWA APPROVAL		7/26/2005	APP'D	Brian D. Gower	
DESIGNED	J.F.F.	DETAILED	J.F.F.	QUANTITIES	TRACED
DESIGN CK.	B.D.G.	DETAIL CK.	B.D.G.	QUAN. CK.	TRACE CK.

KDOT Graphics Certified 06-26-2012

Sh. No. 70

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te309.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	71	99

SUMMARY OF PAVEMENT MARKINGS

LOCATION	4" Solid WHITE Edge Line	6" Solid WHITE Edge Line	6" Broken WHITE Lane Line	6" Broken WHITE Lane Line (PCP)	6" Dotted WHITE Extension Line	6" Broken WHITE Lane Drop Line	6" Solid WHITE Lane Line	8" Broken WHITE Lane Drop Line	8" Solid WHITE Gore Line	8" Dotted WHITE Extension Line	12" Solid WHITE Diagonal Line	12" Solid WHITE Chevron Line	12" Solid WHITE Type I Crosswalk Line	24" Solid WHITE Type II Crosswalk Line	24" Solid WHITE Stop Line	4" Solid YELLOW Edge Line	4" Solid YELLOW Double Line	4" Solid YELLOW Line	4" Broken YELLOW Line	6" Solid YELLOW Edge Line	12" Solid YELLOW Diagonal Line
JOHNSON DR			192				266						529		87		382				
TOTALS			192				266						529		87		764				

RECAPITULATION OF QUANTITIES

ITEMS	TOTAL	UNITS
PAVEMENT MARKING (MULTI-COMPONENT)(WHITE)(4")		FT
PAVEMENT MARKING (MULTI-COMPONENT)(WHITE)(6")	458	FT
PAVEMENT MARKING (MULTI-COMPONENT)(WHITE)(8")		FT
PAVEMENT MARKING (MULTI-COMPONENT)(WHITE)(12")		FT
PAVEMENT MARKING (MULTI-COMPONENT)(YELLOW)(4")	764	FT
PAVEMENT MARKING (MULTI-COMPONENT)(YELLOW)(6")		FT
PAVEMENT MARKING (MULTI-COMPONENT)(YELLOW)(12")		FT
PAVEMENT MARKING (THERMOPLASTIC)(WHITE)(4")		FT
PAVEMENT MARKING (THERMOPLASTIC)(WHITE)(6")		FT
PAVEMENT MARKING (THERMOPLASTIC)(WHITE)(8")		FT
PAVEMENT MARKING (THERMOPLASTIC)(WHITE)(12")		FT
PAVEMENT MARKING (THERMOPLASTIC)(YELLOW)(4")		FT
PAVEMENT MARKING (THERMOPLASTIC)(YELLOW)(6")		FT
PAVEMENT MARKING (THERMOPLASTIC)(YELLOW)(12")		FT
PAVEMENT MARKING (EPOXY)(WHITE)(4")		FT
PAVEMENT MARKING (EPOXY)(WHITE)(6")		FT
PAVEMENT MARKING (EPOXY)(WHITE)(8")		FT
PAVEMENT MARKING (EPOXY)(WHITE)(12")		FT
PAVEMENT MARKING (EPOXY)(YELLOW)(4")		FT
PAVEMENT MARKING (EPOXY)(YELLOW)(6")		FT
PAVEMENT MARKING (EPOXY)(YELLOW)(12")		FT
PAVEMENT MARKING (INTERSECTION GRADE)(WHITE)(12")	529	FT
PAVEMENT MARKING (INTERSECTION GRADE)(WHITE)(24")	87	FT
PAVEMENT MARKING (INTERSECTION GRADE)(YELLOW)(12")		FT
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(WHITE)(ARROW - THROUGH RIGHT)	4	EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(WHITE)(ARROW - THROUGH)	4	EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(WHITE)(ARROW - LEFT)	4	EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(WHITE)( )		EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(WHITE)( )		EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(US-SHIELD)( )		EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(K-SHIELD)( )		EACH
PAVEMENT MARKING SYMBOL (INTERSECTION GRADE)(I-SHIELD)( )		EACH
PAVEMENT MARKING (PATTERNED COLD PLASTIC)(WHITE)(6")		FT
PAVEMENT MARKING (PATTERNED COLD PLASTIC)(WHITE)(8")		FT
PAVEMENT MARKING (PATTERNED COLD PLASTIC)(WHITE)(12")		FT
PAVEMENT MARKING REMOVAL		FT

SUMMARY OF WORD & SYMBOL MARKINGS

LOCATION	STOP	ONLY	X-ING	SCHOOL	70	435	24	400	18
JOHNSON DR	4		4		4				
TOTALS	4		4		4				

NOTE: FOR SPECIFIC PAVEMENT MARKING DETAILS AND DIMENSIONS SEE PLAN SHEETS

NOTE: ALL TOTALS REFLECT ACTUAL QUANTITY OF PAVEMENT MARKING MATERIALS REQUIRED.

NOTE:
WORDS & SYMBOLS SHALL CONFORM TO THE LATEST EDITION OF "STANDARD ALPHABETS FOR HIGHWAY SIGNS AND PAVEMENT MARKINGS" PRINTED BY THE U.S. DEPARTMENT OF TRANSPORTATION, FEDERAL HIGHWAY ADMINISTRATION.

PRIOR TO COMMENCEMENT OF PAVEMENT MARKING WORK THE ENGINEER WILL ESTABLISH THE LIMITS FOR "NO PASSING" ZONES. THESE LIMITS SHALL BE USED FOR THE LOCATION OF "NO PASSING" LINES AND FOR THE COMPUTATION OF ACTUAL MARKING QUANTITIES FOR THIS LINE TYPE.

NO.	DATE	REVISIONS	BY	APP'D
2	5/25/12	Added Line Types, Symbols, and Shields	B.A.H.	B.D.G.
1	7/26/05	New FHWA Approval Date	J.F.F.	B.D.G.

KANSAS DEPARTMENT OF TRANSPORTATION

SUMMARY AND RECAPITULATION OF PAVEMENT MARKING QUANTITIES

TE311

FHWA APPROVAL		5/25/2012	APP'D	Brian D. Gower	
DESIGNED	J.F.F.	DETAILED	J.F.F.	QUANTITIES	TRACED
DESIGN CK.	B.D.G.	DETAIL CK.	B.D.G.	QUAN. CK.	TRACE CK.

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te311.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	72	99

GENERAL NOTE:

~~On surfacing~~ projects, the 6" of Compaction Type AA, shown for the center ~~portion~~ on the roadbed, ~~is for the purpose~~ of restoring the original ~~Compaction~~ Type AA which may have been lost since grading operations. ~~The exact~~ locations of this Compaction Type AA, which will be required, ~~is to be~~ determined by the ~~Engineer at the~~ time of construction. ~~This work shall~~ be paid under the bid item "Compaction of Earthwork (Type AA) (MR-)".

~~Over all structures~~, unless otherwise directed by the Engineer, ~~where the top of~~ the hubguard is ~~level with or above the finished shoulder grade~~, the earth cover over the structure ~~slab shall be removed and backfilled with~~ material ~~as directed by the Engineer~~. The removal of this material will be subsidiary.

~~The~~ material used to ~~backfill over the structure shall be paid for at the prices shown in the contract.~~

The earth shoulders shall be compacted full depth (Type -MR) except, when ordered by the Engineer, the top 3" shall be left uncompacted for seeding.

All side roads and house entrances shall be surfaced with to the R/W line as indicated on the detail. All side roads and house entrances with existing asphalt surface shall be surfaced with at least to the R/W line or to the end of construction, as directed by the Engineer. Each mailbox turnout (ON PROJECTS WHERE STABILIZED SHOULDERS ARE NOT SPECIFIED) shall be surfaced with to the limits shown on the detail.

Surfacing material (SA-) shall be used for surfacing house entrances and side roads (C.Y./SQ. YD.) beyond the limits of the asphalt surface to the limits of construction as determined by the Engineer.

~~The thickness~~ of side road and entrance surfacing may be ~~increased to the same thickness as the stabilized shoulder within the approximate limits of the shoulder.~~

On projects which specify both ~~asphalt base and~~ surface course materials, side roads, ~~house entrances~~ and mailbox turnouts may be ~~surfaced with~~ both materials ~~at the~~ contractors option, with the approval of the Engineer.

~~Quantities for aggregate for shoulders, AS-1, are calculated on the basis of 150 lbs. per cu. ft. Quantities for stabilized base course, AB-3, are calculated on the basis of 156 lbs. per cu. ft. Weight/cu. ft. includes moisture allowed by specification.~~

~~The base course shall be constructed to the plan thickness as shown.~~

Thicknesses indicated for all construction which is paid for on a weight or volume basis are approximate and may vary to correct for unevenness in the foundations or for other normal unevenness encountered in placement operations.

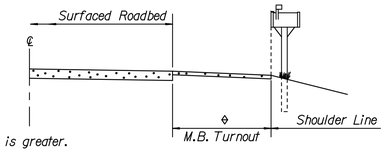
~~A tack coat of SS-1HP shall be provided between each lift of all base courses and surface courses and under the first lift of base or surface courses when they are placed on an existing asphalt, brick, or concrete surface, when so ordered by the Engineer and at the rate designated by him. Quantities are included for these tacks calculated at the rate of 0.05 gal./sq. yd.~~

~~Asphalt Material quantities are calculated on the basis of 8.328 lbs. per gal.~~

~~Shoulder rumble strips will not be constructed as part of this project.~~

HMA Commercial Grade (Class A)					
Location	*Station to Station*		*Thickness (in.)*	*Tons*	*Remarks*
Johnson Drive	*52+36.75*	*52+52.66*	*12"*	*9.5*	*S.W. Corner Bridge*
Johnson Drive	*52+45.85*	*52+52.66*	*12"*	*1.0*	*N.W. Corner Bridge*
			Total=	*10.5*	

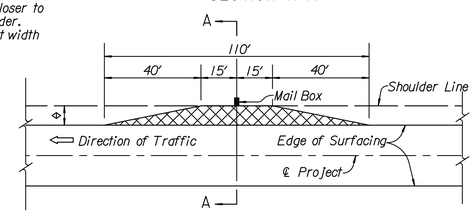
Asphalt Unit weight used for calculation = 145 lb/cu.ft.
HMA quantity includes an additional 3/8" for contingency.



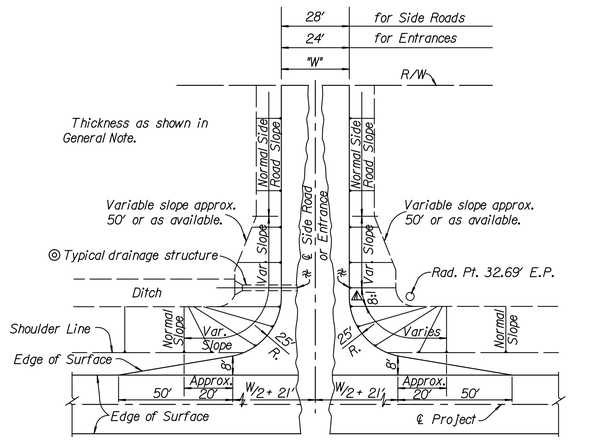
◊ *Width shall be 8' or shoulder width, whichever is greater.*

SECTION A-A

Note: The face of Mail Box should be no closer to the roadway than the edge of the shoulder. Align with edge of turnout when turnout width is greater than shoulder width.

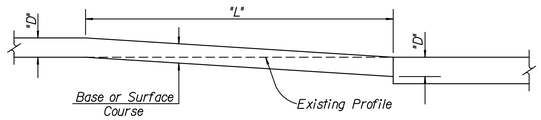


DETAIL FOR SURFACING OF MAIL BOX TURNOUTS



WITH DRAINAGE STRUCTURE MOUND ENTRANCE OR SIDE ROAD

DETAIL FOR SURFACING OF SIDE ROADS & HOUSE ENTRANCES



TYPICAL PROFILE AT GRADE CONTROL POINTS

The Contractor shall cut the subgrade in accordance with this profile at all grade control points, i.e.; existing pavements, grade bridges and R.R. crossings, also at changes in thickness of base or surface courses. Corresponding dimensions of "D" and "L" shall be as given in the table below.

The work of cutting the subgrade and disposing of excess excavated material shall be subsidiary to other items in the contract.

TABLE OF DIMENSIONS											
D	*L*	*D*	*L*	*D*	*L*	*D*	*L*	*D*	*L*	*D*	*L*
1"	*25'*	*3"*	*75'*	*5"*	*125'*	*7"*	*175'*	*9"*	*225'*	*11"*	*275'*
2"	*50'*	*4"*	*100'*	*6"*	*150'*	*8"*	*200'*	*10"*	*250'*	*12"*	*300'*

RATES OF APPLICATION				
RATE	*UNIT*	*ITEM*		

† *Computed at the rate of*
†† *Computed at the rate of*

RECAPITULATION OF QUANTITIES					
ITEM				*TOTAL*	*UNIT*
HMA Commercial Grade (Type A)				*11*	*Tons*

⚠ *8:1 Slope at the appropriate clear zone shall apply to all mound entrances and mound side roads to 10' fill height. Normal Slope (but not steeper than 6:1) for over 10' fill height.*

◎ *Normal Slope (but not steeper than 6:1) at approximate ℄ Structure or appropriate clear zone width.*

✻ *On side roads and entrances which slope toward the highway, a low point approx. 6" deep shall be constructed to divert surface drainage into the highway ditch, unless otherwise shown on the plans.*

NO.	DATE	REVISIONS	BY	APP'D
12	1-10-07	Changed bituminous to asphalt	S.W.K.	J.O.B.
11	8-30-06	Changed tack type/rate	S.W.K.	J.O.B.
10	3-24-05	Revised compaction, tack type/rate	S.W.K.	J.O.B.
9	6-12-02	Added low point off shoulder.	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

SUMMARY OF QUANTITIES (Surfacing)

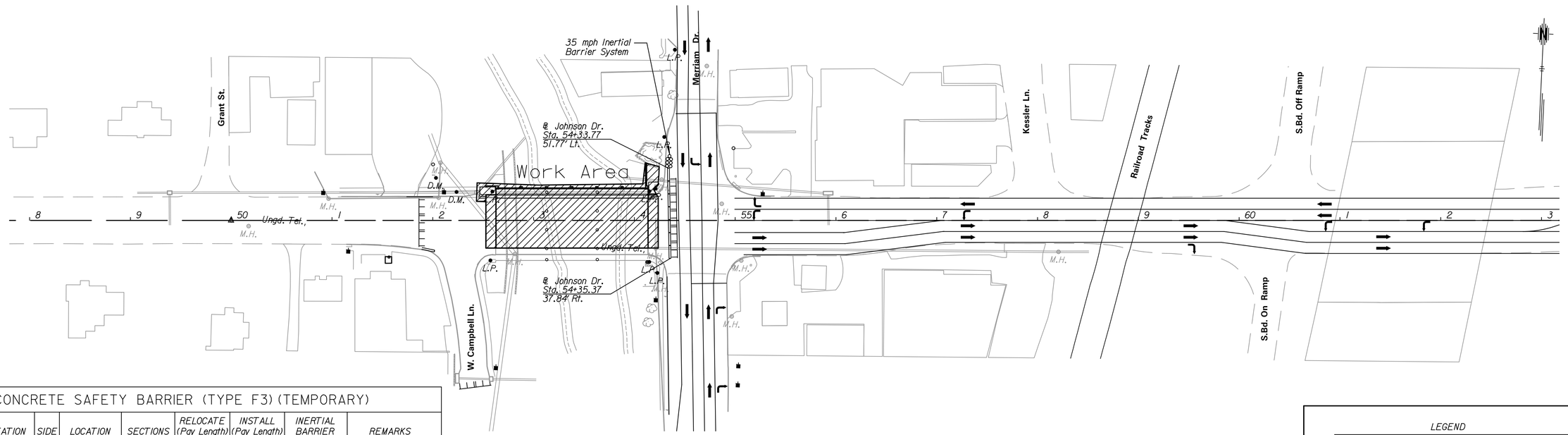
RD051

FHWA APPROVAL	9-06-06	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. Hecht

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\MicroHighways\Client Standards\rd051.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	73	99

DATE	BY	REFERENCES NOTED / REFERENCES CHECKED

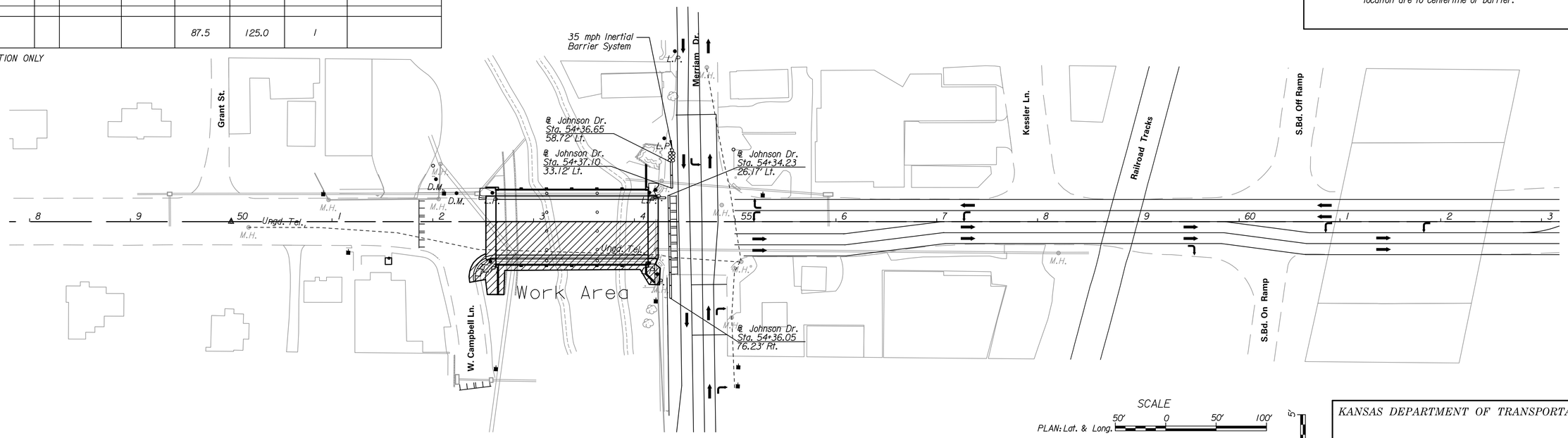


Sequence 1

CONCRETE SAFETY BARRIER (TYPE F3)(TEMPORARY)

STATION TO STATION		SIDE	LOCATION	SECTIONS	RELOCATE (Pay Length) (ft)	INSTALL (Pay Length) (ft)	INERTIAL BARRIER SYSTEMS	REMARKS
54+33.77	54+35.37	Lt/Rt	Johnson Dr.	7		87.5	1	Seq. 1
			Subtotal			87.5		
54+36.65	54+37.10	Lt	Johnson Dr.	5	25.0	37.5	1	Seq. 2
54+36.05	54+35.50	Lt/Rt	Johnson Dr.	5	62.5			
			Subtotal		87.5	37.5		
TOTAL					87.5	125.0	1	

FOR INFORMATION ONLY



Sequence 2

LEGEND

- Construction completed in current sequence
- Temporary Concrete Safety Barrier (TCSB)
- Mainline Traffic Flow

NOTE: Station and offset callouts for barrier location are to centerline of barrier.

SCALE

PLAN: Lat. & Long. 50' 0 50' 100'

PROFILE: Horiz. same as above

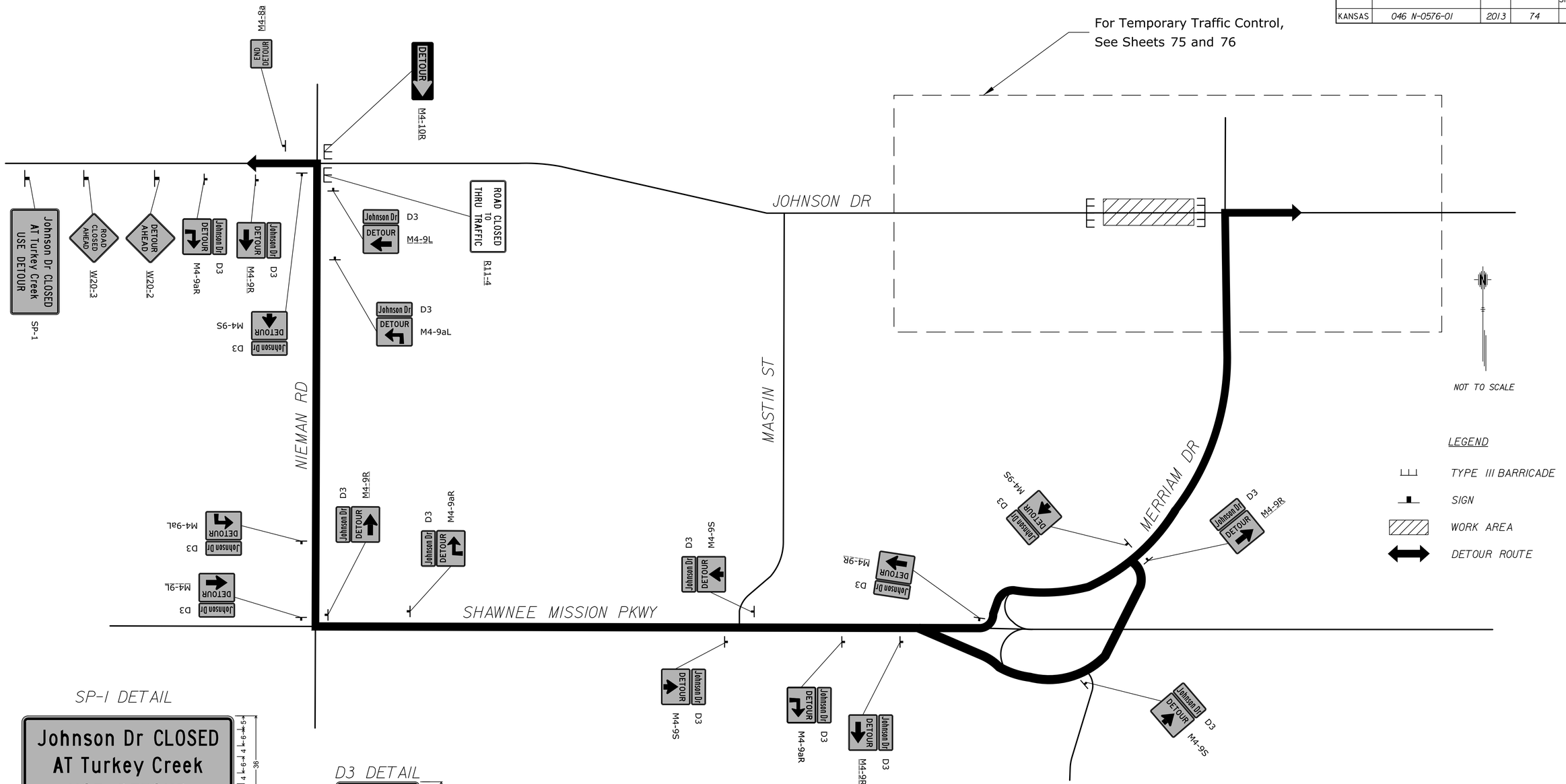
Vert. 5 0

KANSAS DEPARTMENT OF TRANSPORTATION

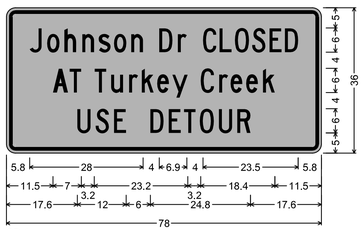
Construction Sequencing

Drawn By : cluedke
File : G:\12664\MicroHighways\N057601rcs-01.dgn
Plotted : 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	74	99



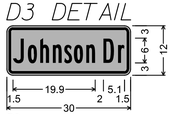
SP-1 DETAIL



SP-1;
2.3" Radius, 0.9" Border, 0.6" Indent, Black on Orange;
[Johnson Dr CLOSED] C 80% spacing;
[AT Turkey Creek] C 80% spacing; [USE DETOUR] C;
Table of distances between letter and object lefts.

	J	o	h	n	s	o	n		D	r	C	L	O	S	E	D															
5.8	4.3	4.2	4.5	4.0	3.8	4.3	6.9	4.6	6.3	4.1	3.8	4.3	3.8	3.2	5.8																
	A	T	T	u	r	k	e	y	C	r	e	e	k																		
11.5	4.0	6.2	4.2	4.5	3.1	3.8	3.8	7.0	4.6	2.8	3.9	4.1	3.0	11.5																	
	U	S	E	D	E	T	O	U	R																						
17.6	4.5	4.5	9.0	4.6	3.7	4.0	4.7	4.6	3.2	17.6																					

D3 DETAIL



D3;
1.5" Radius, 0.6" Border, 0.4" Indent, Black on Orange;
[Johnson] B 2K;
[Dr] B 2K;
Table of distances between letter and object lefts.

	J	o	h	n	s	o	n	D	r	
1.5	3.6	3.0	3.0	2.8	2.3	3.0	4.2	3.5	1.6	1.5

DATE		
BY		
REFERENCES NOTED		
REFERENCES CHECKED		

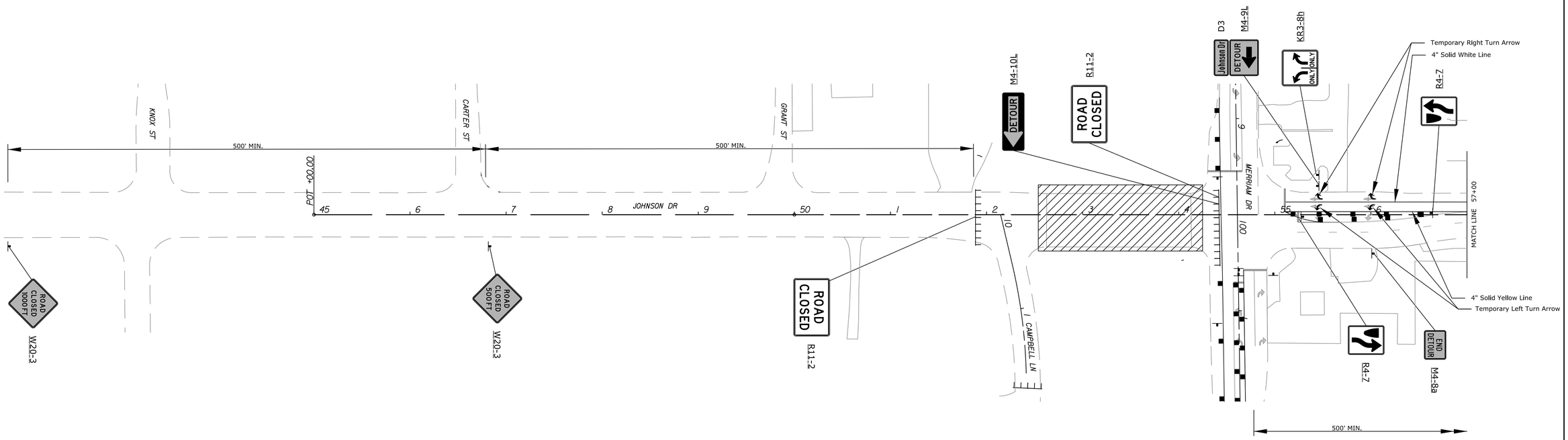
Drawn By : cluedke
File : G:\12664\Micro\Traffic\12664_5450.dgn
Plotted : 9/13/2013

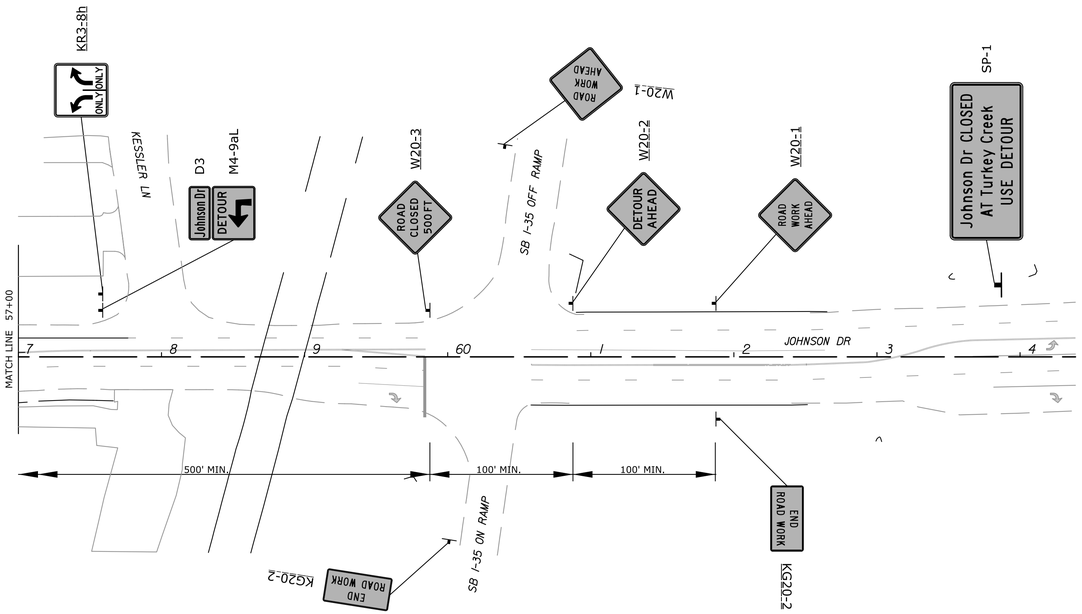
KANSAS DEPARTMENT OF TRANSPORTATION

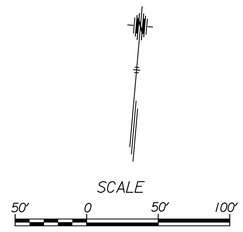
DETOUR PLAN

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	75	99

DATE	BY	REFERENCES NOTED	REFERENCES CHECKED







NOTE: For Temporary Traffic Control on Merriam Dr and Campbell Ln, See Sheet 76

LEGEND:

- Work Area
- Complete Closure using Type III Barricades
- Channelizing Device
- Sign

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL PLAN

JOHNSON DR

Drawn By : cluedke
Plotted : 9/13/2013
File : G:\12664\MicroTraffic\12664_5401.dgn

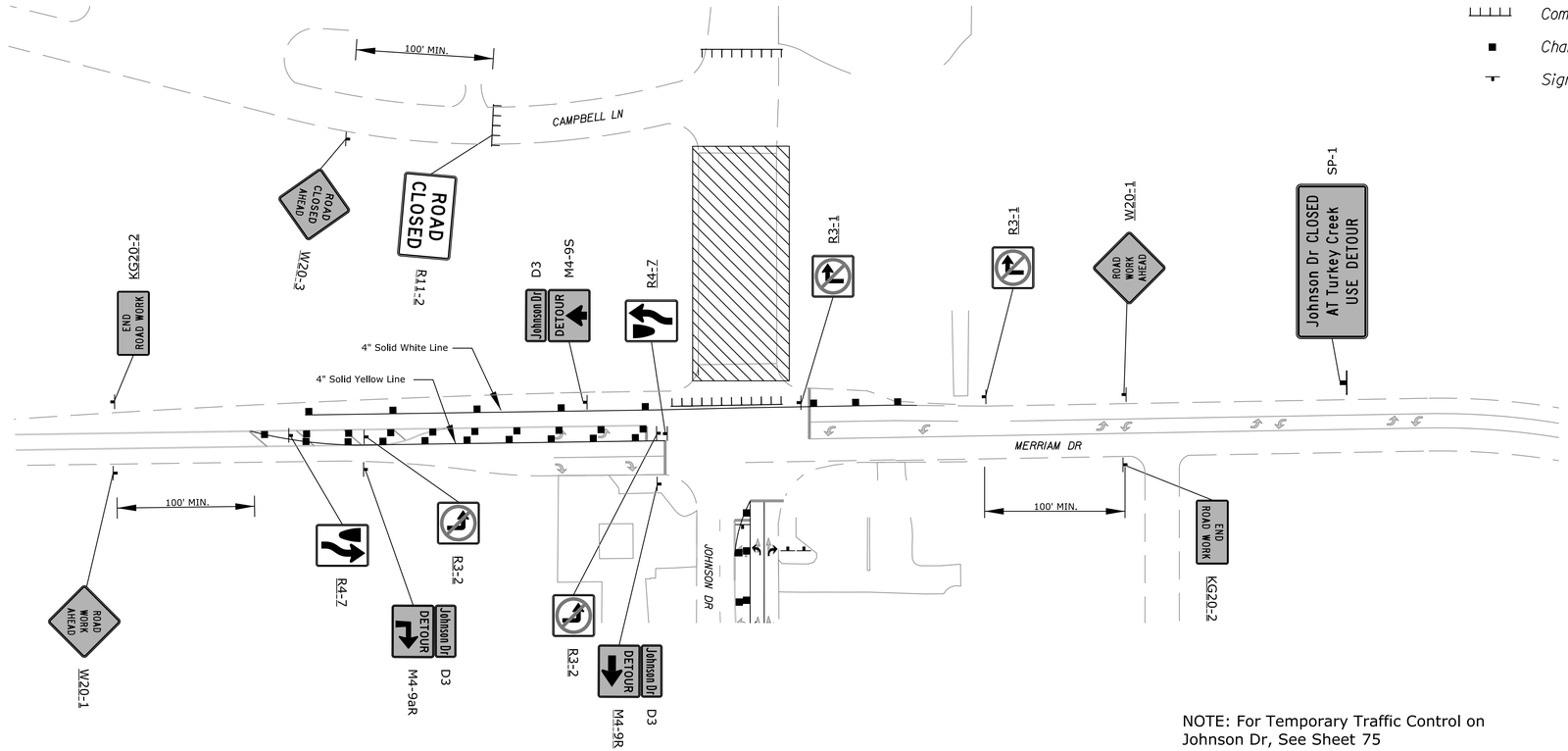
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	76	99

SCALE

50' 0 50' 100'

LEGEND:

- Work Area
- Complete Closure using Type III Barricades
- Channelizing Device
- Sign



NOTE: For Temporary Traffic Control on Johnson Dr, See Sheet 75

DATE	BY	
		REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\Micro\Traffic\12664_5402.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

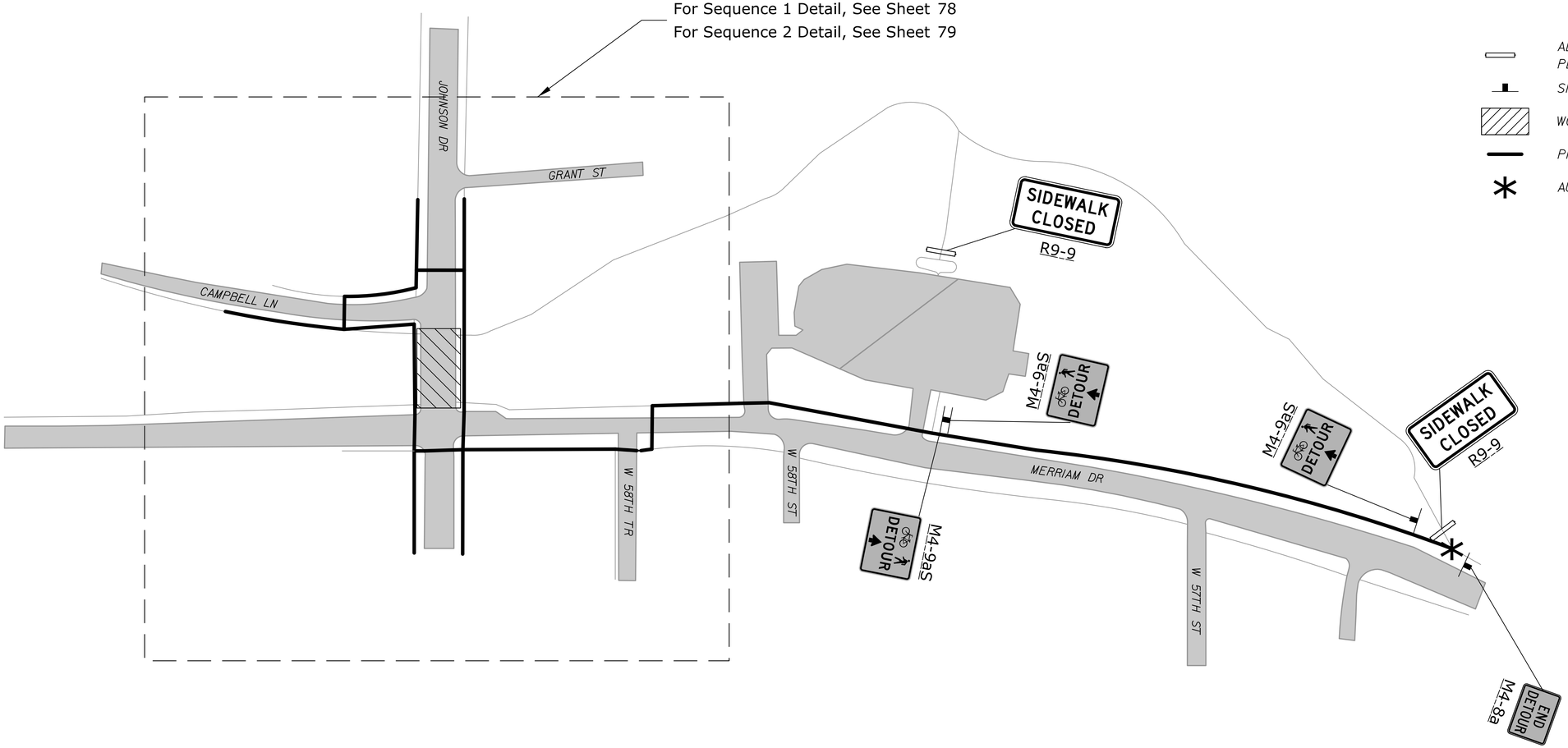
TRAFFIC CONTROL PLAN

MERRIAM DR

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	77	99

NOT TO SCALE

For Sequence 1 Detail, See Sheet 78
For Sequence 2 Detail, See Sheet 79



LEGEND

- ADA COMPLIANT TEMPORARY PEDESTRIAN BARRICADE
- SIGN
- WORK AREA
- PROPOSED SIDEWALK DETOURS
- AUDIBLE INFORMATION DEVICE

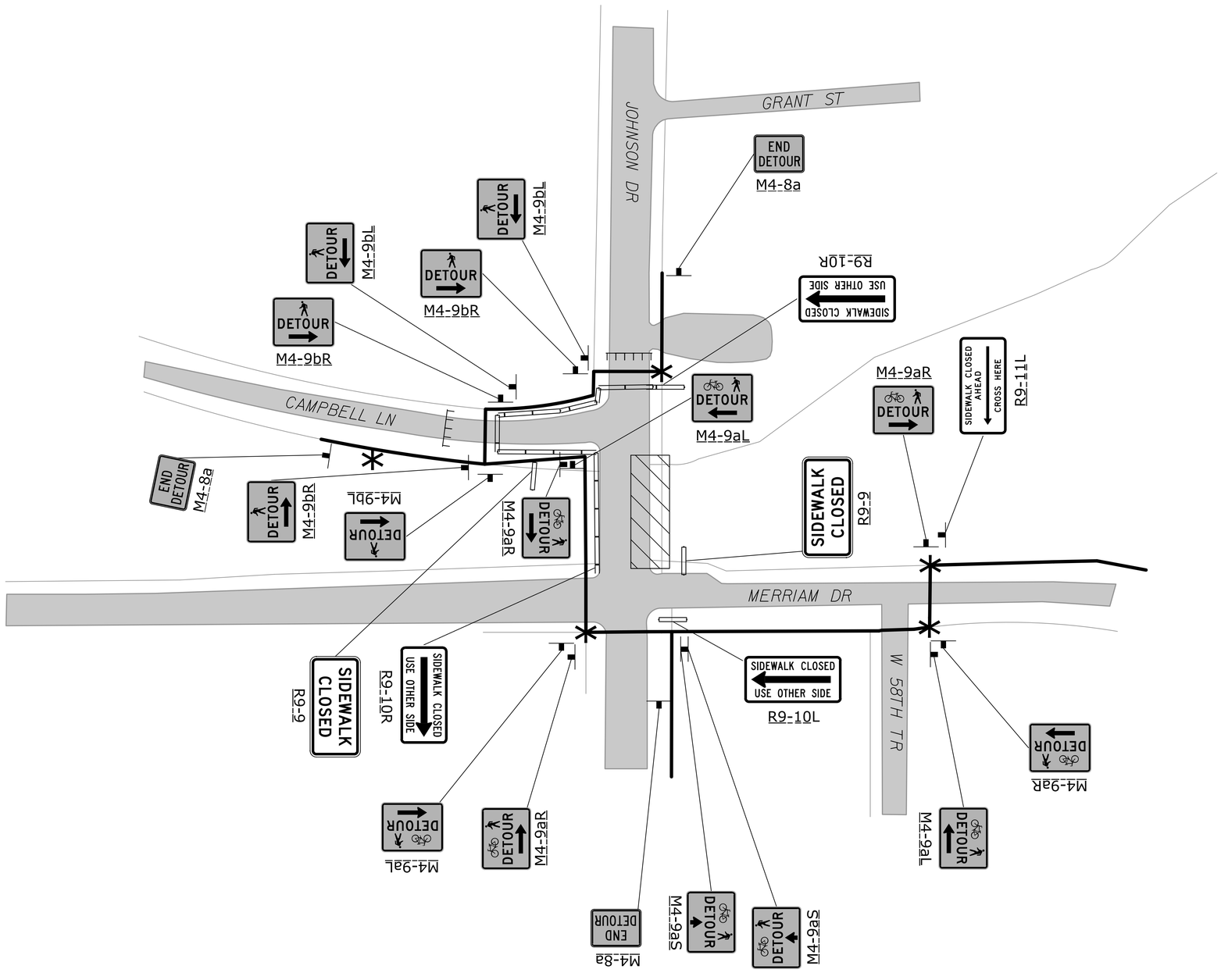
DATE	BY	REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke Plotted :9/13/2013
File : G:\12664\Micro\Traffic\12664_5451.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL
SIDEWALK DETOUR

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	78	99



NOT TO SCALE

LEGEND

- ADA COMPLIANT TEMPORARY PEDESTRIAN BARRICADE
- COMPLETE CLOSURE USING TYPE III BARRICADES
- SIGN
- WORK AREA
- PROPOSED SIDEWALK DETOUR
- AUDIBLE INFORMATION DEVICE

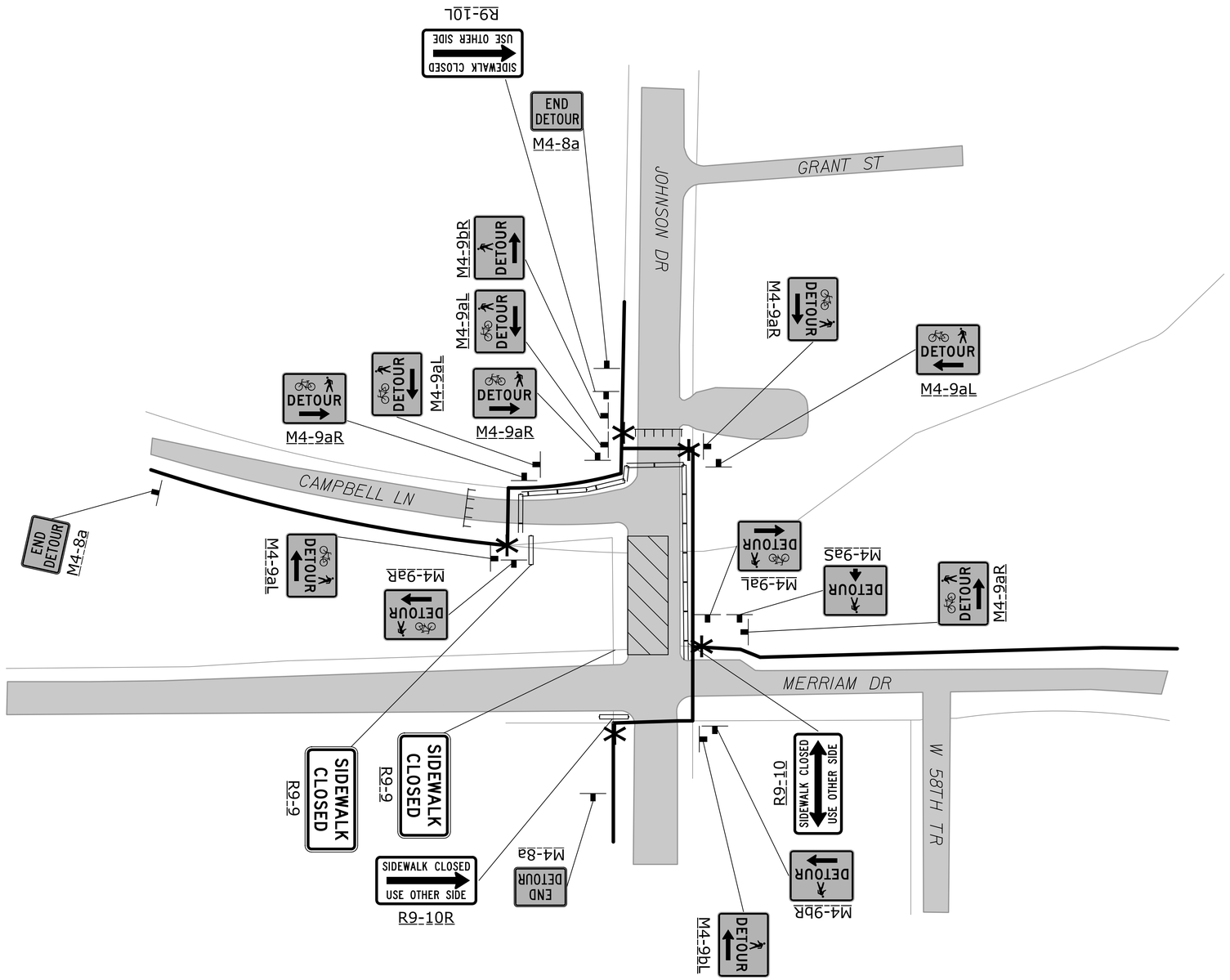
DATE	BY	
		REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke
File : G:\12664\MicroTraffic\12664_5452.dgn
Plotted : 9/13/2013

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL
SIDEWALK DETOUR
SEQUENCE I

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	79	99



NOT TO SCALE

LEGEND

- ADA COMPLIANT TEMPORARY PEDESTRIAN BARRICADE
- COMPLETE CLOSURE USING TYPE III BARRICADES
- TEMPORARY SIGN
- WORK AREA
- PROPOSED SIDEWALK DETOUR
- AUDIBLE INFORMATION DEVICE

DATE	BY	REFERENCES NOTED
		REFERENCES CHECKED

Drawn By : cluedke Plotted : 9/13/2013
File : G:\12664\Micro\Traffic\12664_5453.dgn

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL
SIDEWALK DETOUR
SEQUENCE 2

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	80	99

1. MUTCD COMPLIANCE:

ALL TEMPORARY TRAFFIC CONTROL DEVICES AND THEIR INSTALLATION AND MAINTENANCE SHALL COMPLY WITH THE LATEST EDITION OF THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (MUTCD) FOR STREETS AND HIGHWAYS WHICH HAS BEEN ADOPTED BY THE SECRETARY OF TRANSPORTATION. WHENEVER THE TEMPORARY TRAFFIC CONTROL STANDARDS CONFLICT WITH THE MUTCD, THE STANDARDS SHALL GOVERN.

2. DESIGN SPEED:

THOSE ITEMS DELEGATED TO TEMPORARY TRAFFIC CONTROL SHOULD BE DESIGNED AND INSTALLED USING THE POSTED/LEGAL SPEED OF THE ROADWAY PRIOR TO WORK STARTING.

3. CLEAR ZONE:

ALL CONSTRUCTION EQUIPMENT (INCLUDING VEHICLES), MATERIALS, AND DEBRIS SHALL BE STORED OUT OF THE CLEAR ZONE. WHERE THIS CANNOT BE ACHIEVED, THE CONTRACTOR SHALL PLACE APPROPRIATE SIGNS, OBJECT IDENTIFIERS, AND/OR BARRICADES AS DESIGNATED BY THE ENGINEER. TEMPORARY TRAFFIC CONTROL DEVICES NEEDED FOR THIS CONDITION SHALL BE CONSIDERED SUBSIDIARY TO OTHER BID ITEMS.

4. MINIMUM LANE WIDTHS:

LANE WIDTHS SHALL BE A MINIMUM OF 11' (MEASURED BETWEEN CENTERLINES OF PAVEMENT MARKINGS) OR AS SHOWN ON THE PLANS, OR AS DIRECTED BY THE ENGINEER. A LANE WIDTH LESS THAN 11' MAY REQUIRE RESTRICTED ROADWAY WIDTH SIGNING.

5. FLAGGER:

A MINIMUM OF ONE FLAGGER SHALL BE STATIONED WITHIN EACH MULTI-LANE ROADWAY ACTIVITY AREA WHERE WORK IS IN A CLOSED LANE ADJACENT TO TRAFFIC AND NOT SEPARATED BY A CONCRETE SAFETY BARRIER SYSTEM.

6. PAVEMENT MARKING:

WHEN THE WORK WILL OCCUPY A LOCATION MORE THAN THREE DAYS, ALL CONFLICTING PAVEMENT MARKINGS SHALL BE REMOVED OR MASKED AND ALL TRANSITION TAPERS, CROSSOVERS, AND EDGE LINES ALONG CHANNELIZING DEVICES SHALL BE MARKED WITH SOLID 4" WIDE PAVEMENT MARKING.

7. FIRST MODULE OF IBS:

THE FIRST MODULE OF EACH INERTIAL BARRIER SYSTEM (IBS) SHALL HAVE A MINIMUM OF 2 SQ. FT. OF FLUORESCENT ORANGE ASTM TYPE IV SHEETING FACING TRAFFIC. EITHER A VERTICAL RECTANGLE OR DIAMOND SHAPE MAY BE USED.

8. PEDESTRIAN / BICYCLE SAFETY:

WORK ZONE SIGNS SHALL NOT INHIBIT PEDESTRIAN AND BICYCLE TRAFFIC ON SIDEWALKS OR OTHER AREAS DESIGNATED FOR PEDESTRIAN OR BICYCLE USE.

CONSIDERATION SHOULD BE MADE TO SEPARATE PEDESTRIAN AND BICYCLE MOVEMENTS FROM BOTH WORK SITE ACTIVITY AND VEHICULAR TRAFFIC. UNLESS A REASONABLE SAFE ROUTE THAT DOES NOT INVOLVE CROSSING THE ROADWAY CAN BE PROVIDED, PEDESTRIANS AND BICYCLISTS SHOULD BE APPROPRIATELY DIRECTED WITH ADVANCE SIGNING THAT ENCOURAGES THEM TO CROSS TO THE OPPOSITE SIDE OF THE ROADWAY. IN URBAN AND SUBURBAN AREAS WITH HIGH VEHICULAR TRAFFIC VOLUMES, THESE SIGNS SHOULD BE PLACED AT INTERSECTIONS (RATHER THAN MIDBLOCK LOCATIONS) SO THAT PEDESTRIANS AND BICYCLISTS ARE NOT CONFRONTED WITH MIDBLOCK WORK SITES THAT WILL INDUCE THEM TO ATTEMPT SKIRTING THE WORK SITE OR MAKING A MIDBLOCK CROSSING.

WHEN EXISTING PEDESTRIAN FACILITIES ARE DISRUPTED, CLOSED, OR RELOCATED, THE TEMPORARY FACILITIES SHALL BE DETECTABLE AND INCLUDE ACCESSIBILITY FEATURES CONSISTENT WITH THE FEATURES PRESENT IN THE EXISTING PEDESTRIAN FACILITY.

9. CHANGED STOP CONDITIONS:

ATTACH TWO FLUORESCENT RED-ORANGE FLAGS AND A RED TYPE "B" HIGH INTENSITY WARNING LIGHT TO ANY STOP SIGN THAT CREATES A NEW STOP CONDITION OR MOVES THE STOP CONDITION TO A NEW LOCATION. LEAVE FLAGS AND LIGHTS IN PLACE FOR AT LEAST THE FIRST 30 DAYS. INSTALL W3-1 (SYMBOLIC STOP AHEAD) SIGN IN ADVANCE OF STOP SIGN IF STOP SIGN IS NOT VISIBLE FOR A MINIMUM OF DISTANCE 'A' (SEE CHART ON TE710) OR IF STOP CONDITION IS MOVED TO LESS THAN DISTANCE 'A' FROM AN EXISTING STOP AHEAD SIGN.

10. LUMP SUM BIDDING:

WHEN TRAFFIC CONTROL IS BID LUMP SUM, ADDITIONAL DEVICES WILL BE PAID FOR AS EXTRA WORK.

11. NIGHTTIME LIGHTING:

WHEN NIGHTTIME WORK IS REQUIRED, FLOODLIGHTS SHOULD BE USED TO ILLUMINATE FLAGGER STATIONS, EQUIPMENT CROSSINGS, AND OTHER AREAS WHERE EXISTING LIGHTING IS NOT ADEQUATE FOR THE WORK TO BE PERFORMED SAFELY.

IN NO CASE SHALL FLOODLIGHTS BE PERMITTED TO CREATE A DISABLING GLARE FOR THE DRIVER. THE ADEQUACY OF THE FLOODLIGHT PLACEMENT AND ELIMINATION OF POTENTIAL GLARE SHOULD BE CHECKED BY DRIVING THROUGH THE PROJECT.

12. NCHRP REPORT 350 CRASHWORTHY REQUIREMENTS:

TRAFFIC CONTROL DEVICES SHALL MEET THE EVALUATION CRITERIA IN NCHRP REPORT 350 OR IN MASH REPORT 2009 AS SUPPLEMENTED BY FHWA MEMORANDUM "IDENTIFYING ACCEPTABLE HIGHWAY SAFETY FEATURES," DATED JULY 25, 1997. AVAILABLE ON THE INTERNET AT http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/policy_memo/

ANY DEVICE NOT ADDRESSED BY THE TE STANDARDS MAY BE APPROVED ON A CASE BY CASE BASIS BY THE ENGINEER. THE DEVICE SHALL BE ACCOMPANIED BY AND INSTALLED ACCORDING TO MASH REPORT 2009. ANY DEVICE ACCEPTED PRIOR TO THE ADOPTION OF MASH REPORT 2009 USING CRITERIA FROM NCHRP REPORT 350 MAY REMAIN IN PLACE AND CONTINUE TO BE USED. ANY TRAFFIC CONTROL DEVICE ACCEPTED USING NCHRP REPORT 350 CRITERIA IS NOT REQUIRED TO BE TESTED UNDER MASH REPORT 2009. HOWEVER, NEW TRAFFIC CONTROL DEVICES NOT PREVIOUSLY EVALUATED MUST UTILIZE MASH REPORT 2009 FOR TESTING AND EVALUATION.

THE CONTRACTOR SHALL:

1) PROVIDE TO THE ENGINEER A COPY OF THE MANUFACTURER'S SELF-CERTIFICATION THAT ANY CATEGORY 1 (i.e. - PLASTIC CONICAL DELINEATORS, TUBULAR MARKERS, DRUMS WITHOUT ATTACHMENTS) AND CATEGORY 2 (i.e. - PORTABLE SIGN STANDS (WITH SIGNS), TYPE II AND III BARRICADES, AND VERTICAL PANELS) DEVICES USED ON THE PROJECT ARE NCHRP REPORT 350 OR MASH REPORT 2009 COMPLIANT.

2) PROVIDE TO THE ENGINEER A COPY OF THE ENTIRE FHWA ACCEPTANCE LETTER (WZ-xxx) FOR ANY CATEGORY 2 DEVICE (i.e. - PORTABLE SIGN STANDS (WITH SIGNS), TYPE II AND III BARRICADES, AND VERTICAL PANELS) USED ON THE PROJECT. WORK ZONE FHWA ACCEPTANCE LETTERS (WZ-xxx) ARE AVAILABLE ON THE INTERNET AT: http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/wzd/

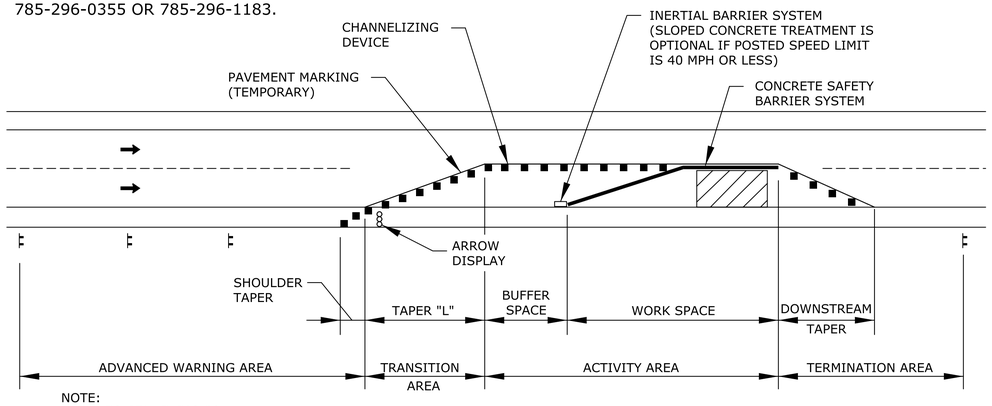
3) CERTIFY THAT THE TRUCK MOUNTED ATTENUATORS (TMA'S) (WHICH ARE DEFINED AS CATEGORY 3 DEVICES BY THE FHWA MEMORANDUM) MEET CURRENT CRASHWORTHY SPECIFICATIONS AS DEFINED ABOVE AND INCLUDE A COPY OF THE ENTIRE FHWA ACCEPTANCE LETTER. ALL CATEGORY 1 & 2 DEVICES SHALL BE NCHRP REPORT 350 OR MASH REPORT 2009 COMPLIANT.

13. LEAD IN CHANNELIZING DEVICES ON CENTERLINE:

TEMPORARY RUMBLE STRIPS MAY BE USED IN LIEU OF LEAD IN CENTERLINE CHANNELIZING DEVICES WHEN THE ROADWAY IS LESS THAN OR EQUAL TO 30' (FEET) INCLUDING PAVED SHOULDERS. WHEN EXTENUATING CIRCUMSTANCES EXIST, THE AREA ENGINEER MAY ELECT TO ELIMINATE BOTH THE LEAD IN CHANNELIZERS AND THE RUMBLE STRIPS.

14. TEMPORARY RUMBLE STRIPS:

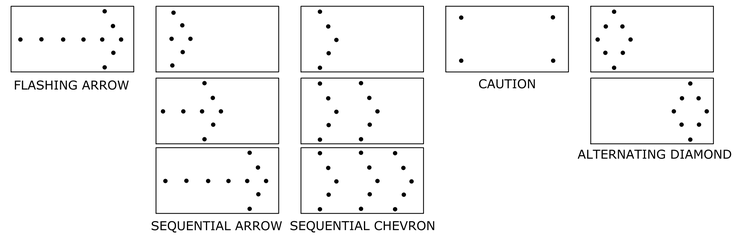
ALTERNATIVE TEMPORARY RUMBLE STRIP OPTIONS MAY BE AVAILABLE. PLEASE CONTACT THE TEMPORARY TRAFFIC CONTROL UNIT FOR MORE INFORMATION AT 785-296-0355 OR 785-296-1183.



NOTE:
REFER TO STD. TE702 FOR TAPER "L" FORMULA.

TYPICAL WORK ZONE COMPONENTS

ARROW DISPLAYS



ARROW DISPLAY ELEMENTS SHALL BE CAPABLE OF A MINIMUM 50 PERCENT DIMMING FROM THEIR FULL-RATED LAMP VOLTAGE. FULL LAMP VOLTAGE SHOULD BE USED DURING THE DAY AND DIMMED MODE SHALL BE USED AT NIGHT. FOR SHOULDER WORK, ROADSIDE WORK NEAR THE SHOULDER, BLOCKING THE SHOULDER, OR FOR TEMPORARY CLOSING ONE LANE ON A TWO-LANE, TWO-WAY ROADWAY, AN ARROW PANEL SHALL BE USED ONLY IN THE CAUTION MODE.

BUFFER SPACE

SPEED (MPH) ✳	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*	*65*	*70*	*75*
LENGTH (ft)	*115*	*155*	*200*	*250*	*305*	*360*	*425*	*495*	*570*	*645*	*730*	*820*

✳ POSTED SPEED PRIOR TO WORK STARTING

NEITHER WORK ACTIVITY NOR STORAGE OF EQUIPMENT, VEHICLES, OR MATERIAL SHOULD OCCUR IN THE BUFFER SPACE. WHEN A PROTECTION VEHICLE IS PLACED IN ADVANCE OF THE WORK SPACE, ONLY THE SPACE UPSTREAM OF THE VEHICLE CONSTITUTES THE BUFFER SPACE.

IF TEMPORARY CONCRETE SAFETY BARRIER SYSTEM IS USED TO SEPARATE APPROACHING TRAFFIC FROM THE WORK SPACE, THE BARRIER SYSTEM SHALL BE CONSIDERED PART OF THE ACTIVITY AREA. A FULL LANE WIDTH SHOULD BE AVAILABLE THROUGHOUT THE LENGTH OF THE BUFFER SPACE. SEE TYPICAL WORK ZONE COMPONENTS.

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Removed Note 13. Added Alternating Diamonds	J.A.M.	K.P.
2	10/4/11	Modified Notes 9,12 & 15. Added Note 15	J.A.M.	K.P.
1	11/30/09	Added Note 14	J.A.M.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

GENERAL TRAFFIC CONTROL

TE700

FHWA APPROVAL		10/16/12	APP'D	Kristina Pyle
DESIGNED B.A.H.	DETAILED B.A.H.		QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.		QUAN. CK.	TRACE CK.

KDOT Graphics Certified 10-23-2012

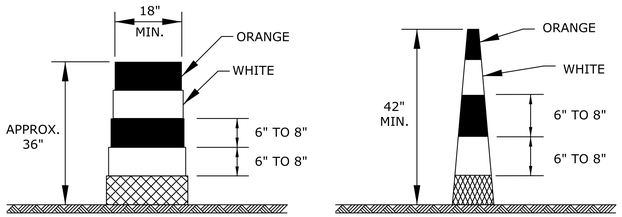
Sh. No. 80

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te700.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	81	99

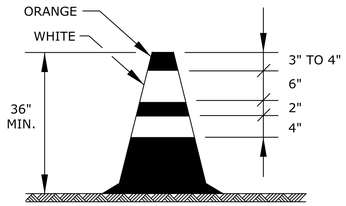


DRUM

CONICAL DELINEATOR

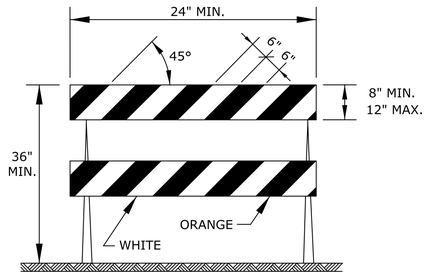
DRUMS AND CONICAL DELINEATORS SHALL HAVE AT LEAST TWO ORANGE AND TWO WHITE 6" TO 8" WIDE RETROREFLECTIVE STRIPES. ADDITIONAL STRIPES MAY BE NON-RETROREFLECTIVE. IF THERE ARE NON-RETROREFLECTIVE SPACES BETWEEN ADJACENT STRIPES, THEY SHALL BE NO MORE THAN 3" WIDE.

ALL RETROREFLECTIVE STRIPES ON DRUMS SHALL BE ASTM TYPE III SHEETING. THE WHITE STRIPES ON CONICAL DELINEATORS SHALL BE ASTM TYPE III SHEETING. ORANGE STRIPES ON ALL CONICAL DELINEATORS SHALL BE FLUORESCENT ORANGE ASTM TYPE IV SHEETING.



TRAFFIC CONE

TRAFFIC CONES MAY BE USED AS CHANNELIZING DEVICES FOR DAYTIME OPERATIONS ONLY. THEY WILL NOT BE PAID FOR SEPARATELY, BUT WILL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS. THE ENGINEER MAY REQUIRE THAT TRAFFIC CONES BE SUPPLEMENTED BY OTHER TRAFFIC CONTROL DEVICES IN CERTAIN SITUATIONS.

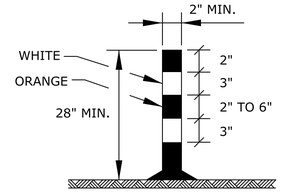


TYPE II BARRICADE

FOR RAILS LESS THAN 36" LONG, 4" WIDE STRIPES MAY BE USED.

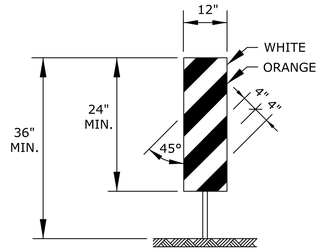
THE ENTIRE AREA OF BARRICADE RAILS, BOTH FRONT AND BACK, SHALL BE ASTM TYPE III SHEETING.

THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.



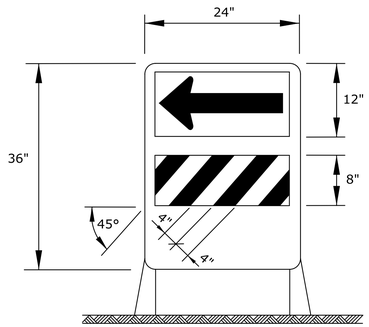
TUBULAR MARKER

THE TWO WHITE RETROREFLECTIVE STRIPES SHALL BE ASTM TYPE III SHEETING. STRIPING AS SHOWN FOR UP TO 42".



VERTICAL PANEL

THE ENTIRE AREA OF VERTICAL PANELS, BOTH FRONT AND BACK, SHALL HAVE ASTM TYPE III SHEETING. THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.



DIRECTION INDICATOR BARRICADE

THE ARROW PANEL SHALL BE BLACK ON FLUORESCENT ORANGE ASTM TYPE IV SHEETING. THE STRIPES SHALL BE ORANGE AND WHITE ASTM TYPE III SHEETING SLOPING DOWNWARD IN THE DIRECTION TRAFFIC IS TO PASS.

THE DIRECTION INDICATOR BARRICADE SHALL BE USED IN SERIES TO DIRECT THE MOTORIST INTO THE INTENDED LANE OF TRAVEL.

THE ARROW PANEL SHOULD NOT BE VISIBLE TO OPPOSING TRAFFIC.

TAPER FORMULAS:

L = WS FOR SPEEDS OF 45 MPH OR MORE

$L = WS^2/60$ FOR SPEEDS OF 40 MPH OR LESS

WHERE: L = MINIMUM LENGTH OF TAPER IN FEET
S = NUMERICAL VALUE OF POSTED SPEED PRIOR TO WORK STARTING IN MPH
W = WIDTH OF OFFSET IN FEET

CHANNELIZER PLACEMENT:

(A) THE SPACING BETWEEN DEVICES IN TRANSITION AREA (TAPER) SHOULD NOT EXCEED A DISTANCE IN FEET EQUAL TO 1/2 THE POSTED SPEED LIMIT IN MPH PRIOR TO WORK STARTING.

(B) THE SPACING BETWEEN DEVICES IN THE ADVANCED WARNING AREA AND THE ACTIVITY AREA SHOULD NOT EXCEED A DISTANCE IN FEET EQUAL TO TWO TIMES THE POSTED SPEED LIMIT IN MPH PRIOR TO WORK STARTING.

(C) CHANNELIZING DEVICES SHALL BE PLACED FOR OPTIMUM VISIBILITY, NORMALLY AT RIGHT ANGLES TO THE TRAFFIC FLOW.

(D) CHANNELIZING DEVICES PLACED ALONG SHOULDER EDGES OR IN DROPOFFS SHALL HAVE A MINIMUM OF 24" FROM THE TOP OF THE CHANNELIZING DEVICE TO THE TOP OF THE PAVEMENT.

ITEM / LOCATION		CROSS-OVERS	SHOOFLY DIVERSIONS	TANGENTS	TAPERS	RAMPS	HEAD TO HEAD	OBJECT IDENTIFIER	LEAD IN DEVICES	GORES
PORTABLE										
	DRUMS	YES	YES	YES	YES	YES	(1)	YES	YES	YES
	CONICAL DELINEATORS	YES	YES	YES	YES	YES	(1)	YES	YES	YES
	VERTICAL PANELS	(2)	(2)	(2)	(2)	(2)	(1,2)	YES	(2)	(2)
	DIRECTION INDICATOR BARRICADE	NO	NO	NO	YES	NO	NO	NO	NO	NO
	TYPE II BARRICADE	(2)	(2)	(2)	(2)	NO	NO	YES	NO	NO
FIXED										
	TUBULAR MARKERS	(3)	(3)	(3)	NO	(3)	YES	NO	YES	YES
	VERTICAL PANELS	(3)	(3)	(3)	(3)	(3)	(3)	YES	(2,3)	(2)

(1) NOT ALLOWED ON CENTERLINE DELINEATION ALONG FREEWAYS OR EXPRESSWAYS.

(2) THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.

(3) MAY BE USED UPON THE APPROVAL OF THE ENGINEER.

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Added Lead In Devices Into Matrix Table	J.A.M.	K.P.
2	10/4/11	Added Dimension To Tubular Marker Detail	J.A.M.	K.P.
1	4/20/09	Channelizer Placement & Traffic Cone Detail	J.A.M.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

CHANNELIZING DEVICES

TE702

FHWA APPROVAL		10/16/12	APP'D	Kristina Pyle	
DESIGNED	L.E.R.	DETAILED	B.A.H.	QUANTITIES	TRACED
DESIGN CK.		DETAIL CK.		QUAN. CK.	TRACE CK.

KDOT Graphics Certified 10-23-2012

KDOT Graphics Certified

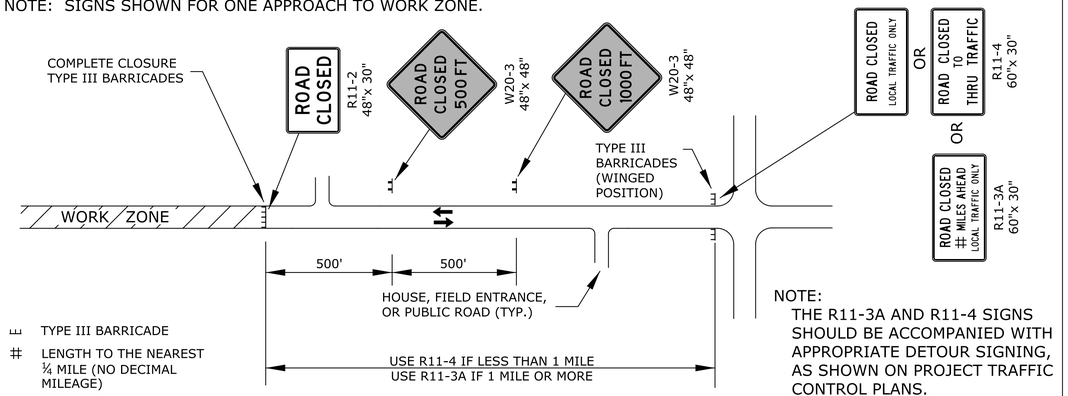
Sh. No. 81

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te702.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	82	99

NOTE: SIGNS SHOWN FOR ONE APPROACH TO WORK ZONE.



NOTE:
THE R11-3A AND R11-4 SIGNS SHOULD BE ACCOMPANIED WITH APPROPRIATE DETOUR SIGNING, AS SHOWN ON PROJECT TRAFFIC CONTROL PLANS.

FIGURE 1: TYPICAL SIGNING FOR ROAD CLOSURE

NOTE: SIGNS SHOWN FOR ONE APPROACH TO INTERSECTION (WORK ZONE).

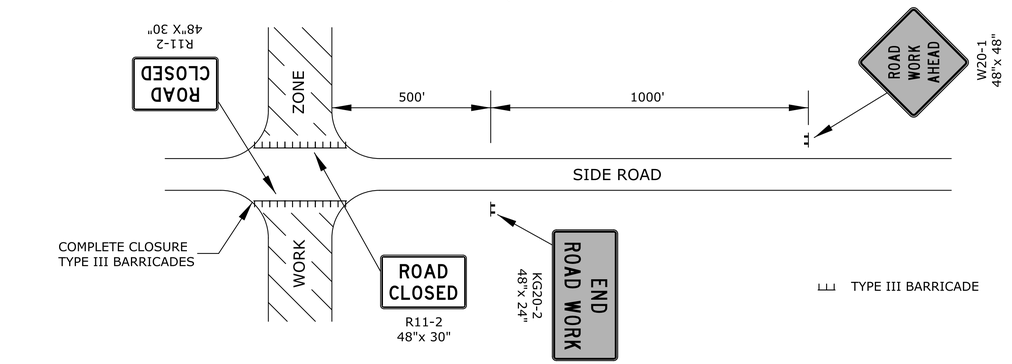


FIGURE 2: TYPICAL SIGNING FOR SIDE ROAD OPEN

NOTE: SIGNS SHOWN FOR ONE APPROACH TO INTERSECTION (WORK ZONE).

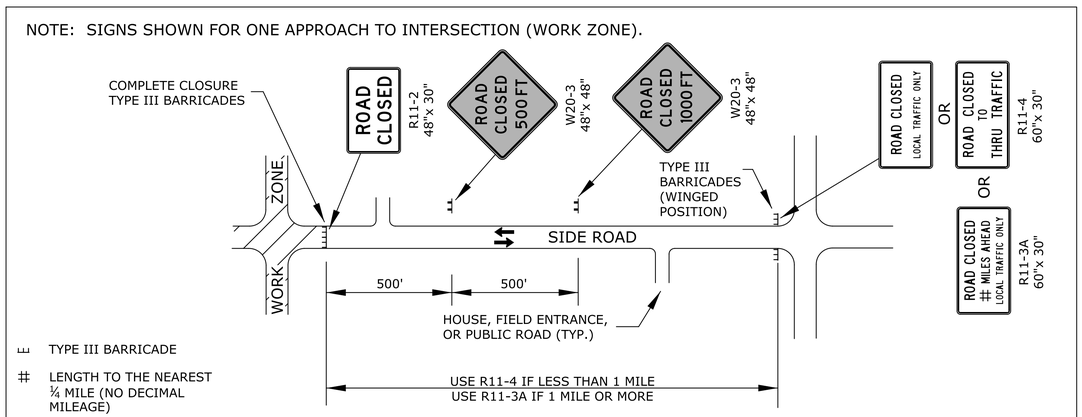


FIGURE 3: TYPICAL SIGNING FOR SIDE ROAD CLOSED

NOTE: SIGNS SHOWN FOR ONE APPROACH TO WORK ZONE.

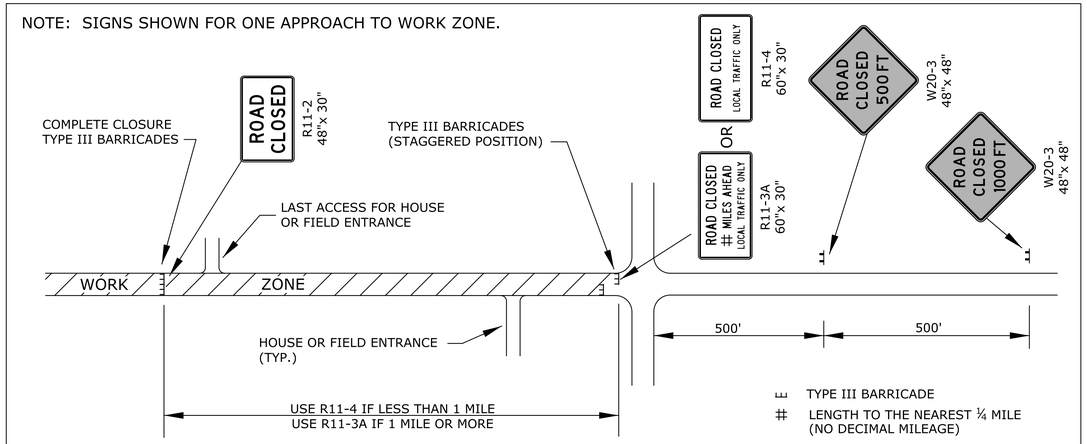


FIGURE 4: TYPICAL SIGNING FOR ROAD CLOSURE - LOCAL TRAFFIC ACCESS

NOTES:

1. SIGNS:

THE R11-4 (ROAD CLOSED TO THRU TRAFFIC OR ROAD CLOSED LOCAL TRAFFIC ONLY) SIGN SHALL BE USED WHEN THE DISTANCE TO THE POINT OF COMPLETE CLOSURE OF THE ROADWAY IS LESS THAN 1 MILE.

THE R11-3A (ROAD CLOSED # MILES AHEAD LOCAL TRAFFIC ONLY) SIGN SHALL BE USED WHEN THE DISTANCE TO THE POINT OF COMPLETE CLOSURE OF THE ROADWAY IS 1 MILE OR GREATER.

THE WORDS "BRIDGE OUT" (OR BRIDGE CLOSED) MAY BE SUBSTITUTED FOR THE WORDS "ROAD CLOSED" ON THE R11-3A OR R11-4 SIGN WHERE APPLICABLE.

2. BARRICADE PLACEMENT:

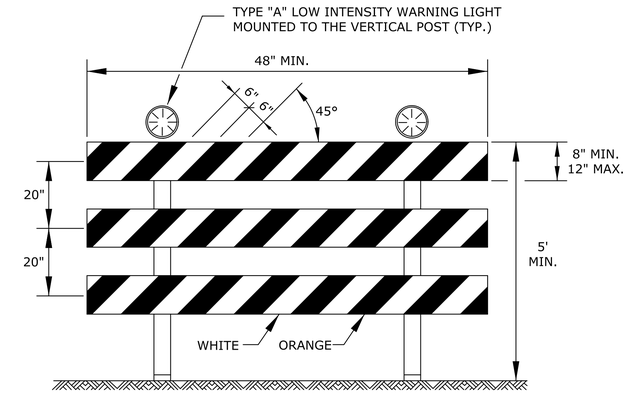
A) COMPLETE ROAD CLOSURE

WHEN A ROADWAY IS CLOSED, TYPE III BARRICADES SHALL BE PLACED END-TO-END TO COMPLETELY COVER THE ROADWAY AND SHOULDERS. WHEN ACCESS MUST BE ALLOWED FOR CONSTRUCTION OR OTHER OFFICIAL/GOVERNMENT VEHICLES, TYPE III BARRICADES SHALL BE LONGITUDINALLY STAGGERED FAR ENOUGH APART FROM ONE ANOTHER TO ALLOW SAFE PASSAGE OF VEHICLES AND MAINTAIN THE APPEARANCE OF A CLOSED ROADWAY. TYPE III BARRICADES SHALL BE REALIGNED AND PLACED END-TO-END TO DENY ANY ACCESS WHEN THE CONSTRUCTION ACTIVITY HAS CEASED FOR THE DAY.

B) ROAD CLOSED - LOCAL TRAFFIC

AS SHOWN IN FIGURE 4, WHEN LOCAL TRAFFIC MUST BE ALLOWED ACCESS INTO THE WORK ZONE, TYPE III BARRICADES SHALL BE LONGITUDINALLY STAGGERED TO MAINTAIN THE APPEARANCE OF A CLOSED ROADWAY. A SECOND LINE OF END-TO-END TYPE III BARRICADES SHALL BE PLACED JUST BEYOND THE LAST ACCESS POINT IN THE WORK ZONE, TO COMPLETELY CLOSE THE ROADWAY AS DESCRIBED IN NOTE 2-A.

AS SHOWN IN FIGURE 1 AND FIGURE 3, AT THE POINT WHERE THRU TRAFFIC MUST DETOUR AND LOCAL TRAFFIC CAN PROCEED TO THE LOCATION WHERE THE ROADWAY IS COMPLETELY CLOSED, THE R11-3A (ROAD CLOSED # MILES AHEAD LOCAL TRAFFIC ONLY) OR R11-4 (ROAD CLOSED LOCAL TRAFFIC ONLY OR ROAD CLOSED TO THRU TRAFFIC) SIGN SHALL BE USED WITH TYPE III BARRICADES (WINGED POSITION), PLACED ON THE SHOULDERS OF ROADWAY.



TYPE III BARRICADE WITH LIGHTS

THE ENTIRE AREA OF BARRICADE RAILS, BOTH FRONT AND BACK, SHALL HAVE ASTM TYPE III SHEETING.

THE STRIPES SHALL SLOPE DOWNWARD TO THE SIDE TRAFFIC IS TO PROCEED OR TOWARD THE CENTER OF THE ROADWAY AT ROAD CLOSURES.

APPROVED SIGNS MOUNTED ON TYPE III BARRICADES SHOULD NOT COVER MORE THAN 50% OF THE TOP TWO RAILS OR 33% OF THE TOTAL AREA OF THE THREE RAILS.

WHEN BARRICADES ARE PLACED END-TO-END OR STAGGERED, A TYPE "A" LOW INTENSITY WARNING LIGHT SHALL BE MOUNTED TO THE VERTICAL POST NEAR EACH OUTSIDE CORNER OF THE END BARRICADES.

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Modified Type III Barricade Note	J.A.M.	K.P.
2	8/8/07	Added Position To Type III Barricade	M.B.	A.A.A.
1	12/29/05	Note #1 Modified	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL TRAFFIC CONTROL
ROAD CLOSURES

TE704

FHWA APPROVAL		10/16/12	APP'D	Kristina Pyle			
DESIGNED	B.A.H.	DETAILED	B.A.H.	QUANTITIES		TRACED	
DESIGN CK.		DETAIL CK.		QUAN. CK.		TRACE CK.	

KDOT Graphics Certified 10-23-2012

Sh. No. 82

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te704.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	83	99

SIGN LAYOUT INFORMATION



KG20-2

STD. SIZE EXPWY/FREEWAY
6" C
48"x 24"



KG20-5

STD. SIZE EXPWY/FREEWAY
6" C
48"x 24"



KM4-2O

STD. SIZE	EXPWY/FREEWAY
3" C	6" C
24"x 6"	48"x 12"



W7-3a

MILEAGE TO BE DETERMINED BY THE ENGINEER.



W8-11

STD. SIZE EXPWY/FREEWAY
8" D
48"x 48"



W8-17

STD. SIZE EXPWY/FREEWAY
48"x 48"



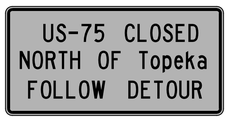
W8-17P
(OPTIONAL)

STD. SIZE EXPWY/FREEWAY
30"x 24"



SP-01
(SPECIAL SIGN)

STD. SIZE	EXPWY/FREEWAY
6" C	10" D



SP-02
(SPECIAL SIGN)

STD. SIZE	EXPWY/FREEWAY
UPPERCASE: 6" C	UPPERCASE: 10" D
LOWERCASE: 4.5" C	LOWERCASE: 8" D

ALL CITY NAMES AND STREET NAMES ON SPECIAL SIGNS AND DESTINATION SIGNS MUST HAVE UPPER AND LOWER CASE LETTERS.

ALL SIGNS SHALL BE BLACK ON ORANGE RETROREFLECTIVE SHEETING.

GENERAL NOTES

1. MAINTENANCE:

THE CONTRACTOR SHALL MAINTAIN ALL SIGNS AND DEVICES IN AN UPRIGHT POSITION. THE CONTRACTOR SHALL CLEAN OR REPLACE ANY DAMAGED OR ILLEGIBLE SIGN OR DEVICE AS DIRECTED BY THE ENGINEER.

2. EXISTING SIGNS:

IF EXISTING SIGNS THAT ARE TO REMAIN (WHETHER DENOTED ON THE PLANS OR NOT) INTERFERE WITH CONSTRUCTION WORK, THE CONTRACTOR SHALL REMOVE, STORE, AND RESET THE SIGNS. THIS SHALL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS. SIGNING DAMAGED BY THE CONTRACTOR SHALL BE REPLACED AT THE CONTRACTOR'S EXPENSE.

3. CONFLICTING SIGNS, SIGNS NOT IN USE, AND TRAFFIC SIGNALS:

SIGNS AND TRAFFIC SIGNALS THAT ARE IN CONFLICT WITH THE TRAFFIC CONTROL PLAN OR DO NOT APPLY TO THE TRAFFIC OPERATIONS SHALL BE IMMEDIATELY REMOVED, TURNED SO NOT VISIBLE TO TRAFFIC FROM ANY DIRECTION, OR COMPLETELY COVERED WITH ADEQUATE OPAQUE BREATHABLE MATERIAL. TAPE SHALL NOT BE APPLIED TO THE FACE OF THE SIGN.

4. PORTABLE AND POST MOUNTED SIGNS:

TEMPORARY TRAFFIC CONTROL SIGNS THAT ARE ANTICIPATED TO REMAIN IN PLACE FOR 3 DAYS OR LESS ARE CONSIDERED "PORTABLE." PORTABLE SIGNS SHALL BE MOUNTED ON AN APPROVED SUPPORT AT A MINIMUM HEIGHT OF 12" ABOVE THE TRAVELED WAY. TRAFFIC CONTROL SIGNS IN PLACE FOR OVER 3 DAYS ARE REQUIRED TO BE MOUNTED ON APPROVED POSTS. A MINIMUM OF 42" OF THE APPROVED POST MUST BE BELOW THE GROUND SURFACE WITH ADEQUATE BACKFILL AND COMPACTION. ALL POSTS AT MINIMUM SHALL EXTEND TO THE TOP EDGE OF THE SIGN AND NO GREATER THAN 6" ABOVE THE SIGN.

WHEN THE SIGN WIDTH IS EQUAL TO OR GREATER THAN 9', THREE OR MORE WOOD POSTS MAY BE USED WITH A MINIMUM OF 4' BETWEEN THE CENTERLINE OF EACH POST. ALL SIGNS LESS THAN 9' IN WIDTH SHALL USE A MAXIMUM OF TWO WOOD POSTS.

"ROLL-UP" SIGNS MAY BE USED FOR PORTABLE WARNING SIGNS. THEY MUST BE FLUORESCENT ORANGE ASTM TYPE IV SIGNS OF OPAQUE MATERIAL. MESH SIGNS ARE NOT ALLOWED.

IN THE CASE OF HITTING ROCK WHEN DRIVING POSTS
1. SHIFT THE SIGN LOCATION. DO NOT VIOLATE MINIMUM SIGN SPACING.
2. WITH THE ENGINEER'S APPROVAL, USE ACCEPTABLE ALTERNATIVE SIGN STANDS.

5. SHEETING:

ALL ORANGE SIGNS SHALL HAVE FLUORESCENT ORANGE ASTM TYPE IV SHEETING. ALL OTHER SIGNS SHALL HAVE ASTM TYPE III SHEETING OF STANDARD COLORS.

6. SIGNS INVOLVING SPEEDS:

THE W3-5 (SPEED REDUCTION) SHOULD BE USED ONLY IF THE ENGINEER DETERMINES THAT A REDUCED SPEED IS REQUIRED ON THE PROJECT.

THE KM4-20 (WORK ZONE) PLAQUE SHALL BE PLACED ABOVE ALL SPEED LIMIT SIGNS, (R2-1), EXISTING AND TEMPORARY. MOUNT THE WORK ZONE PLAQUES TO THE POST. DO NOT OVERLAP THE R2-1 AND KM4-20 SIGNS.

FOR SPEEDS OF 30 MPH OR LESS, THE W1-1(TURN) OR W1-3(REVERSE TURN) SHOULD BE USED. FOR SPEEDS OF 35 MPH OR MORE, THE W1-2(CURVE) OR W1-4(REVERSE CURVE) SHOULD BE USED. THE W13-1(MPH) IS TO BE ELIMINATED IF THE ADVISORY SPEED IS WITHIN 5 MPH OF THE SPEED LIMIT.

7. SIGNS CONTROLLING WORK ZONE:

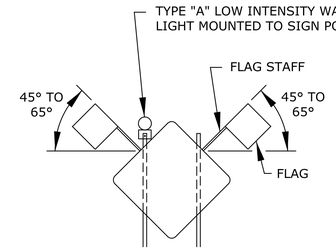
THE KG20-2(END ROAD WORK) SHOULD BE PLACED 500' FROM THE END OF THE ACTUAL WORK SPACE, NOT NECESSARILY AT THE EXTREME LIMITS OF THE PROJECT. THE KG20-2 SHOULD BE MOUNTED ON TWO POSTS. THE KG20-2 MAY BE MOUNTED ON ONE POST IF IN URBAN AREAS WHERE UTILITIES ARE A PROBLEM AND WIND LOADS ARE NOT AN ISSUE.

WHERE TWO WORK ZONES ARE LESS THAN 1 MILE APART IN RURAL AREAS OR ¼ MILE APART IN URBAN AREAS, THE KG20-2(END ROAD WORK) FOR THE FIRST WORK ZONE AND THE W20-1(ROAD WORK) FOR THE SECOND WORK ZONE SHOULD BE ELIMINATED.

8. WARNING LIGHTS ON SIGNS:

A TYPE "A" LOW INTENSITY WARNING LIGHT IS AN L.E.D. BI-DIRECTIONAL FLASHING WORK ZONE WARNING LIGHT. TYPE "A" LOW INTENSITY WARNING LIGHTS SHOULD BE USED WITH ALL CONSTRUCTION ACTION WARNING SIGNS AND SHALL NOT BE USED ON SIGNS MOUNTED LESS THAN 5' HIGH ON TEMPORARY SUPPORTS. ON ALL OTHER CONSTRUCTION WARNING SIGNS, TYPE "A" LOW INTENSITY WARNING LIGHTS ARE TO BE USED AS DIRECTED BY THE ENGINEER.

TYPE "A" LOW INTENSITY WARNING LIGHTS SHALL BE MAINTAINED SO AS TO BE CAPABLE OF BEING VISIBLE ON A CLEAR NIGHT FROM A DISTANCE OF 3000 FT. IF A TYPE "A" LOW INTENSITY WARNING LIGHT HAS A SEPARATE BATTERY CASE, THE BATTERY CASE SHALL BE MOUNTED NO HIGHER THAN 12" ABOVE THE GROUND AND MOUNTED BEHIND THE SIGN POST. A TYPE "A" LOW INTENSITY WARNING LIGHT WHERE THE LENS AND BATTERY ARE ONE UNIT SHALL BE MOUNTED ON THE TEMPORARY SIGN POST NEAREST TO THE TRAVELED WAY. FLAGS SHALL NOT INTERFERE WITH THE VISABILITY OF THE TYPE "A" LOW INTENSITY WARNING LIGHT.



TWO (2) 18" x 18" FLUORESCENT RED-ORANGE FLAGS SHALL BE ATTACHED (IN THE POSITION SHOWN) ON THE W20-2(DETOUR),W1-1(TURN), W1-2(CURVE), W1-3(REVERSE TURN), W1-4(REVERSE CURVE), W3-3(SIGNAL AHEAD), W4-2(LANE REDUCTION), W20-4(ONE LANE ROAD), W20-5(LANE CLOSED), W20-7A(FLAGGER), AND W3-4 (BE PREPARED TO STOP) SIGNS AND ANY OTHER ACTION SIGNS AS SHOWN ON THE PLANS OR DIRECTED BY THE ENGINEER. THE FLAGS AND STAFFS ARE TO BE ATTACHED IN SUCH A MANNER THAT THE SIGN WILL NOT BE OBSCURED. THE FLAGS MAY BE EITHER A CLOTH OR VINYL MATERIAL. THE FLAGS SHALL BE SUBSIDIARY TO THE CONSTRUCTION SIGN BID ITEMS.

MINIMUM ADVANCE WARNING SIGN SPACING (IN FEET):

	A	*B*	*C*
URBAN (40 MPH OR LOWER)	*100*	*100*	*100*
URBAN (45 MPH OR HIGHER)	*350*	*350*	*350*
RURAL (55 MPH OR LOWER)	*500*	*500*	*500*
RURAL (60 MPH OR HIGHER)	*750*	*750*	*750*
EXPRESSWAY/FREEWAY	*1000*	*1500*	*2640*

THE MINIMUM SPACING BETWEEN SIGNS SHALL BE NO LESS THAN 100', UNLESS DIRECTED BY THE ENGINEER.

THE SPACING BETWEEN ANY SIGNS MAY BE INCREASED BEYOND THE MINIMUM VALUES IN THE TABLE ABOVE AS APPROVED BY THE ENGINEER IN ORDER TO MAXIMIZE VISIBILITY.

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Removed Note 9, Modified Sign Layout Detail	J.A.M.	K.P.
2	10/4/11	Modified Note 3	J.A.M.	K.P.
1	2/24/10	Modified AFAD Note	J.A.M.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL SIGNS

TE710

FHWA APPROVAL		10/16/12	APP'D	Kristina Pyle	
DESIGNED	B.A.H.	DETAILED	B.A.H.	QUANTITIES	TRACED
DESIGN CK.		DETAIL CK.		QUAN. CK.	TRACE CK.

KDOT Graphics Certified 10-23-2012

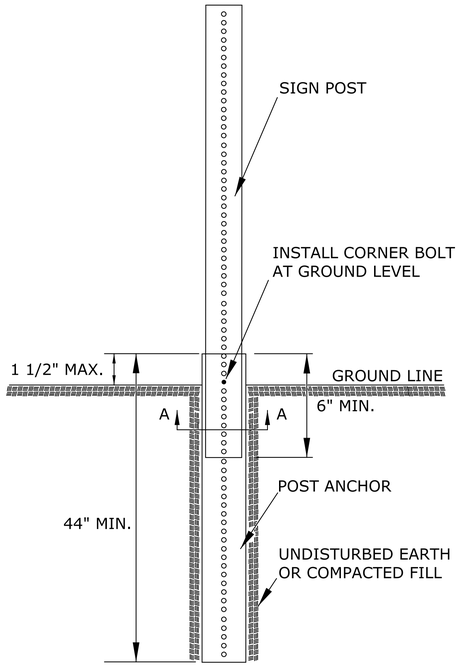
Sh. No. 83

KDOT Graphics Certified

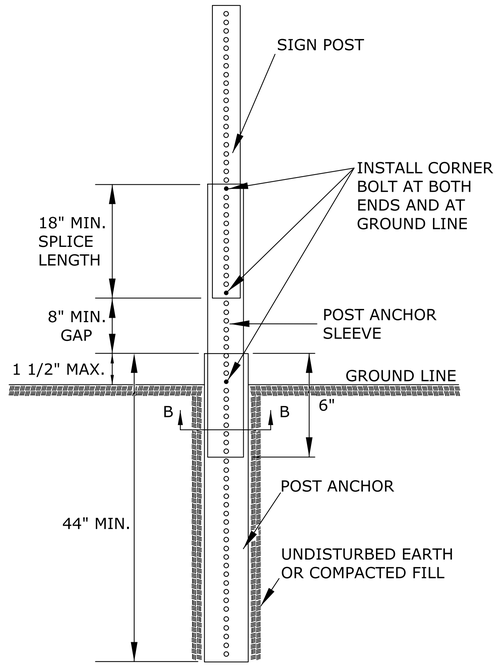
Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te710.dgn

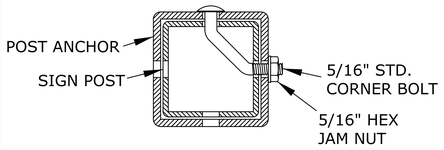
PERFORATED SQUARE STEEL TUBE (P.S.S.T.) POST SETUP



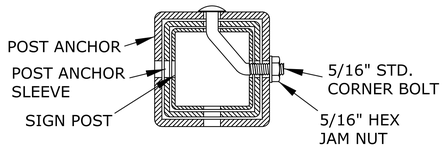
P.S.S.T. DETAIL



TELESCOPING P.S.S.T. DETAIL



SECTION A-A

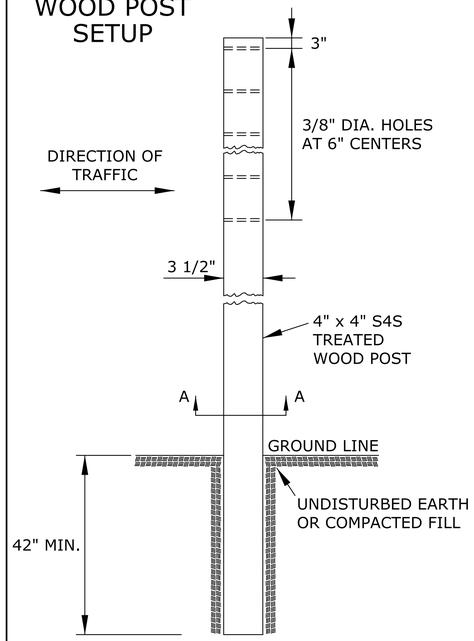


SECTION B-B

DETAILS FOR 2", 2 1/4", OR 2 1/2" SIGN POST

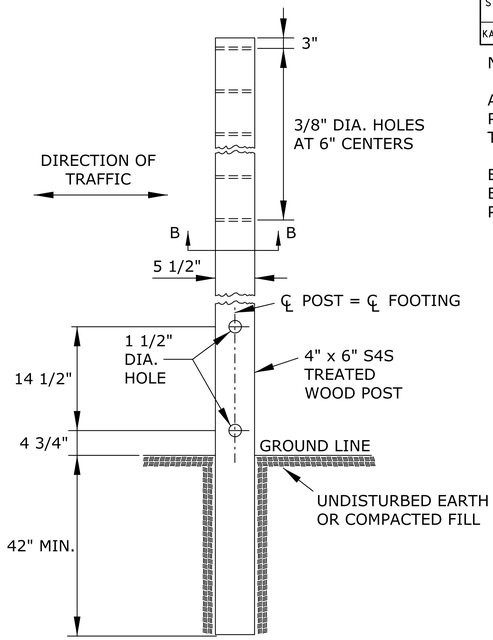
PLACE BOLTS IN THE SAME CORNER ALONG EACH SIGN POST.

WOOD POST SETUP



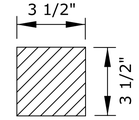
SIDE ELEVATION

4" X 4" WOOD POST IN SOIL

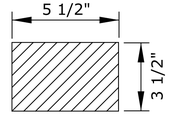


SIDE ELEVATION

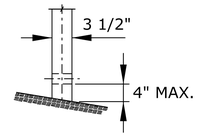
4" X 6" WOOD POST IN SOIL



SECTION A-A



SECTION B-B



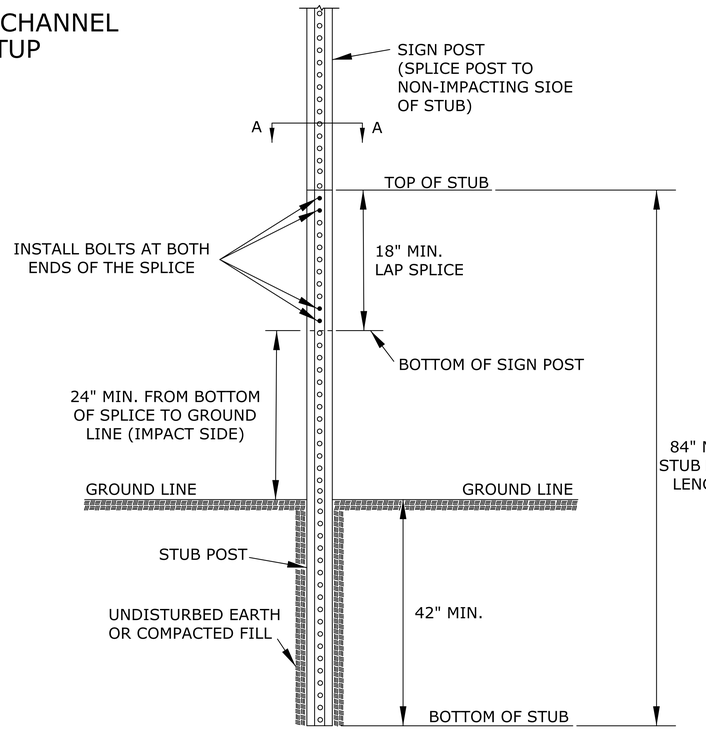
FRONT ELEVATION

NOTES:

ALL SIGN MOUNTING HOLES IN THE WOOD POSTS SHALL BE DRILLED PRIOR TO TREATING.

BREAKAWAY HOLES AND FIELD CUTS SHALL BE TREATED IN ACCORDANCE WITH THE PRESERVATIVE TREATMENT SPECIFICATIONS.

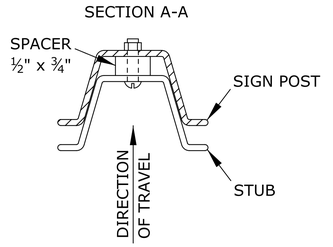
3 LB/F U-CHANNEL SETUP



NOTES:

PLACE TWO BOLTS AT BOTH ENDS OF THE SPLICE THROUGH THE HOLES NEAREST THE ENDS OF THE SPLICE.

SPACERS WILL BE USED OVER THE BOLTS BETWEEN THE SPLICED PIECES OF U-CHANNEL. THREADED OR UNTHREADED SPACERS MAY BE USED. DO NOT SUBSTITUTE PIPE OR OTHER "ITEMS" FOR THE SPACERS.



STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	84	99

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Added Spacer Dimension, Removed PSST Note	J.A.M.	K.P.
2	10/4/11	Removed Washer On PSST Detail	J.A.M.	K.P.
1	6/1/10	Modified Post Anchor Sleeve Dimension	J.A.M.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

APPROVED TEMPORARY POST SETUPS

TE712

FHWA APPROVAL			10/16/12	APP'D	Kristina Pyle
DESIGNED	B.A.H.	DETAILED	B.A.H.	QUANTITIES	TRACED
DESIGN CK.		DETAIL CK.		QUAN. CK.	TRACE CK.

KDOT Graphics Certified 10-23-2012

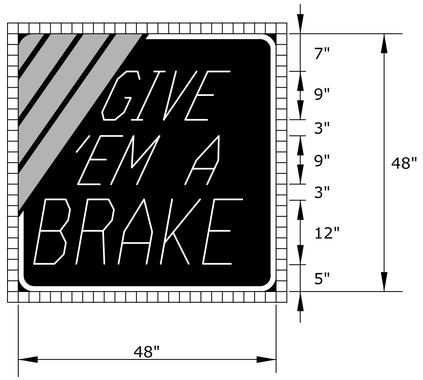
Sh. No. 84

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te712.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	85	99

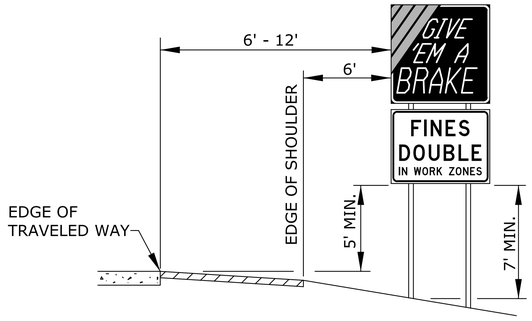


KI-104a

COLORS:
- BACKGROUND - BLACK
- BORDER - WHITE
- LEGEND - WHITE
- STRIPES - ORANGE

BORDER WIDTH - 1"
CORNER RADIUS - 4"
STRIPE WIDTH - 3"

LEGEND:
" GIVE " - DUTCH 801 ROMAN SWC - 25 DEGREE SLANT
" 'EM A " - DUTCH 801 ROMAN SWC - 25 DEGREE SLANT
" BRAKE " - DUTCH 801 ROMAN SWC - 10 DEGREE SLANT



HEIGHT AND LATERAL PLACEMENT

NOTE:
IF THERE IS NO SHOULDER, THE SIGN SHALL BE PLACED 6'- 12' FROM THE EDGE OF TRAVELED WAY AND NOT LESS THAN 6' FROM THE EDGE OF PAVED SHOULDER. SIGNS SHALL NOT OVERLAP EACH OTHER.

NOTES:

THE SIGN BLANK MATERIAL SHALL BE ALUMINUM, WOOD, OR FIBERGLASS REINFORCED PLASTIC.

THE ORANGE PORTION OF THE KI-104a SIGN SHALL BE ASTM TYPE IV SHEETING. THE WHITE PORTION SHALL BE ASTM TYPE III SHEETING.

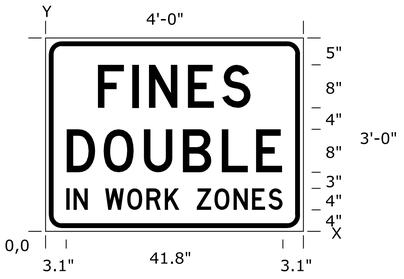
THE KI-105a SIGN FACE SHALL BE COVERED WITH ASTM TYPE III SHEETING.

THE SIGNS ARE TO BE MOUNTED ON CRASHWORTHY SUPPORTS, BRACING, GUY WIRES AND TIE-DOWNS ARE NOT ALLOWED.

TYPICALLY, THERE ARE TWO SETS OF INFORMATIONAL SIGNS INSTALLED PER PROJECT: ONE FOR EACH DIRECTION OF TRAFFIC.

INSTALL SIGNS A MINIMUM OF 250' IN ADVANCE OF THE ROAD WORK AHEAD SIGN. THE ENGINEER MAY DESIGNATE A MORE APPROPRIATE LOCATION IF CONDITIONS DICTATE.

THE INFORMATIONAL SIGNS ARE NOT TO INTERFERE WITH THE TRAFFIC CONTROL SIGNS FOR THE PROJECT.



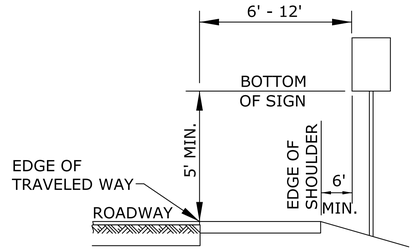
KI-105a

SIGN NUMBER	FINES DOUBLE
WIDTH x HEIGHT	4'-0" x 3'-0"
BORDER WIDTH	0.9"
CORNER RADIUS	3.0"
MOUNTING	GROUND
BACKGROUND	TYPE: REFLECTIVE
	COLOR: WHITE
LEGEND/BORDER	TYPE: NON-REFLECTIVE
	COLOR: BLACK

DIMENSIONS IN INCHES — SPACINGS ARE TO START OF NEXT LETTER

Y FONT	LETTER SPACINGS																					HT LEN
23.0 D		F	I	N	E	S																8.0
	9.7	6.4	3.2	7.3	6.4	5.4	9.7															28.6
11.0 D		D	O	U	B	L	E															8.0
	3.9	6.9	7.5	7.3	7.3	6.4	4.9	3.9														40.3
4.0 D		I	N		W	O	R	K		Z	O	N	E	S								4.0
	3.1	1.6	2.7	3.2	4.3	3.8	3.6	2.8	3.2	3.4	3.8	3.6	3.2	2.7	3.1							41.8

HEIGHT AND LATERAL DIMENSIONS FOR GROUND MOUNTED SIGNS (SIGNS LEFT IN PLACE OVER 3 DAYS)



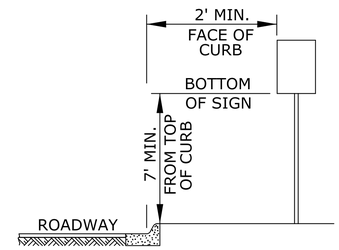
RURAL DISTRICT

RURAL:
1) GROUND-MOUNTED SIGNS SHALL BE MOUNTED AT A MINIMUM HEIGHT OF 5' MEASURED FROM THE BOTTOM OF SIGN TO THE NEAR EDGE OF THE PAVEMENT.

2) LARGE SIGNS HAVING AN AREA EXCEEDING 50 SQUARE FEET INSTALLED ON MULTIPLE BREAKAWAY POSTS SHALL BE MOUNTED A MINIMUM OF 7' ABOVE THE GROUND.

* 3) THE HEIGHT OF THE SECONDARY SIGN MOUNTED BELOW ANOTHER SIGN MAY BE 4' MEASURED FROM THE BOTTOM OF THE SIGN TO THE NEAR EDGE OF THE PAVEMENT. SIGNS SHALL NOT OVERLAP EACH OTHER.

4) SIGN SUPPORTS SHALL BE CRASHWORTHY.



URBAN DISTRICT

URBAN:
1) SIGNS SHALL BE MOUNTED AT A MINIMUM HEIGHT OF 7' MEASURED FROM THE BOTTOM OF SIGN TO THE NEAR EDGE OF THE PAVEMENT.

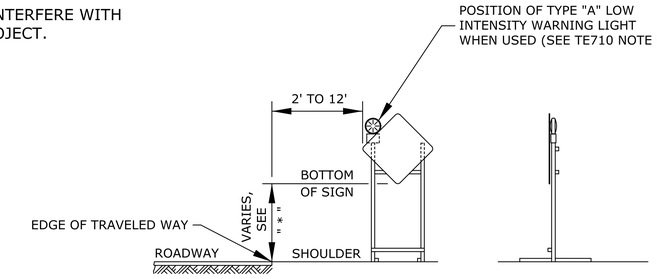
2) NEITHER PORTABLE NOR PERMANENT SIGN SUPPORTS SHOULD BE LOCATED ON SIDEWALKS OR AREAS DESIGNATED FOR PEDESTSRIAN OR BICYCLE TRAFFIC.

3) SIGNS MOUNTED LOWER THAN 7' SHOULD NOT PROJECT MORE THAN 4" INTO PEDESTRIAN FACILITIES.

* 4) THE HEIGHT FROM OF THE SECONDARY SIGN MOUNTED BELOW ANOTHER SIGN MAY BE 6' MEASURED FROM THE BOTTOM OF SIGN TO THE NEAR EDGE OF THE PAVEMENT. SIGNS SHALL NOT OVERLAP EACH OTHER.

5) LARGE SIGNS HAVING AN AREA EXCEEDING 50 SQUARE FEET INSTALLED ON MULTIPLE BREAKAWAY POSTS SHALL BE MOUNTED A MINIMUM OF 7' ABOVE THE GROUND.

6) SIGN SUPPORTS SHALL BE CRASHWORTHY.



(SEE NOTE 4 ON TE710 FOR "ROLL-UP" SIGNS OPTION)

HEIGHT AND LATERAL DIMENSIONS FOR SIGNS MOUNTED ON SKIDS OR OTHER SUPPORTS ON PAVEMENT

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Modified Note 3 & 4 On Ground Mounted Signs	J.A.M.	K.P.
2	10/4/11	Modified Note On Skid Mounted Detail	J.A.M.	K.P.
1	6/1/10	Changed Type A Warning Light Note	J.A.M.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

GROUND AND SKID MOUNTED DETAIL DETAILS FOR "GIVE 'EM A BRAKE" AND "FINES DOUBLE" SIGNS

TE714

FHWA APPROVAL		10/16/12	APP'D	Kristina Pyle	
DESIGNED	B.A.H.	DETAILED	B.A.H.	QUANTITIES	TRACED
DESIGN CK.		DETAIL CK.		QUAN. CK.	TRACE CK.

KDOT Graphics Certified 10-23-2012

Sh. No. 85

KDOT Graphics Certified

Plotted : 13-SEP-2013 10:54

Drawn By : cluedke
File : te714.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	86	99

SUMMARY OF TRAFFIC CONTROL DEVICES (EACH)

IBS REPLACEMENT MODULES	
REPLACEMENT MODULE SIZES	QUANTITY
REPLACEMENT MODULES (F2OO)	
REPLACEMENT MODULES (F4OO)	1
REPLACEMENT MODULES (F7OO)	1
REPLACEMENT MODULES (F14OO)	1
REPLACEMENT MODULES (F21O0)	2

WORK ZONE SIGN (SPECIAL)		
SIGN NO.	16.25 SQ.FT. & LESS	16.26 SQ.FT. & OVER
SP-1		1
D3	22	

PEDESTRIAN DEVICES		
ITEM	QUANTITY	UNIT
ADA COMPLIANT PEDESTRIAN BARRICADE	535	LIN. FT.
AUDIBLE INFORMATION DEVICE	6	EACH

NOTE: ITEMS ABOVE ARE SUBSIDIARY TO OTHER TRAFFIC CONTROL ITEMS.

SUMMARY OF TRAFFIC CONTROL DEVICES (EACH PER DAY)

✳ QUANTITY MOST USED ON THE PROJECT AT ANY ONE TIME

WORK ZONE SIGNS ✳			
SIGN NO.	SIZE - SQ.FT.		
	0-9.25	9.26-16.25	16.26 & OVER
W20-7		2	
KG20-2	4		
KR3-8h	2		
M4-8a	6		
M4-9aL	8		
M4-9aR	9		
M4-9aS	5		
M4-9bL	3		
M4-9bR	3		
M4-9L	3		
M4-9R	6		
M4-9S	6		
M4-10L	1		
M4-10R	1		
R3-1	2		
R3-2	2		
R4-7	4		
R9-9	4		
R9-10	1		
R9-10L	1		
R9-10R	2		
R9-11L	1		
R11-2		3	
R11-4		1	
W20-1		4	
W20-2		2	
W20-3		5	

AUDIBLE INFORMATION DEVICE	BARRICADES✳		CHANNELIZING DEVICES✳	
	(PEDESTRIAN) ADA COMPLIANT	TYPE III (4' TO 12')	FIXED	PORTABLE
6	90	24		36

LIGHTED DEVICES ✳	
WORK ZONE WARNING LIGHT (TYPE "A" LOW INTENSITY)	10
WORK ZONE WARNING LIGHT (RED TYPE "B" HIGH INTENSITY)	
ARROW DISPLAY	
PORTABLE CHANGEABLE MESSAGE SIGN	

RECAPITULATION OF QUANTITIES		
ITEM	QUANTITY	UNIT
WORK ZONE SIGNS (O TO 9.25 SQ.FT.)		EACH PER DAY
WORK ZONE SIGNS (9.26 TO 16.25 SQ.FT.)		EACH PER DAY
WORK ZONE SIGNS (16.26 SQ.FT. & OVER)		EACH PER DAY
WORK ZONE BARRICADES (TYPE III - 4' TO 12')		EACH PER DAY
CHANNELIZER (FIXED)		EACH PER DAY
CHANNELIZER (PORTABLE)		EACH PER DAY
WORK ZONE WARNING LIGHT (TYPE "A" LOW INTENSITY)		EACH PER DAY
WORK ZONE WARNING LIGHT (RED TYPE "B" HIGH INTENSITY)		EACH PER DAY
ARROW DISPLAY		EACH PER DAY
PORTABLE CHANGEABLE MESSAGE SIGN		EACH PER DAY
PAVEMENT MARKING (TEMPORARY)		
4" SOLID (TYPE I TAPE OR PAINT)	14.5	STA./LINE
4" SOLID (TYPE II TAPE OR PAINT)		STA./LINE
4" BROKEN (8.0') (TYPE I TAPE OR PAINT)		STA./LINE
4" BROKEN (8.0') (TYPE II TAPE OR PAINT)		STA./LINE
4" BROKEN (8.0') (FLEXIBLE RAISED PAVEMENT MARKER)		STA./LINE
4" BROKEN (3.0') (TYPE I TAPE OR PAINT)		STA./LINE
4" BROKEN (3.0') (TYPE II TAPE PAINT)		STA./LINE
4" BROKEN (3.0') (FLEXIBLE RAISED PAVEMENT MARKER)		STA./LINE
4" DOTTED EXTENSION (TYPE I TAPE OR PAINT)		STA./LINE
4" DOTTED EXTENSION (TYPE II TAPE OR PAINT)		STA./LINE
SOLID (LINE MASKING TAPE)		STA./LINE
BROKEN (LINE MASKING TAPE)		STA./LINE
PAVEMENT MARKING REMOVAL	1,300	LIN. FT.
CONCRETE SAFETY BARRIER (TYPE F3)(TEMPORARY)	125	LIN. FT.
CONCRETE SAFETY BARRIER (TYPE F3)(TEMP.-INSTALL ONLY)		LIN. FT.
CONCRETE SAFETY BARRIER (TYPE F3)(TEMP.-RELOCATE)	88	LIN. FT.
INERTIAL BARRIER SYSTEM	1	EACH
REPLACEMENT MODULES	5	EACH
WORK ZONE SIGN (SPECIAL) (16.25 SQ. FT. & LESS)		EACH
WORK ZONE SIGN (SPECIAL) (16.26 SQ. FT. & MORE)	1	EACH
TEMPORARY RAISED PAVEMENT MARKER (TYPE I)		EACH
TEMPORARY RAISED PAVEMENT MARKER (TYPE II)		EACH
TRAFFIC SIGNAL INSTALLATION (TEMPORARY)		LUMP SUM
TRAFFIC CONTROL (INITIAL SET UP)	LUMP SUM	LUMP SUM
TRAFFIC CONTROL		LUMP SUM
FLAGGER (SET PRICE)	1	HOUR
TWORKS SIGN ASSEMBLY	2	EACH

NO.	DATE	REVISIONS	BY	APP'D
3	10/16/12	Modified Pavement Marking Requirements	J.A.M.	K.P.
2	10/4/11	Added IBS Quantities, Added TWorks Quanity	J.A.M.	K.P.
1	8/8/07	2007 Spec Revision	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

SUMMARY OF DEVICES AND RECAPITULATION OF QUANTITIES

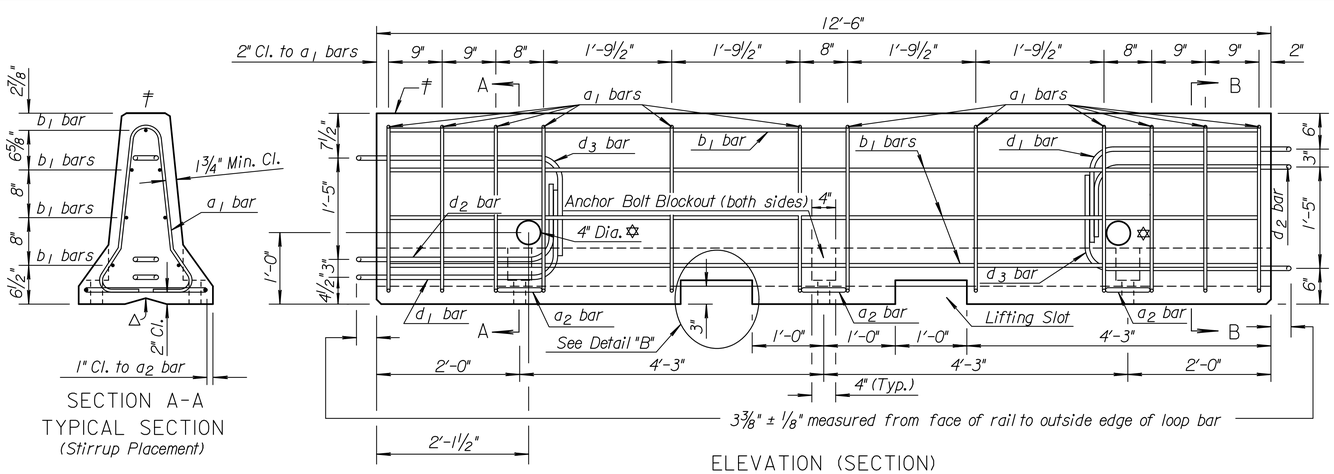
~~TE795~~

FHWA APPROVAL		10/16/12	APP'D
DESIGNED	DETAILED	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Plotted : 13-SEP-2013 10:54

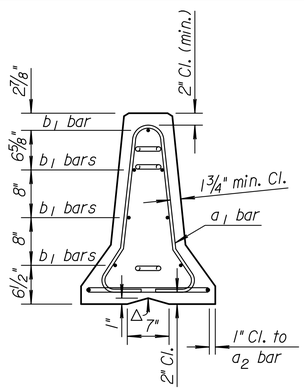
Drawn By : cluedke
File : te795.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	87	99



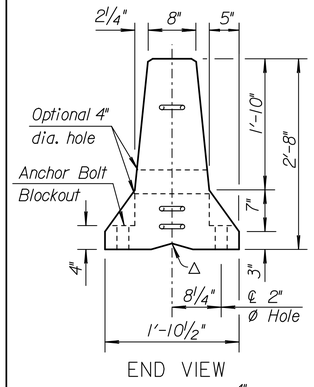
SECTION A-A
TYPICAL SECTION
(Stirrup Placement)

ELEVATION (SECTION)

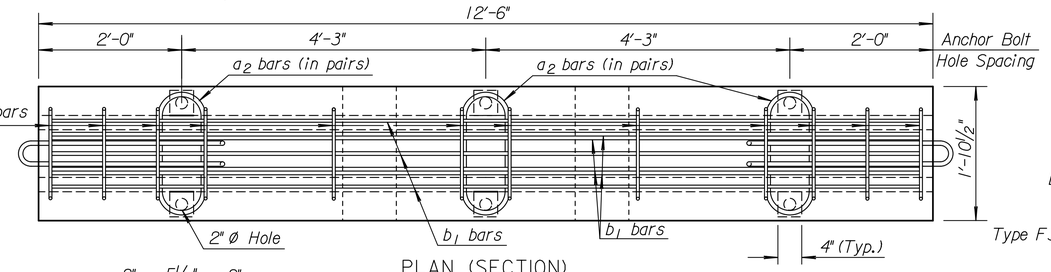


SECTION B-B
TYPICAL SECTION
(Stirrup Placement)

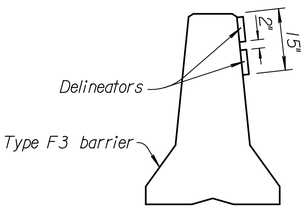
ǂ Marked End
✡ 4" diameter - 11 gauge steel round mechanical tubing sleeve. These holes are optional.
△ V Notch is optional



END VIEW



PLAN (SECTION)



DELINEATOR DETAILS

GENERAL NOTES:

MATERIAL: Use ASTM A615, Grade 60 reinforcing bars, except for the loop bars (d_1, d_2 and d_3).

The loop bars (d_1, d_2 and d_3) shall be ¾" smooth steel bars with a minimum yield of 60 ksi, a tensile strength of not less than 1.25 times the yield strength but a minimum of 80 ksi, a minimum 14% elongation in 8 inches, and passing a 180 degree bend test using a 3.5 D pin bend diameter. The loops shall be installed within ⅛" of the plan dimensions.

Use air-entrained concrete with f'c = 5,000 p.s.i.

SECTION: The section furnished must generally comply with dimensions shown. Requests for minor variations in section geometry and attachments may be submitted to the Engineer for approval.

LIFTING SLOTS: Lifting slots shall be constructed where specified on the plans to facilitate the drainage of water after installation on the roadway.

TEMPORARY CONCRETE SAFETY BARRIER: Furnishing and placing of all materials when required and all labor and equipment required to position the temporary barrier shall be included in the Contract unit price bid for "Concrete Safety Barrier (Type F3)(Temporary)". Any relocation of the barrier required for the project shall be paid in accordance with the Special Provisions under the bid item "Concrete Safety Barrier (Type F3) (Temporary-Relocate)". Unless otherwise noted on the Plans, the Temporary Concrete Safety Barrier shall become the property of the Contractor and shall be removed from the site upon acceptance of the completed project. Approximate weight of one unit equals 2.7 tons.

SURFACE PREPARATION: Barrier shall be placed on a paved surface. All loose dirt and sand shall be removed from the roadway surface just prior to placement of the barrier.

MARKING: The left end (ǂ) of each barrier shall be permanently marked by stamping or forming into the barrier the following information:

- *Type F3*
- *Manufacturer code (as specified by KDOT Bureau of Const. & Maint.)*
- *Date manufactured (month and year)*

DELINEATION: Delineators shall be spaced on 50' centers, except through curves having 1900' or greater curvature where they shall be spaced on 25' centers.

The delineation shall be mounted on the side of the Temporary Concrete Safety Barrier with two delineators at each location. Each delineator shall have a minimum height-to-width ratio of 1.75, and a minimum reflective surface area of 7 sq. in.. The delineators shall be affixed to the Temporary Concrete Safety Barrier as recommended by the manufacturer.

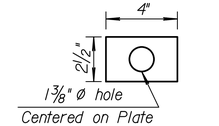
Delineators shall be attached to bridge rail or other structures in construction zones when roadway is narrowed and traffic is adjacent to the structure. The method and location of placement shall be similar to permanent barrier delineation.

When traffic flow is in one direction, the delineators shall be yellow when used on the left, white when used on the right. When traffic flow is in both directions delineators shall be placed back-to-back, and shall correspond to the color of the edge line.

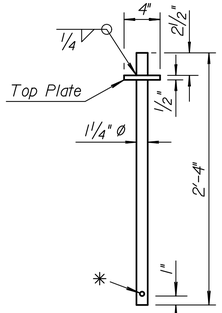
The work and materials required for the installation of delineators as mentioned shall be subsidiary to the bid item "Concrete Safety Barrier (Type F3) (Temporary).

Note: If necessary, include Standard Drawing RD622A for Taper Section, Standard drawing RD622B for anchor and tie down details, Standard Drawing RD622C for Bridges with thermal expansion of 1½" or greater and Standard Drawing RD622D for Barrier Layouts.

Note: The Contractor shall be responsible for maintaining a clear area, shown as dimension "A" on Standard Drawing RD622B. The clear area is located behind the Temporary Concrete Safety Barrier and shall be kept free of any equipment, material stockpiles or other obstacles.

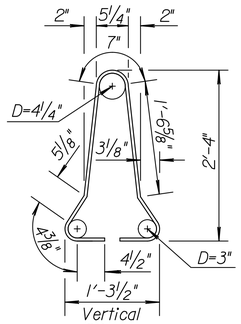


(½" Top Plate Detail)

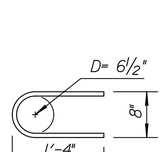


DETAIL A
CONNECTION PIN
(A36 Steel) 10.9 lbs. each

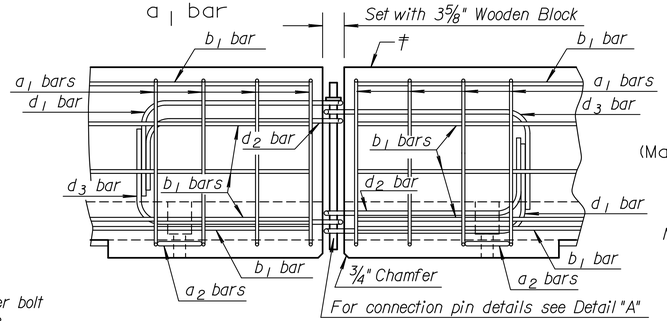
✳ ⅝" Ø hole for retainer bolt. The retainer bolt & nut are installed at Contractor's option.
Note: Retainer bolt & nut required with Tie Down Strap.



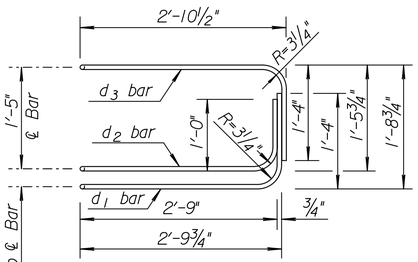
a1 bar



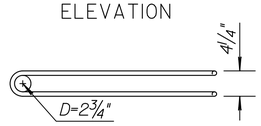
a2 bar



DETAILS OF BARRIER CONNECTION



ELEVATION



PLAN

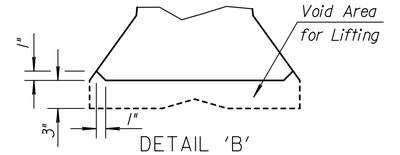
LOOP BAR ASSEMBLY
(Marked end shown, invert for other end)
(Material as stated in General Notes)

NOTE: At no time shall the barriers be lifted, moved, etc. by use of the loop bars: d_1, d_2 or d_3.

Per 12'-6" Barrier Section

REINFORCING A615 Gr. 60					
Bar	*Bar Size*	*Shape*	*No. of Bars*	*Length Ft.*	*Weight Lbs.*
a_1	#4		12	6'-0"	48.1
a_2	#6		6	2'-11"	26.3
b_1	#5		7	12'-2"	88.8
LOOP ASSEMBLY					
d_1	#6		2	8'-5"	25.3
d_2	#6		2	7'-7"	22.8
d_3	#6		2	8'-6"	25.5

Concrete Quantity = 1.3 C.Y.
(Dimensions are out to out of bars unless otherwise noted.)



DETAIL 'B'
LIFTING SLOT DETAIL
(1" Chamfer to prevent spalling)

NO.	DATE	REVISIONS	BY	APP'D
3	06-03-12	Revised General Note, Clear Area	S.W.K.	J.O.B.
2	02-06-07	Revised additional sheets note	S.W.K.	J.O.B.
1	01-10-07	Rev. layout & notes, add. Delineation	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY CONCRETE SAFETY BARRIER TYPE F3

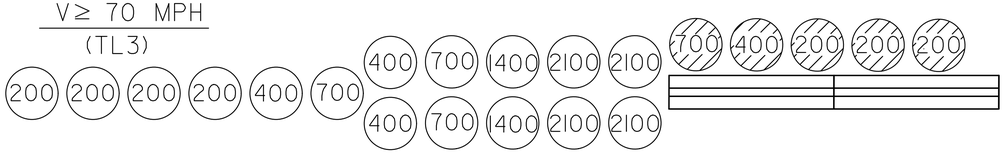
RD622

FHWA APPROVAL	08-01-12	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. Kling

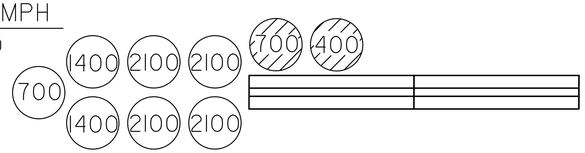
Drawn By: cluedke Plotted: 9/13/2013
File: G:\12664\MicroHighways\Client Standards\rd622.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	88	99

V≥ 70 MPH
(TL3)



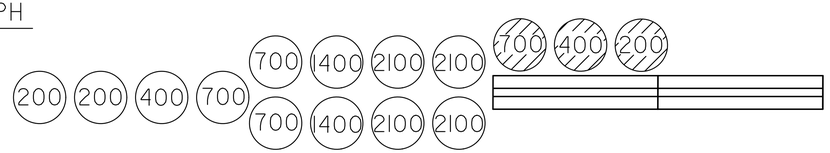
V= 35 MPH
(TL2)



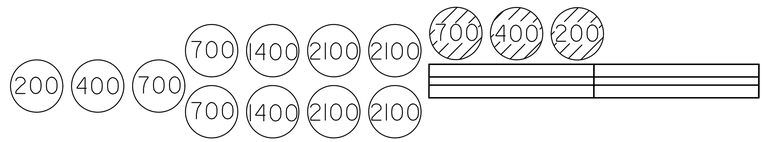
V= 65 MPH
(TL3)



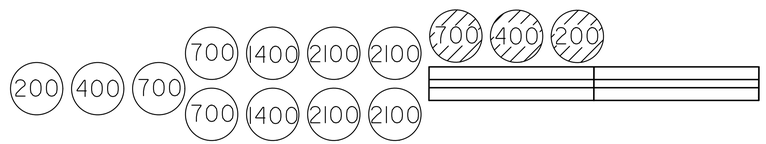
V= 60 MPH
(TL3)



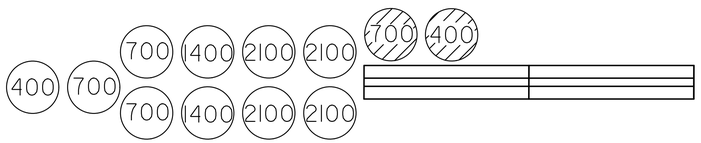
V= 55 MPH
(TL3)



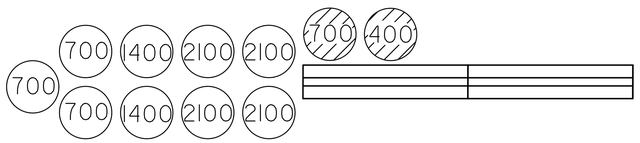
V= 50 MPH
(TL3)



V= 45 MPH
(TL2)



V= 40 MPH
(TL2)



INERTIAL BARRIER SYSTEM			
Station	Side	Design Speed	Comments
54+33.77	Lt.	35 MPH	Sequence 1
54+36.65	Lt.	35 MPH	Sequence 2

GENERAL NOTE

This drawing details general configurations for Inertial Barrier Systems. Some project specific conditions may require variations which are designed to meet prevailing criteria.

Use Inertial Barrier System consisting of the units as shown for the specified design speed, all hardware and attachments.

Install Inertial Barrier System on a flat, stable base with cross-slope no steeper than 10:1. See Manufacturer's recommendations for module materials and method of installation.

See standard specifications for mixture to fill modules requirements.

Provide a 6" spacing between modules and one foot between the end of concrete barrier or other rigid object.

When installed as part of project traffic control, the bid item "Inertial Barrier" includes the original installation and required relocations.

Keep available replacement modules to replace any size module used on site, Engineer's direction.

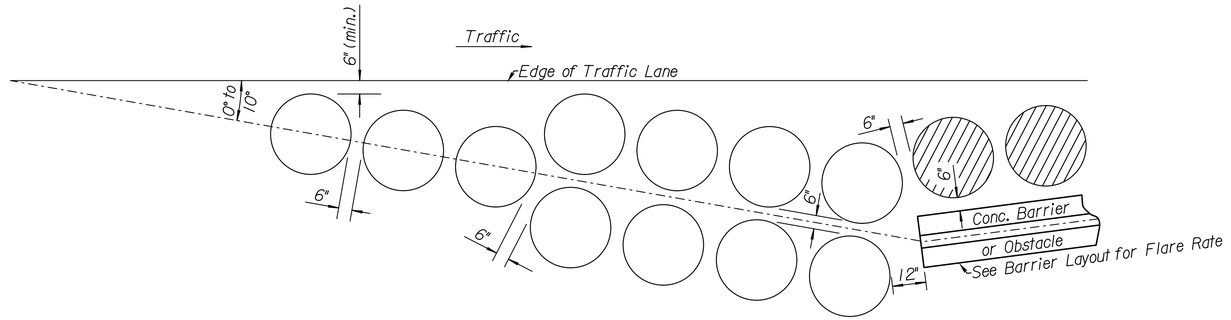
Inertial Barrier System modules damaged by the Contractor during relocation of Inertial Barrier System are replaced at the Contractor's expense.

Module weights shown are in pounds.

Install 270 square inches of Type II High Performance (vertical, rectangular or diamond shape) reflective sheeting on first module of Inertial Barrier System facing traffic.

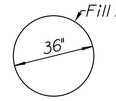
Where sufficient space is available the Inertial Barrier System may be aligned at an angle, not to exceed 10°, in the direction of approach traffic.

No portion of the system shall encroach into the approach traffic lane.



TYPICAL PLAN of INERTIAL BARRIER

When two-way traffic is adjacent to only one side of Concrete Barrier or Obstacle, these additional modules will be placed on the Traffic Side of Concrete Barrier or Obstacle. Traffic adjacent to both sides of the Concrete Barrier or Obstacle require an additional set of modules each side if approach traffic is exposed to the back portion of the Inertial Barrier. These additional modules are not required along the sides of Concrete Barrier or Obstacle when it's location is outside the Clear Zone or one-way directional traffic.



PLAN
Replacement Module

NO.	DATE	REVISIONS	BY	APP'D
6	2-3-12	Revised General Note	S.W.K.	J.O.B.
5	6-27-11	Rdvised notes & Typical Plan detail	S.W.K.	J.O.B.
4	9-10-09	Impact Attenuator to Inertial Barrier	S.W.K.	J.O.B.
3	12-10-08	Add. modules, replacement, std. title	S.W.K.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

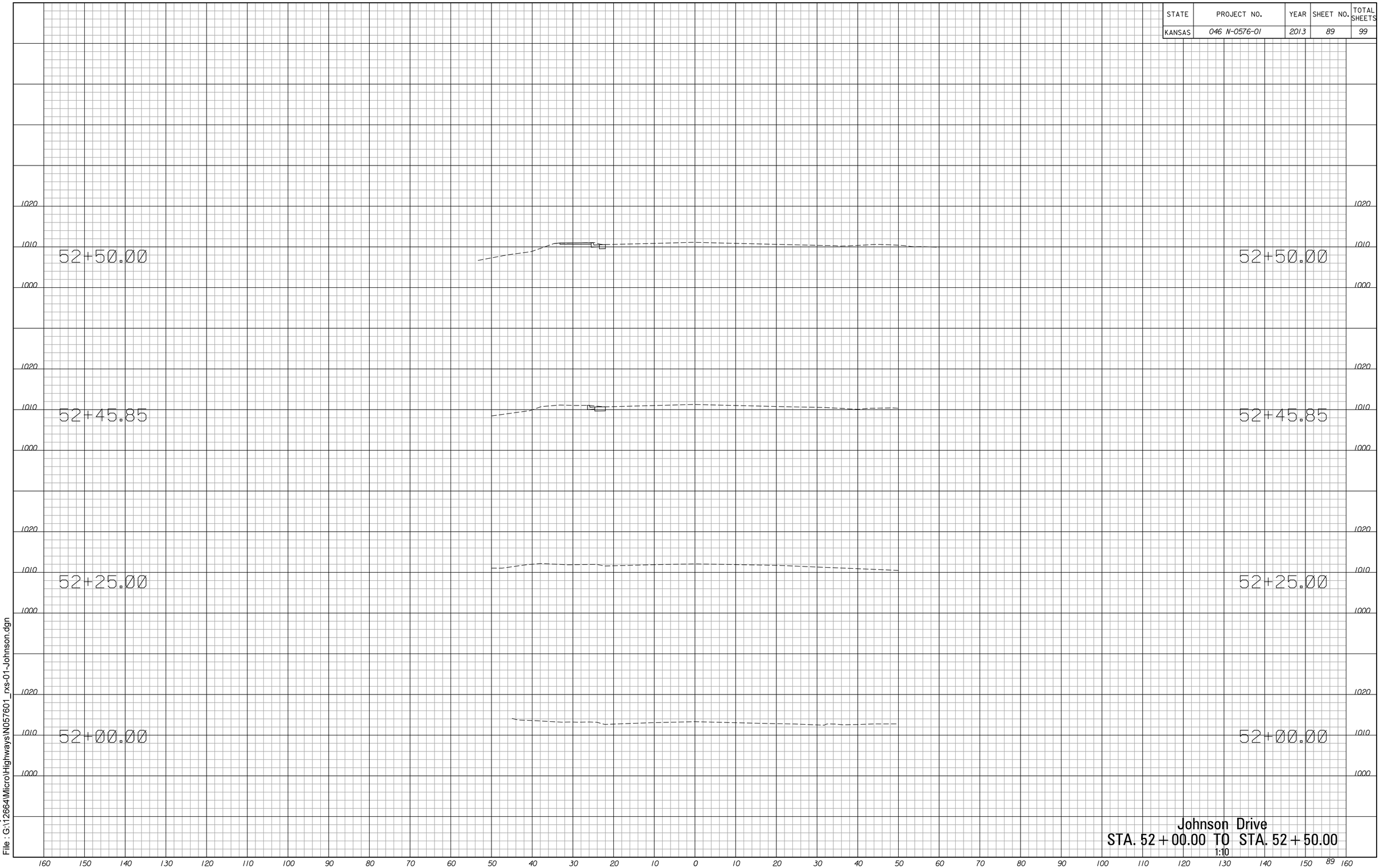
INERTIAL BARRIER (TL2 or TL3)

RD620

FHWA APPROVAL	4-25-12	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. King

Drawn By: cluedke
File: G:\12664\MicroHighways\Client Standards\rd620.dgn
Plotted: 9/13/2013

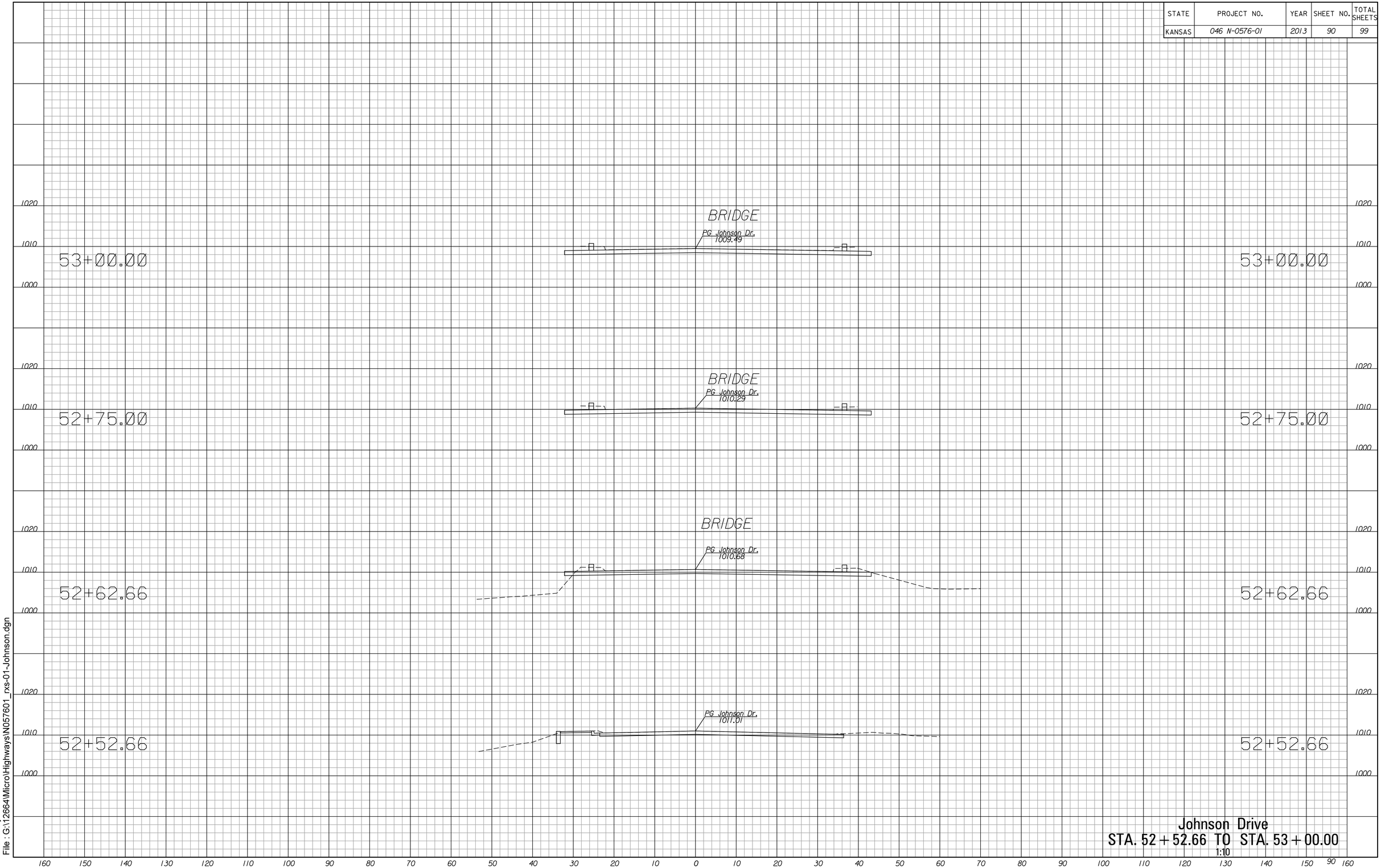
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	89	99



Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-01-Johnson.dgn

Johnson Drive
STA. 52+00.00 TO STA. 52+50.00
1:10

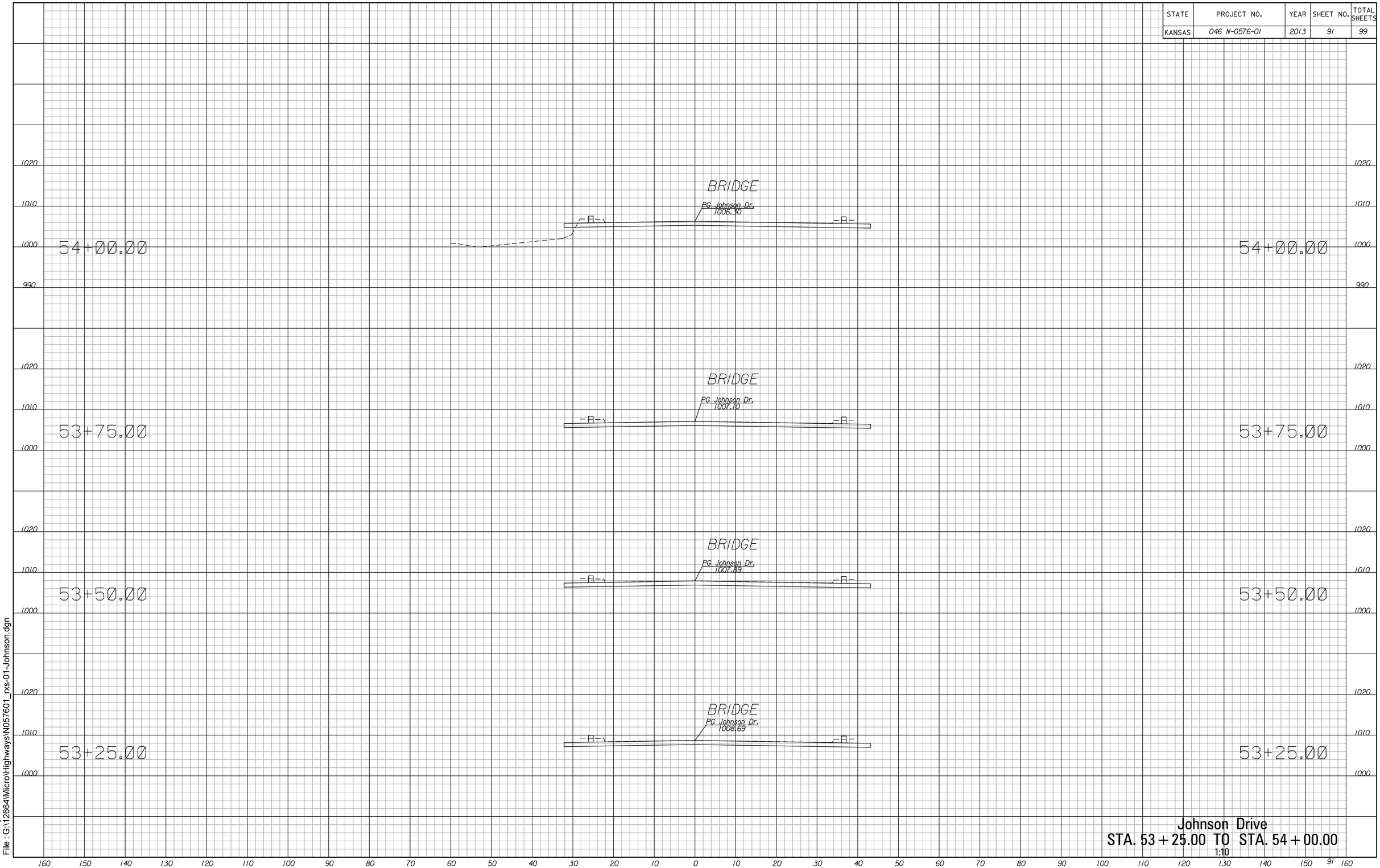
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	90	99



Johnson Drive
STA. 52+52.66 TO STA. 53+00.00
1:10

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-01-Johnson.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	91	99



Johnson Drive
STA. 53 + 25.00 TO STA. 54 + 00.00
1:10

Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-01-Johnson.dgn
Plotted : 9/13/2013

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	92	99

54+25.00

54+25.00

P6 Johnson Dr.
1005.46

54+23.63

54+23.63

BRIDGE

P6 Johnson Dr.
1005.86

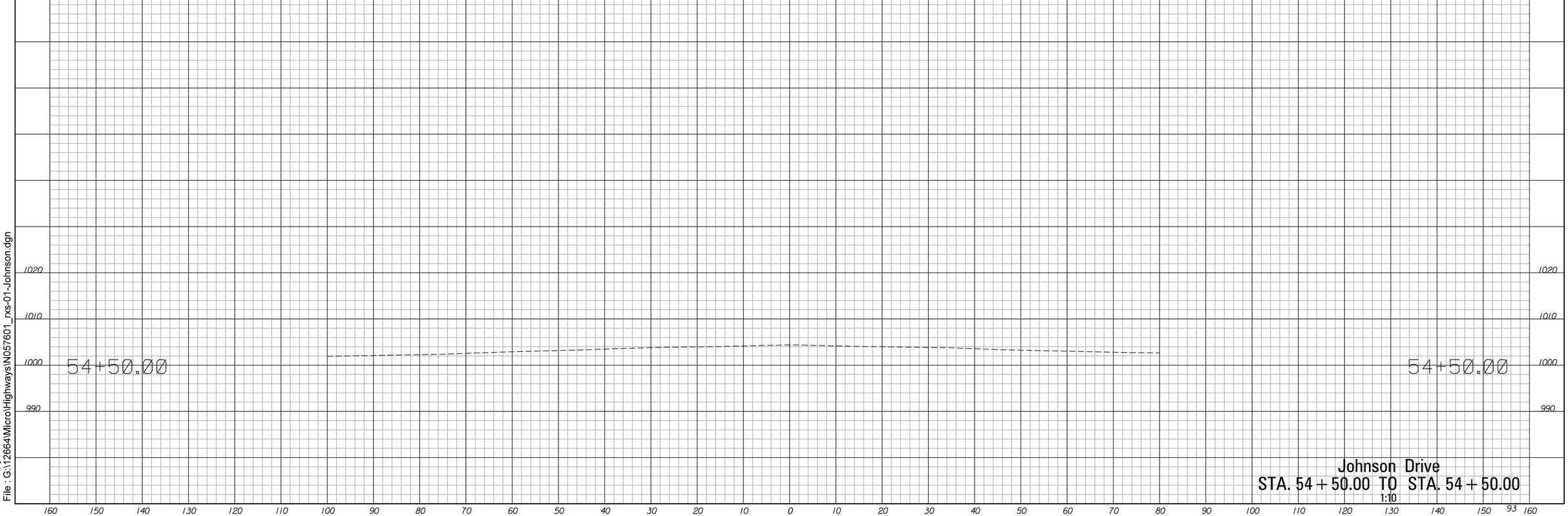
54+13.63

54+13.63

Johnson Drive
STA. 54+13.63 TO STA. 54+25.00
1:10

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-01-Johnson.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	93	99



Johnson Drive
STA. 54 + 50.00 TO STA. 54 + 50.00
1:10

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-01-Johnson.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	94	99

99+40.00

99+25.00

99+00.00

98+80.00

Merriam Drive
STA. 98 + 80.00 TO STA. 99 + 40.00

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\MicroHighways\N057601_rxs-03-Merriam_Drive.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	95	99

99+60.00



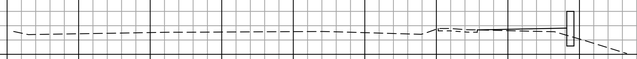
99+60.00

99+55.00



99+55.00

99+50.00



99+50.00

99+43.52

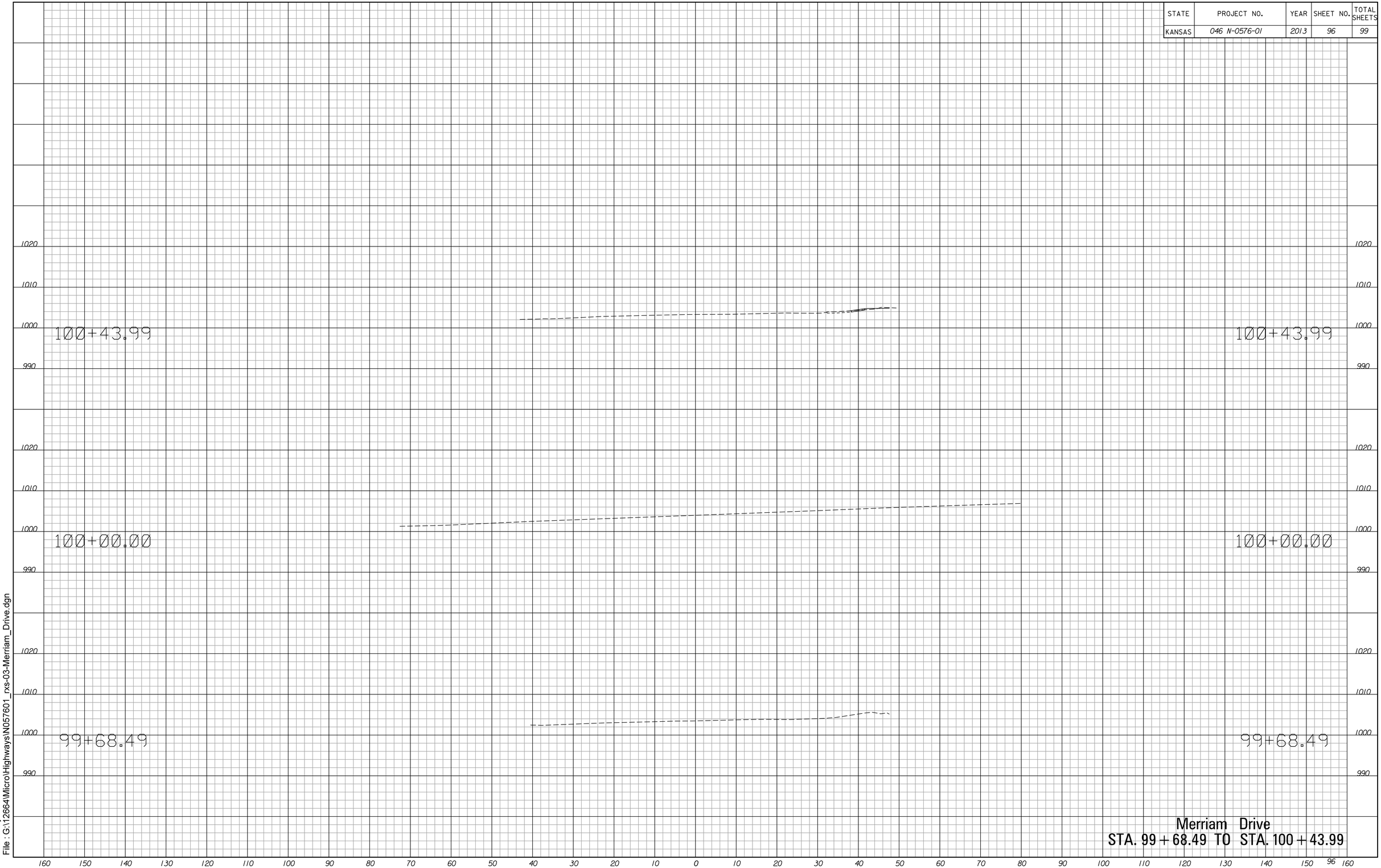


99+43.52

Merriam Drive
STA. 99+43.52 TO STA. 99+60.00

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\IN057601_rxs-03-Merriam_Drive.dgn

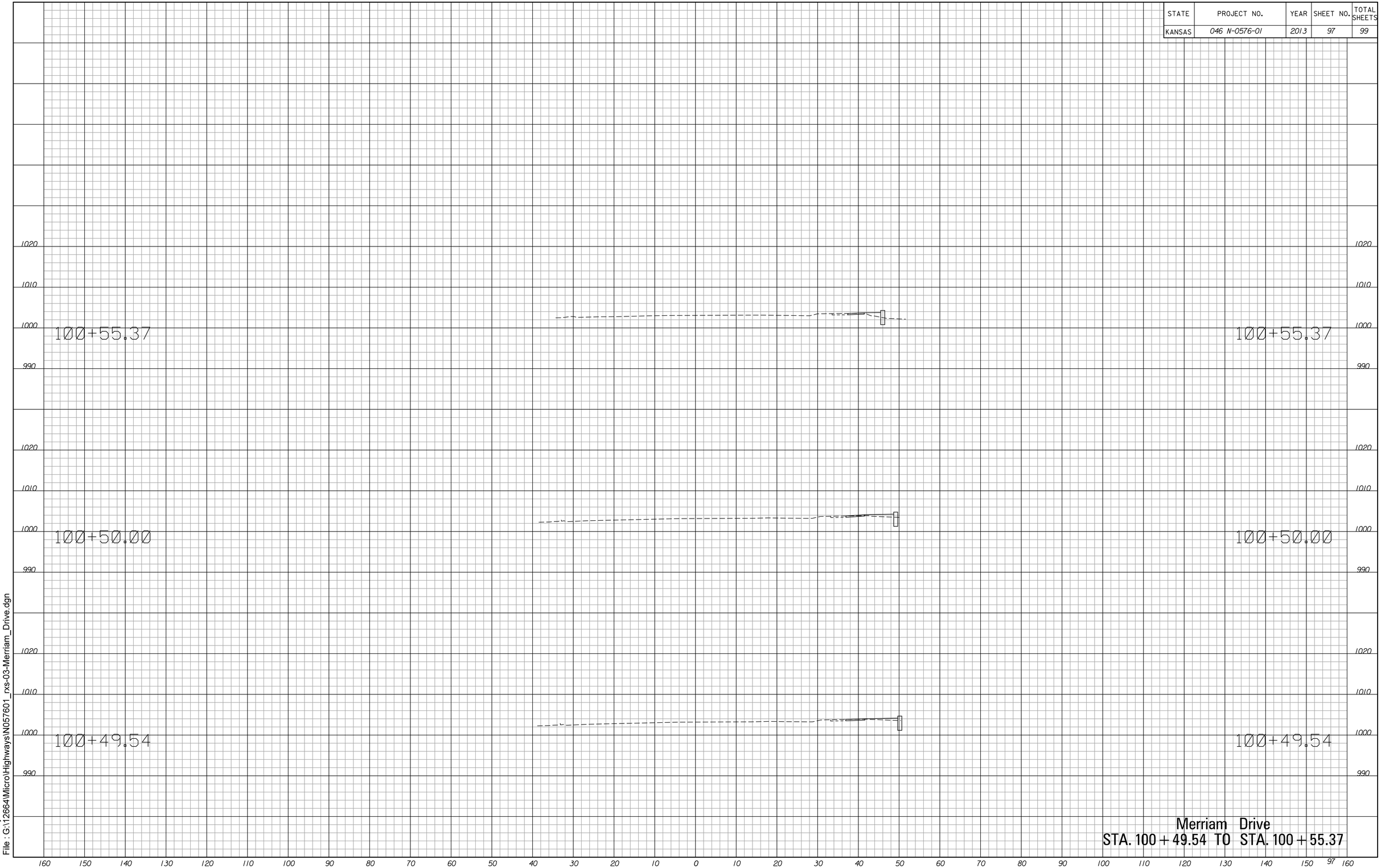
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	96	99



Merriam Drive
STA. 99+68.49 TO STA. 100+43.99

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N0576O1_rxs-03-Merriam_Drive.dgn

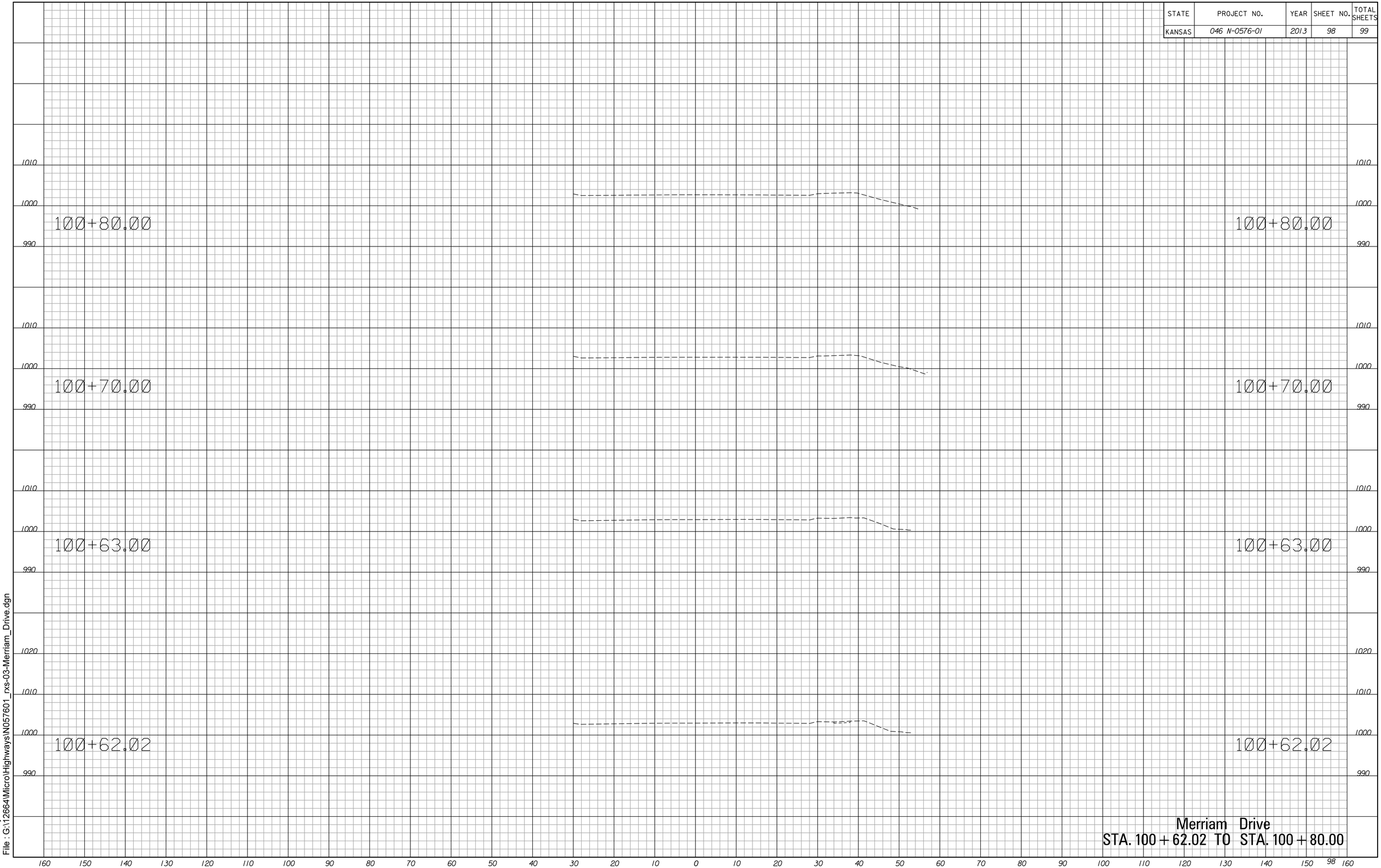
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	97	99



Merriam Drive
STA. 100 + 49.54 TO STA. 100 + 55.37

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-03-Merriam_Drive.dgn

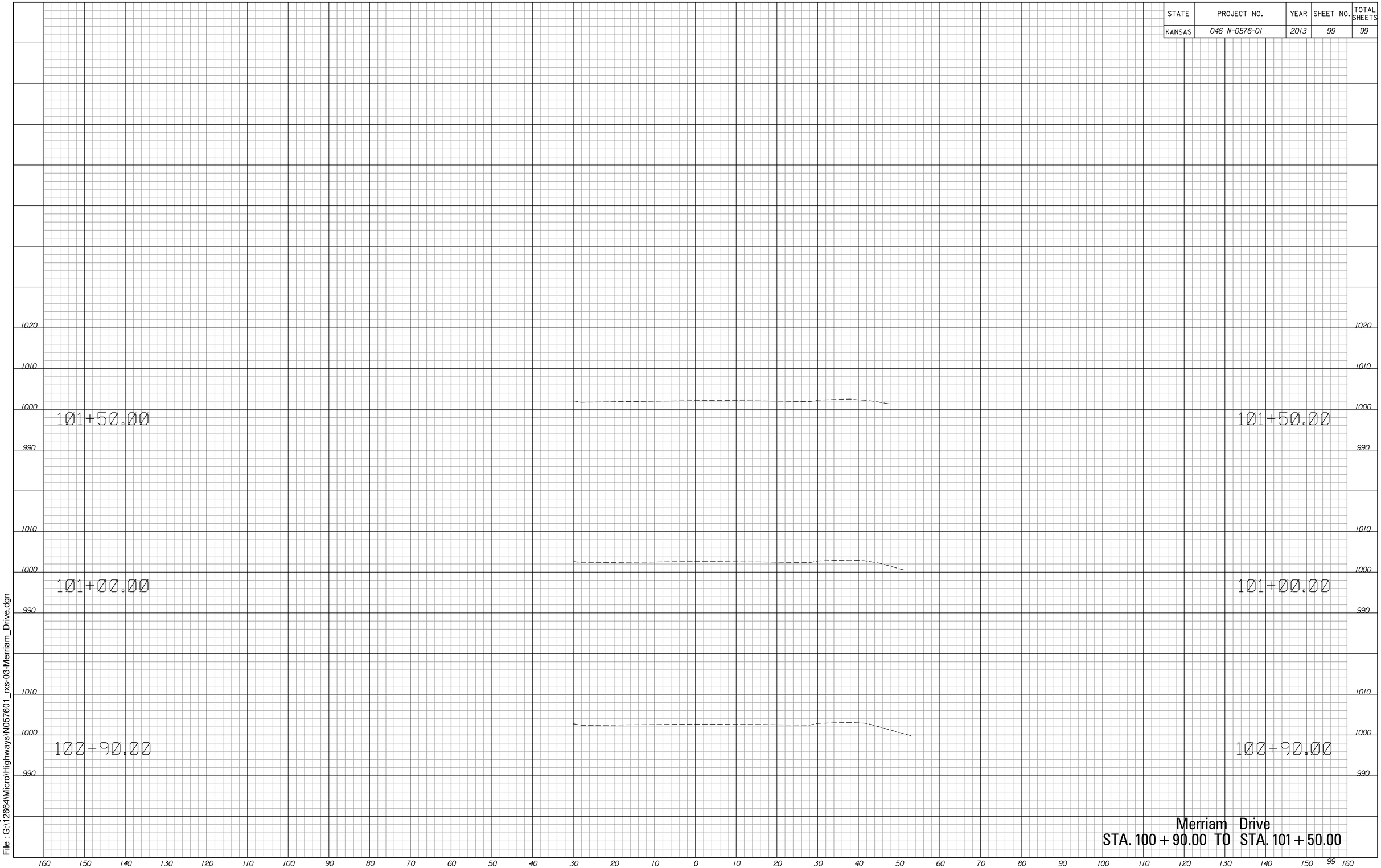
STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	98	99



Merriam Drive
STA. 100+62.02 TO STA. 100+80.00

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\N057601_rxs-03-Merriam_Drive.dgn

STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
KANSAS	046 N-0576-01	2013	99	99



Merriam Drive
STA. 100 + 90.00 TO STA. 101 + 50.00

Plotted : 9/13/2013
Drawn By : cluedke
File : G:\12664\Micro\Highways\IN057601_rxs-03-Merriam_Drive.dgn

INDEX OF SHEETS

SHEET NO.	DESCRIPTION
1	TITLE SHEET
2	GRADING AND LAYOUT @ JOHNSON DRIVE
3	SECTIONS @ JOHNSON DRIVE
4	PROFILE @ JOHNSON DRIVE
5	RETAINING WALLS PLAN AND ELEVATIONS
6	RETAINING WALL SECTIONS "A" AND "C"
7	RETAINING WALL SECTION "B"
8	GRADING AND LAYOUT @ MERRIAM MARKET
9	GENERAL NOTES AND STANDARD DETAILS
10	PEDESTRAIN BRIDGE PLAN AND ELEVATION
11	PEDESTRIAN BRIDGE DETAILS

GENERAL NOTES

1. Information and references to existing conditions shown on these plans are the best information and references available to Shafer, Kline & Warren, Inc., Consulting Engineers, at the time of plan preparation. The plans reflect information provided by field surveys, plats and plans furnished by others who limit their guarantees and responsibilities.

2. By use of these plans the Contractor agrees that he shall be solely responsible for the safety of the construction workers and of the public.

3. The Technical Specifications and Design Criteria for Public Improvements and all amendments thereto for Johnson County Park and Recreation, shall be the specifications for these plans as though fully set forth herein.

4. All existing utilities indicated on the drawings are according to the best information available to the Engineer; however, all utilities actually existing may not be shown. Utilities damaged through the negligence of the contractor to obtain the location of same shall be repaired or replaced.

5. Johnson County Park and Recreation plan review is only for general conformance. Johnson County Park and Recreation is not responsible for the accuracy and adequacy of the design, or dimensions and elevations which shall be confirmed and correlated at the job site. Johnson County Park and Recreation through approval of these documents assumes no responsibility other than as stated above for the completeness and/or accuracy of this document.

6. The Contractor shall thoroughly review and become familiar with the specifications and special conditions of the contract documents, prior to beginning construction on this project.

7. The Contractor shall have one (1) signed copy of the plans (approved by Johnson County Park and Recreation) and one (1) copy of the Specifications at the job site at all times.

8. Construction of the improvements shown on or implied by this set of drawings shall not be initiated or any part thereof undertaken until Johnson County Park and Recreation is notified of such intent, and all required and properly executed bonds and inspection fees are received and approved.

9. The Contractor shall not be allowed to work Sundays or Holidays. The work hours for Monday through Saturday shall be from 8:00 AM TO 6:00 PM.

10. The Contractor will furnish flagmen and other personnel necessary to provide required traffic control which shall include adequate signs, barricades, warning lights, and all other equipment in accordance with the current manual on uniform traffic control devices and procedures.

11. All waterline mains, sanitary sewer mains and stormwater drainage crossings and other utilities under proposed pavements shall be in place or a casing pipe provided for future installation prior to the placement of the curb and gutter.

12. Siltation and erosion control measures shall be in place prior to the disturbance of the ground.

13. The Contractor is responsible for providing berms, silt fences, straw bales, or other means to prevent erosion from reaching Turkey Creek. In the event the prevention measures are not effective, the contractor shall remove any debris and erosion and restore the stream bed to original or better condition.

14. By use of these plans, the Contractor shall be solely responsible for all areas that they or their subcontractors disturb, both within and beyond the project limits. These areas shall be restored, re-seeded, or re-sodded to a state that is equal to or better than the prior condition. The Contractor shall bear the related expenses, unless denoted otherwise within the contract documents.

15. All trees, shrubs, bushes, and brush, within the grading limits shall be removed by the Contractor. Large trees, within the construction easements that are not shown on the plans, shall be brought to the attention of the Engineer prior to disturbance. Exceptionally good trees shall be spared by adjusting grading during construction, as directed by the Engineer. All trees, within the construction limits, to be saved shall have their trunks physically protected, prior to construction operations, by methods approved by the Engineer.

16. All concrete for the project shall comply with the standards and specifications of the Johnson County Concrete Board. Structural concrete shall be 5000 PSI. Non structural concrete shall be 4000 PSI.

UTILITY NOTES

1. Visual indications of utilities are as shown. Underground locations shown, as furnished by the respective utility companies, are approximate and shall be verified in the field at the time of construction. For actual field locations of underground utilities, call 1-800-344-7233, Kansas or 1-800-344-7483, Missouri.

2. The Contractor shall be responsible for contacting all utility companies for field location of all underground utility lines prior to any excavation and for the coordination and scheduling with utility owners of all work required to resolve conflicts with installations, constructions, excavations, removals, placements, relocation and other miscellaneous work shown upon these plans or required witnin contract documents.

3. All manholes, catch basins, utility valves and meter pits shall be adjusted or rebuilt to grade as required.

4. The name and telephone number of utility companies, even if remotely involved with this project are as follows:

POWER:	KPL Electric 913-768-3293
GAS:	KPL Gas Service Co. 913-599-8985 Williams Pipe Line Co. 913-677-2166
WATER:	Water District No. 1 of Johnson County 913-895-1806
TELEPHONE:	Southwestern Bell Telephone Co. 913-676-1862
SEWER:	Johnson County Unified Wastewater District 913-681-3200 Time Warner
CABLE TV:	913-451-5858 Everest Communications 816-471-6141
STREET LIGHTS:	City of Merriam, KS

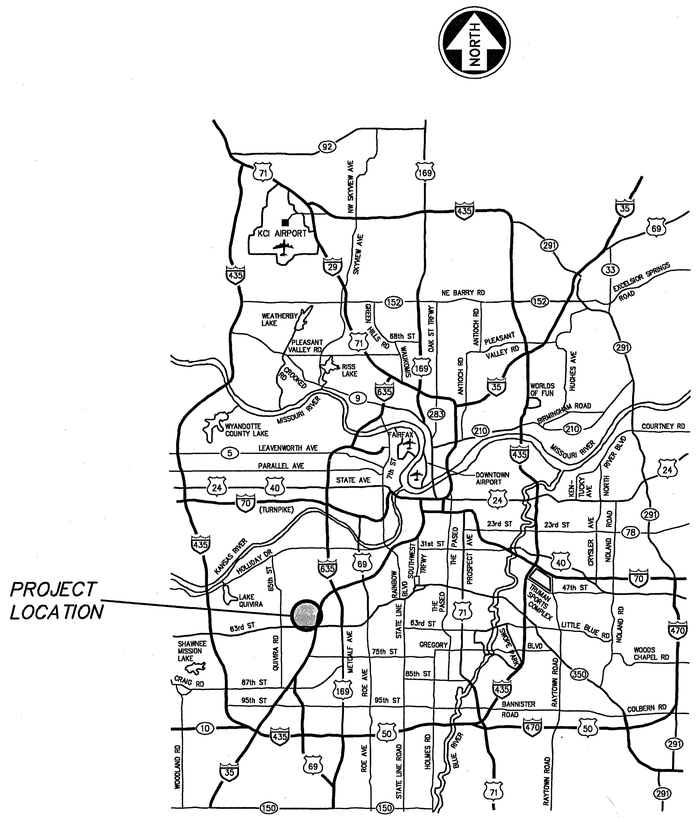
BENCHMARK #825 **ELEV.=915.129**

Barnsten Aluminum Disk stamped BM 825
Top of 15 inch wingwall at the SE corner of bridge over Turkey Creek on the South side of Johnson Drive.

AUTOCAD FILE: 14481GR.DWG	PROJECT: TURKEY CREEK TRAIL	TURKEY CREEK TRAIL	CHKD. BY: ABC
PLOTTED: XX-XX-XX	TIME: XX:XX A.M.	DATE: 2/21/03	DESIGNED: KJB	DRAWN BY: KJB

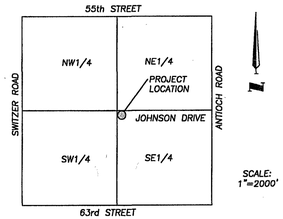
A:\800
P:\104481\CIVIL\BID SET\14481GR1.DWG
May 21,2003 9:33am

PLANS FOR
TURKEY CREEK TRAIL IMPROVEMENTS
JOHNSON COUNTY
MERRIAM, KANSAS
12-12-24



PROJECT LOCATION

LOCATION MAP
NOT TO SCALE



VICINITY MAP
SEC. 12-12-24

NO.	DATE	REVISIONS	BY	APP'D
1				
2				
3				
4				

LOCATION	PROJECT NUMBER	YEAR	SHEET NUMBER	TOTAL SHEETS
TURKEY CREEK TRAIL	104481	2003	1	11

COPYRIGHT © - 2003 - SHAFER, KLINE & WARREN, INC.

SHAFER, KLINE & WARREN, INC.
11100 W. 91st Street, Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS:
Iola, KS
Kansas City, MO
Chillicothe, MO
Macon, MO
CIVIL ENGINEERS ▪ ELECTRICAL ENGINEERS ▪ MECHANICAL ENGINEERS ▪ LANDSCAPE ARCHITECTS ▪ LAND SURVEYORS ▪ LAND PLANNERS

ENGINEER

GERALD C. JOHNSON LICENSED 1382 KANSAS PROFESSIONAL ENGINEER

5/21/03
DATE

OWNER:
JOHNSON COUNTY PARK AND RECREATION DISTRICT
7900 RENNER RD
SHAWNEE MISSION, KS 66219
(913) 438-7275

REVIEWED BY:

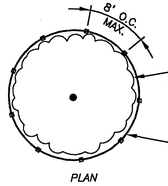
DATE

ACCEPTED BY:

DATE

#4

NOTE:
PRIOR TO COMMENCEMENT OF WORK, THE CONTRACTOR SHALL IDENTIFY, TAG AND PROTECT, EXISTING TREES TO BE SAVED. WORK CANNOT COMMENCE UNTIL TREE PROTECTION IS IN PLACE AND APPROVED BY THE ENGINEER. CONTRACTOR SHALL BE RESPONSIBLE FOR ANY DAMAGE TO EXISTING TREES THAT ARE TO REMAIN.



PLAN

TREE PROTECTION OF EXISTING TREES
NOT TO SCALE

JOHNSON DRIVE

TURKEY CREEK

CAMPBELL LANE

ADJUST VERITCAL ALIGNMENT OF EXISTING STORM SEWER TO PROVIDE 1'-0" COVER BETWEEN WALL FOOTING AND TOP OF PIPE – SEE DETAILS ON SHEET 3 of 11

PROPOSED CONCRETE RETAINING WALLS – SEE DETAILS ON SHEET 5 of 11

REMOVE EXISTING CONCRETE EMBANKMENT, NEW CONCRETE EMBANKMENT TO BE PLACED TO EXISTING LIMITS ABOVE AND BELOW PROPOSED TRAIL

EXISITING SANITARY SEWER MANHOLE TO BE ADJUSTED (SEE SHEET 3 of 11)

EXISITING ROCK WALL

PROVIDE TREE PROTECTION (SEE DETAIL THIS PAGE)

Sta. 0+00 =
N 267809.68
E 2251308.20

PROPOSED 10' CONCRETE TRAIL

INSTALL NEW ASPHALT WALK, MATCH GRADES ON NEW CONCRETE TRAIL

REMOVE EXISTING ASPHALT WALK

EXISTING POWER POLE TO BE RELOCATED BY OTHERS

FUTURE TRAIL BY OTHERS

Sta. 3+96.97 =
N 268185.69
E 2251262.80

BENCHMARK #825 **ELEV.=915.129**
Bernsten Aluminum Disk stamped BM 825 Top of 15 inch wingwall at the SE corner of bridge over Turkey Creek on the South side of Johnson Drive.

LEGEND

BM	= BENCHMARK
BW	= BOTTOM OF WALL
C	= CONCRETE
——X——	= FENCE LINE
℄	= FLOW LINE
GA	= GUY ANCHOR
LPCB	= LIGHT POLE CONC. BASE
LP	= LIGHT POLE
MS	= METAL SIGN
PPL	= POWER POLE W/ LIGHT
PPT	= POWER POLE W/ TRANSFORMER
PP	= POWER POLE
PRISR	= POWER RISER
SSMH	= SANITARY SEWER MANHOLE
SHRUB	= SHRUB
SCV	= SPRINKLER CONTROL VALVE
TC	= TOP BACK OF CURB
TA	= TOP OF ASPHALT
TW	= TOP OF WALL
TRVSTN	= TRAVERSE STATION
TREE	= TREE
WSH	= WATER SPRINKLER HEAD
WS	= WOOD SIGN

SCALE: 1" = 20'
20 10 0 20

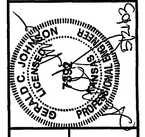
AUTOCAD VER. 2000i INFORMATION BLOCK	
DRAWING: 14481GR1.DWG	BY: KJB DATE: 10/9/02
XREF DWG1: 14481SUR	XREF DWG2: 14481BAS
XREF DWG3: NONE	XREF DWG4: NONE

KJB00
P:\10448\1\CAD\LBID SET\14481GR1.DWG
May 21,2003 8:42am

NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3				
2				
1				

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

Designed By KJB
Drawn By KJB
Checked By ABC
Issue Date: 5/22/03
Job No 104481



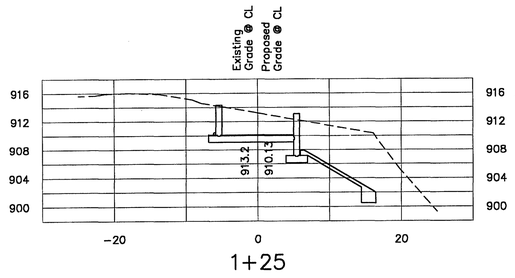
SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS: Kansas City, MO; Macon, MO; Ottawa, KS; Chillicothe, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

TURKEY CREEK TRAIL
MERRIAM, KANSAS

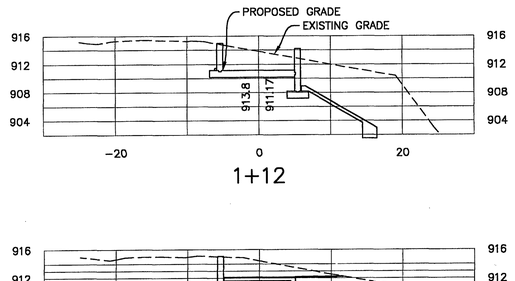
GRADING AND LAYOUT @ JOHNSON DRIVE

SHEET NO.
2 OF 11

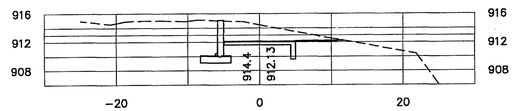
HORIZONTAL SCALE: 1" = 10'
VERTICAL SCALE: 1" = 10'



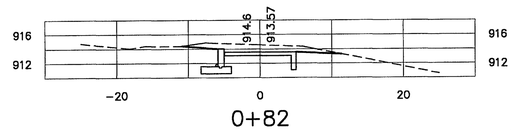
1+25



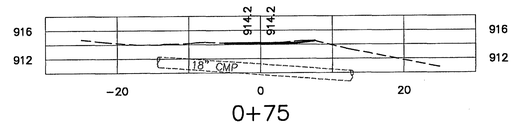
1+12



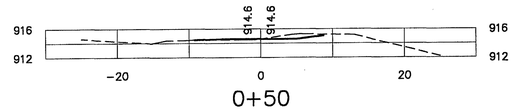
1+00



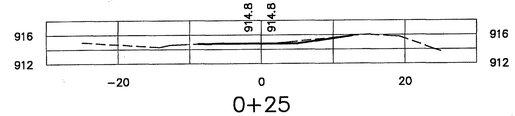
0+82



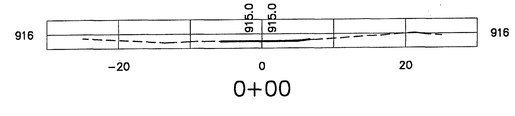
0+75



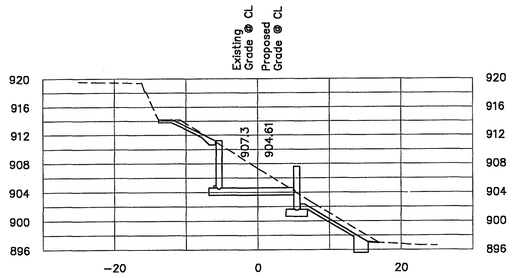
0+50



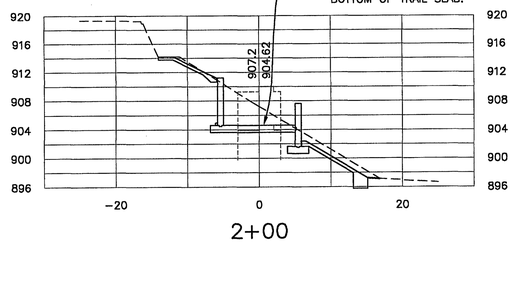
0+25



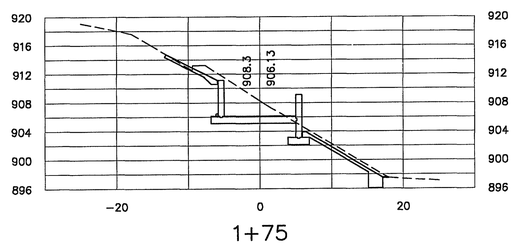
0+00



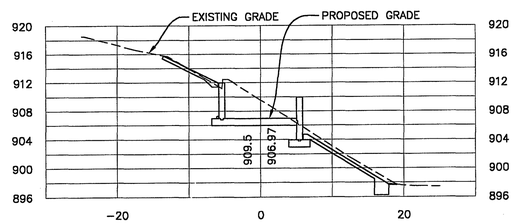
2+09.55



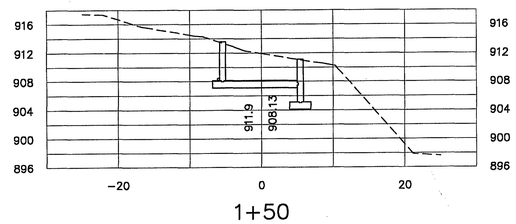
2+00



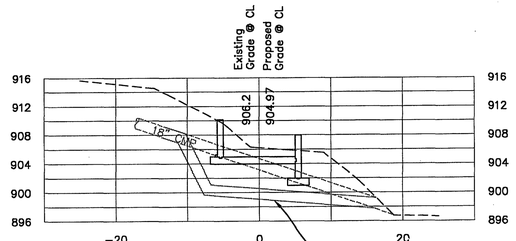
1+75



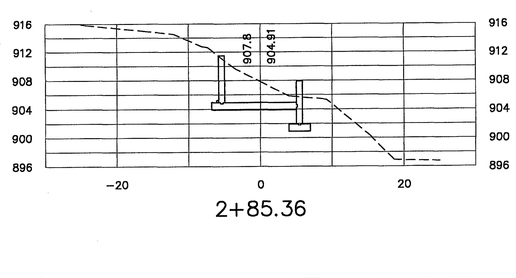
1+64.59



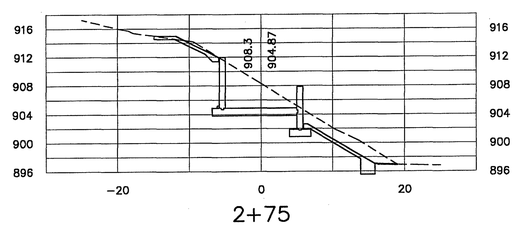
1+50



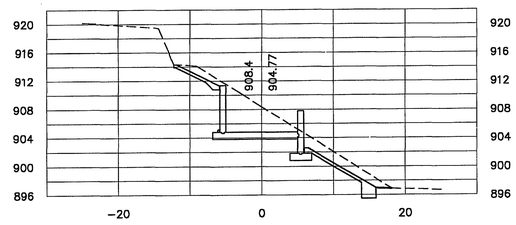
3+00



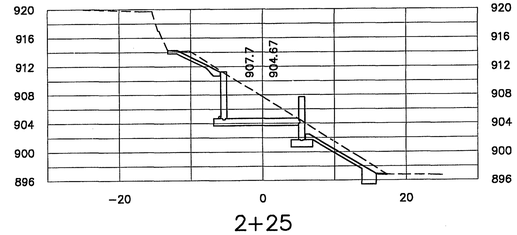
2+85.36



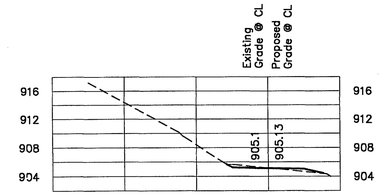
2+75



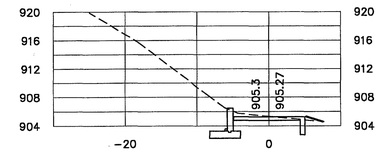
2+50



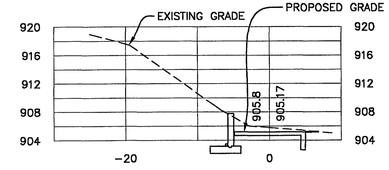
2+25



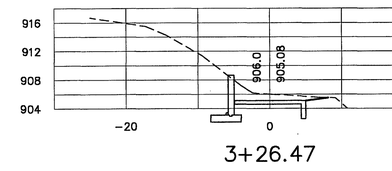
3+96.97



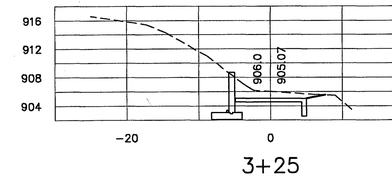
3+75



3+50



3+26.47



3+25

AUTOCAD VER. 2000i INFORMATION BLOCK	
DRAWING: 1448tGR1.DWG	DATE: 10/9/02
XREF DWG1: NONE	XREF DWG2: NONE
XREF DWG3: NONE	XREF DWG4: NONE

K.BOD
P:\10448t\ACAD\BID SET\1448tGR1.DWG
May 21,2003 9:33am

NO.	DATE	REVISIONS	BY	APP'D
1				
2				
3				
4				
5				

© -2002-SHAFER, KLINE & WARREN INC.

Designed By KJB
Drawn By KJB
Checked By ABC
Issue Date: 5/22/03
Job No. 10448t

GERALD C. JOHNSON 7283 PROFESSIONAL ENGINEER 5/21/03

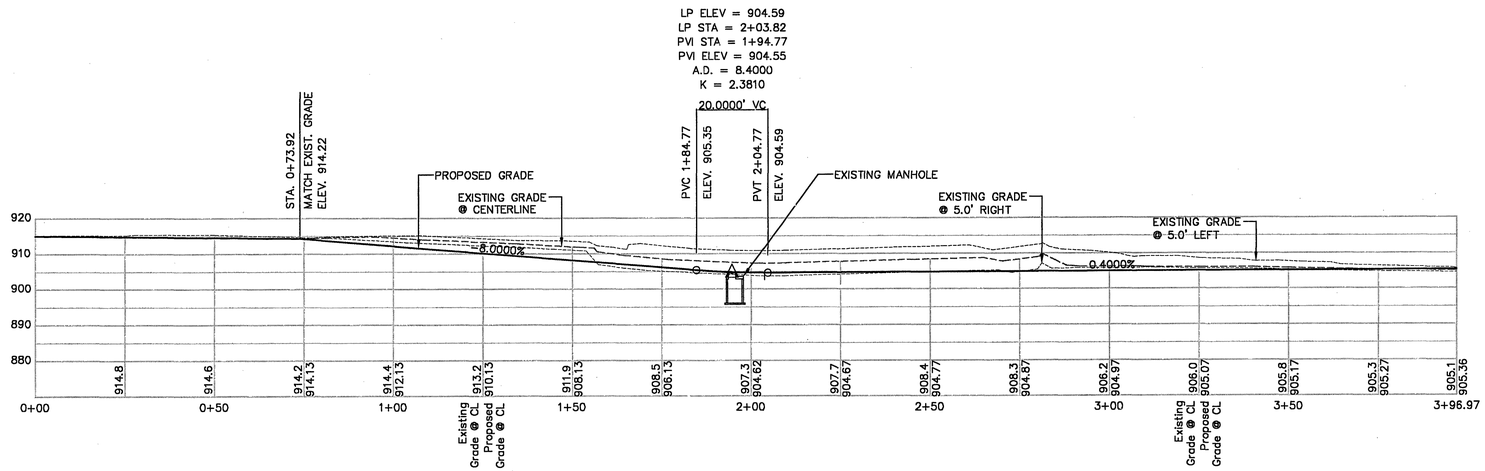
SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS: Iola, KS; Chillicothe, MO; Kansas City, MO; Macon, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

TURKEY CREEK TRAIL
MERRIAM, KANSAS

SECTIONS @ JOHNSON DRIVE

SHEET NO.
3 OF 11

HORIZONTAL SCALE: 1" = 20'
VERTICAL SCALE: 1" = 20'



NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3				
2				
1				

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

Designed By KJB
Drawn By KJB
Checked By ABC
Issue Date: 5/22/03
Job No. 104481

GERALD C. JOHNSON
LICENSED
7392
KANSAS
PROFESSIONAL ENGINEER
5/21/03

SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS:
Iola, KS
Chillicothe, MO
Kansas City, MO
Macon, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

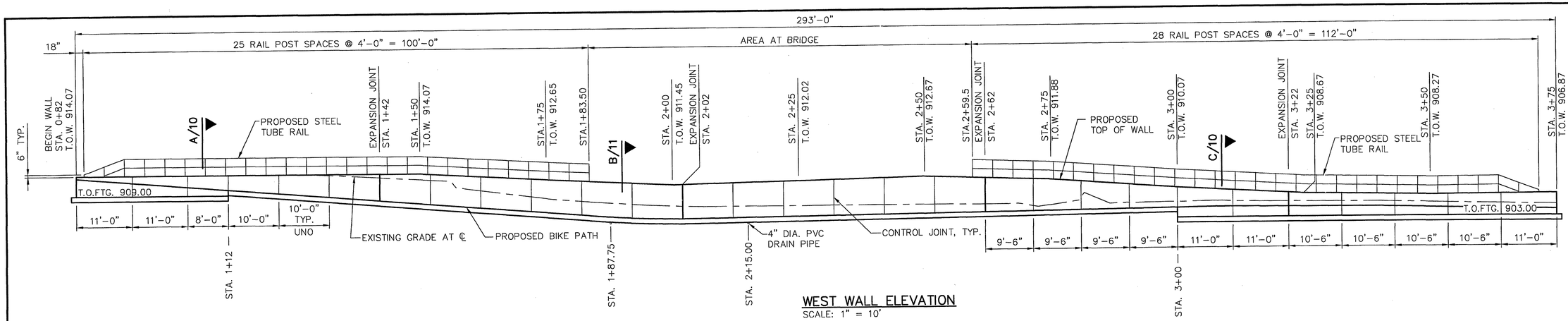
TURKEY CREEK TRAIL
MERRIAM, KANSAS

PROFILE @ JOHNSON DRIVE

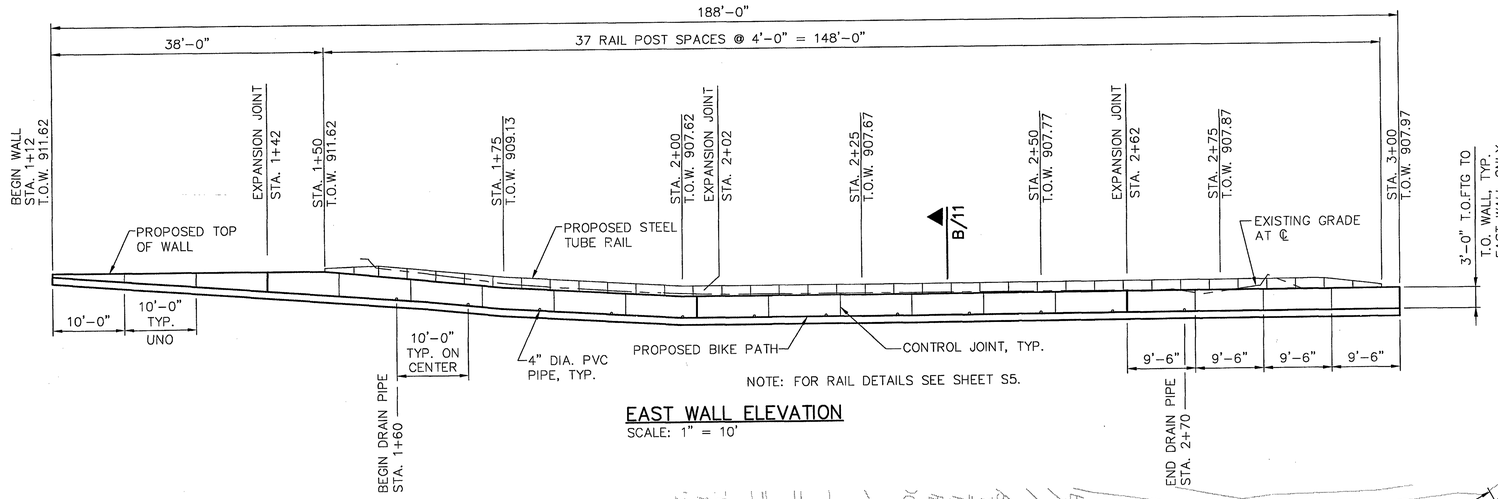
SHEET NO.
4 OF 11

AUTOCAD VER. 2000i INFORMATION BLOCK	
DRAWING: 14481GR1.DWG	BY: KJB DATE: 10/9/02
XREF DWG1: NONE	XREF DWG2: NONE
XREF DWG3: NONE	XREF DWG4: NONE

KJB00
P:\104481\ACAD\BID SET\14481GR1.DWG
May 21,2003 9:33am



WEST WALL ELEVATION
SCALE: 1" = 10'



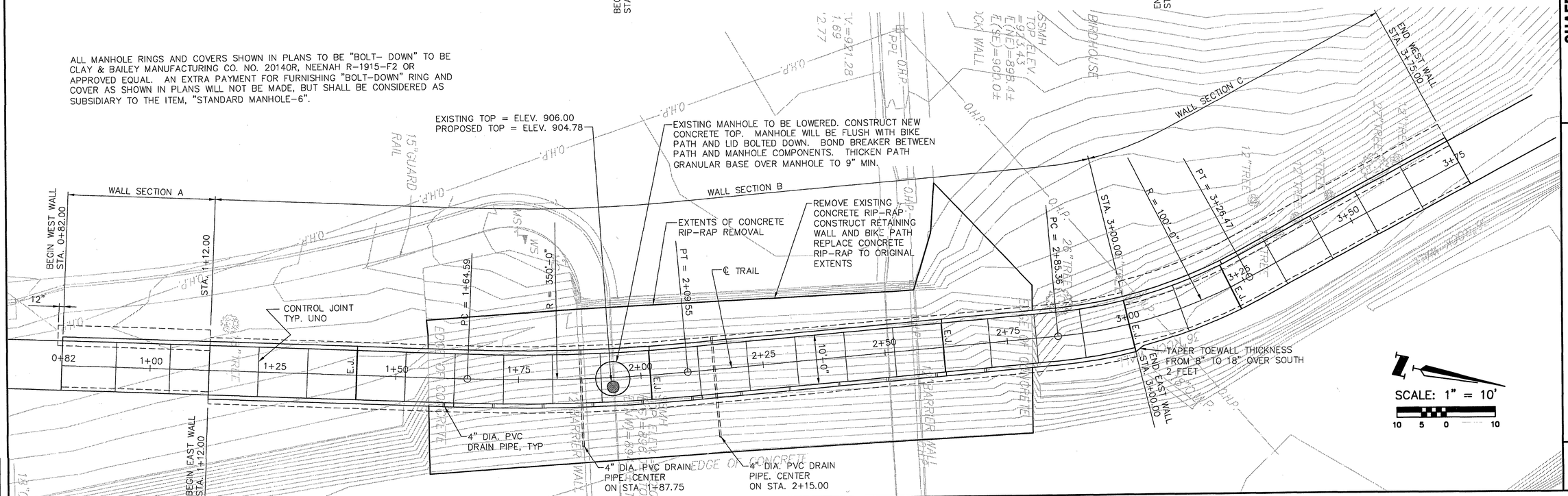
NOTE: FOR RAIL DETAILS SEE SHEET S5.

EAST WALL ELEVATION
SCALE: 1" = 10'

GENERAL NOTES FOR PRE-CAST MANHOLES

1. Reinforcement in all sections shall equal or exceed A.S.T.M. C-478 specifications.
2. Mastic material to be used at all sections joints. O-Rings may be used for joints below the cone section, but the cone section itself shall not have O-ring joints. Ⓔ
3. Standard cast iron manhole steps to be Clay & Bailey Manufacturing Co. No. 2102 or approved equal. Steel core, plastic coated steps may be used (M.A. Ind., Inc. No. PS1-PF, PS2-PF, or approved equal).
4. All manhole rings shall be set in a minimum of two (2) rows of 3/4 to 1 inch pre-formed bitumastic joint sealer.
5. All manhole rings to be placed in pavement or in areas to be subsequently paved shall have "Machined Horizontal Bearing Surfaces" and shall comply with Class #25 as established in ASTM A-48.

ALL MANHOLE RINGS AND COVERS SHOWN IN PLANS TO BE "BOLT- DOWN" TO BE CLAY & BAILEY MANUFACTURING CO. NO. 20140R, NEENAH R-1915-F2 OR APPROVED EQUAL. AN EXTRA PAYMENT FOR FURNISHING "BOLT-DOWN" RING AND COVER AS SHOWN IN PLANS WILL NOT BE MADE, BUT SHALL BE CONSIDERED AS SUBSIDIARY TO THE ITEM, "STANDARD MANHOLE-6".



	NO.	DATE	REVISIONS	BY	APP'D
	5				
	4				
	3				
	2				
	1				

COPYRIGHT © 2002-SHAFER, KLINE & WARREN INC.

Designed By GDS
Drawn By MEJ
Checked By GDS
Issue Date: 5/22/03
Job No. 104481

SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS: Ida, KS Chillicothe, MO Kansas City, MO Macon, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

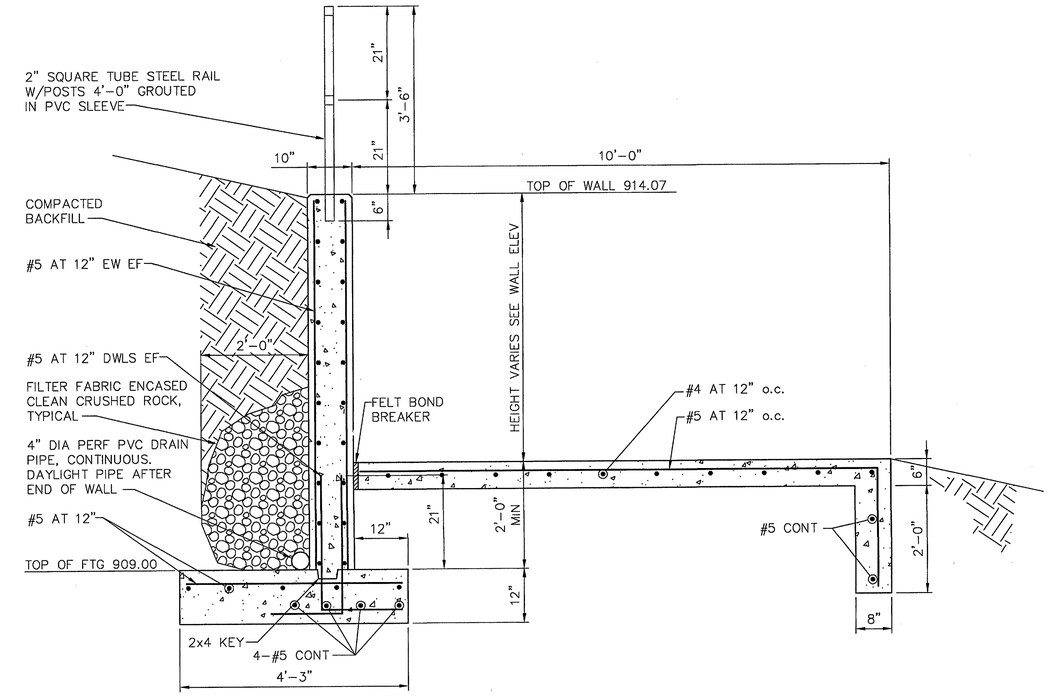
TURKEY CREEK TRAIL
MERRIAM, KANSAS

RETAINING WALLS PLAN AND ELEVATION

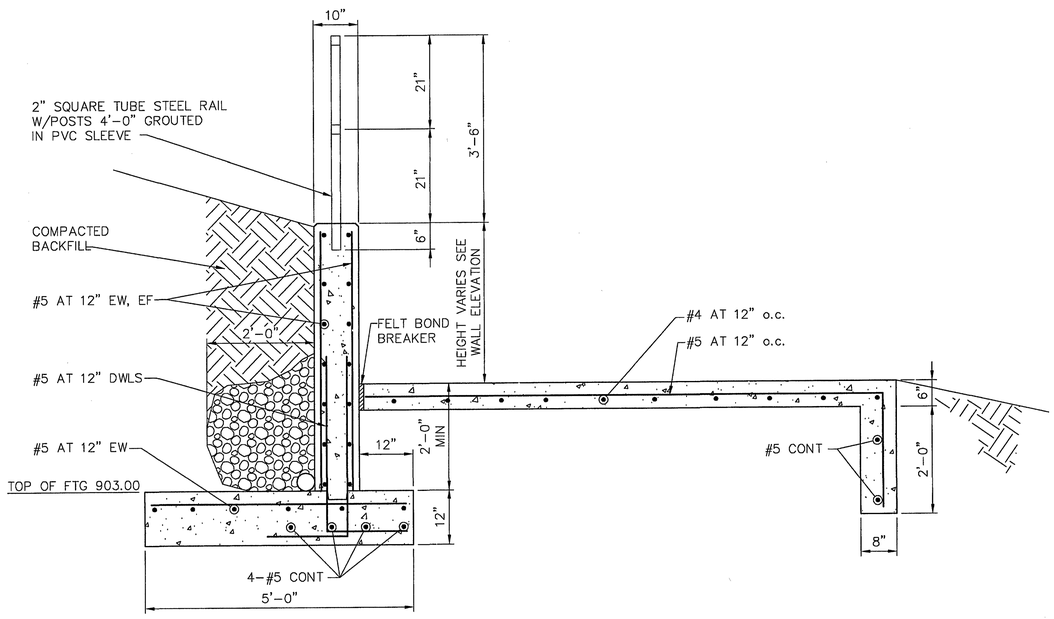
SHEET NO. 5 OF 11

AUTOCAD VER. 2000i INFORMATION BLOCK		
DRAWING: 14481-9-10.DWG	BY: MEJ	DATE: 2/11/03
XREF DWG1: NONE	XREF DWG2: NONE	
XREF DWG3: NONE	XREF DWG4: NONE	

K:\B00\104481\ACAD\PID SET\14481-9-11.DWG
May 21, 2003 9:34am



WALL SECTION A (STATION 0+82 TO 1+12)
SCALE: 3/4" = 1'-0"



WALL SECTION C (STATION 3+00 TO 3+75)
SCALE: 3/4" = 1'-0"

NO.	DATE	REVISIONS	BY	APP'D
1				
2				
3				
4				
5				

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

Designed By GDS
Drawn By MEJ
Checked By GDS
Issue Date: 5/22/03
Job No. 104481

GERALD C. JOHNSON 7392 REGISTERED PROFESSIONAL ENGINEER STATE OF KANSAS

SHAFER, KLINE & WARREN, INC.
11100 West 91st St. Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS: Kansas City, MO
Iola, KS Macon, MO
Chillicothe, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

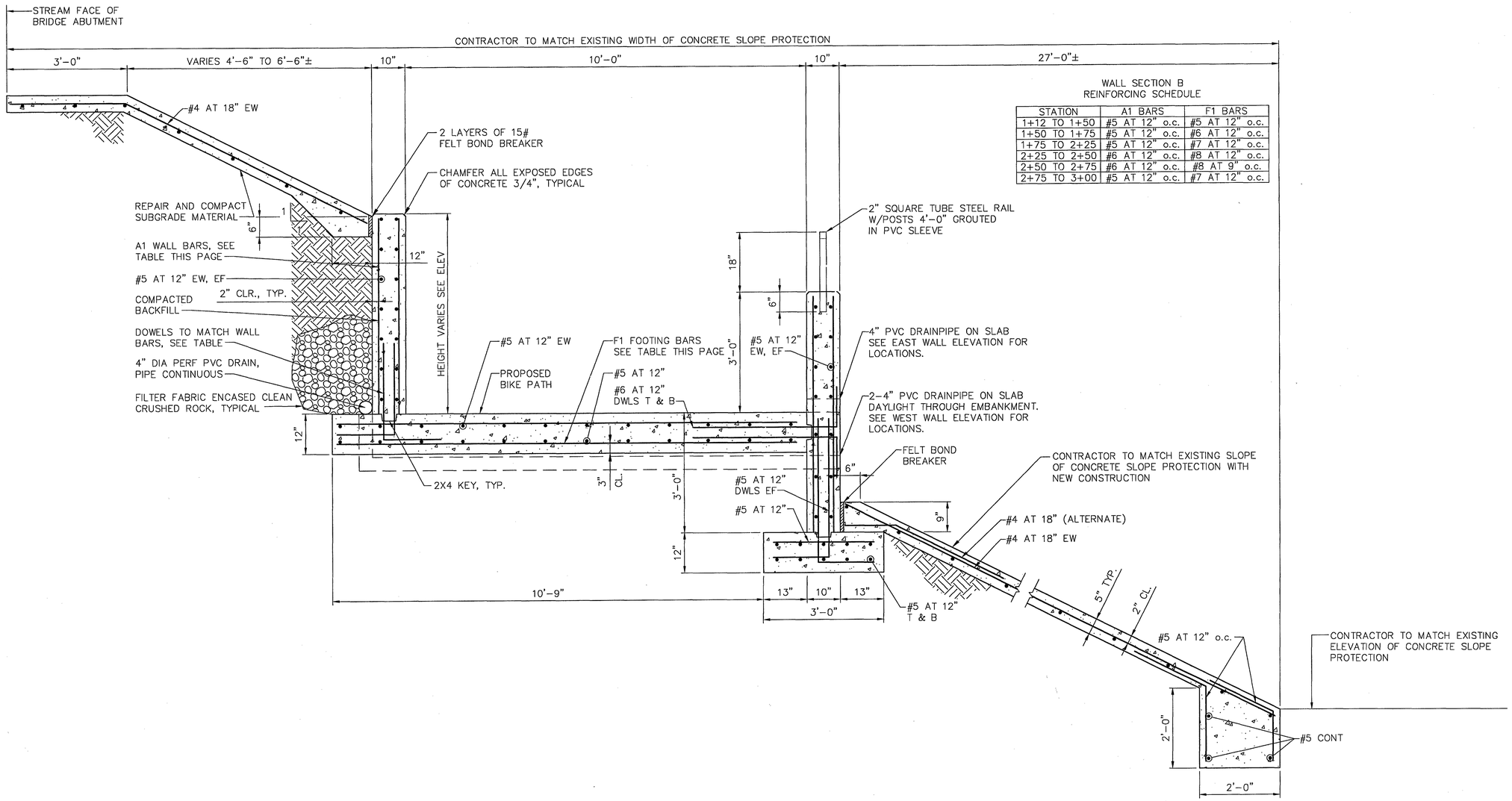
TURKEY CREEK TRAIL
MERRIAM, KANSAS

RETAINING WALL SECTIONS "A" & "C"

SHEET NO.
6 OF 11

AUTOCAD VER. 2000i INFORMATION BLOCK	
DRAWING:14481-10-10.DWG	BY: MEJ DATE:2/11/03
XREF DWG1:NONE	XREF DWG2:NONE
XREF DWG3:NONE	XREF DWG4:NONE

K:&DO
P:\104481\ACAD\&DO SET\14481-10-11.DWG
May 21,2003 8:34am



WALL SECTION B
REINFORCING SCHEDULE

STATION	A1 BARS	F1 BARS
1+12 TO 1+50	#5 AT 12" o.c.	#5 AT 12" o.c.
1+50 TO 1+75	#5 AT 12" o.c.	#6 AT 12" o.c.
1+75 TO 2+25	#5 AT 12" o.c.	#7 AT 12" o.c.
2+25 TO 2+50	#6 AT 12" o.c.	#8 AT 12" o.c.
2+50 TO 2+75	#6 AT 12" o.c.	#8 AT 9" o.c.
2+75 TO 3+00	#5 AT 12" o.c.	#7 AT 12" o.c.

WALL SECTION B
SCALE: 3/4" = 1'-0"

NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3				
2				
1				

Designed By GDS
Drawn By MEJ
Checked By GDS
Issue Date: 5/22/03
Job No. 104481

GERALD C. JOHNSON LICENSED PROFESSIONAL ENGINEER 3392 KANSAS 5/21/03

SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS: Kansas City, MO Macon, MO Iola, KS Chillicothe, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

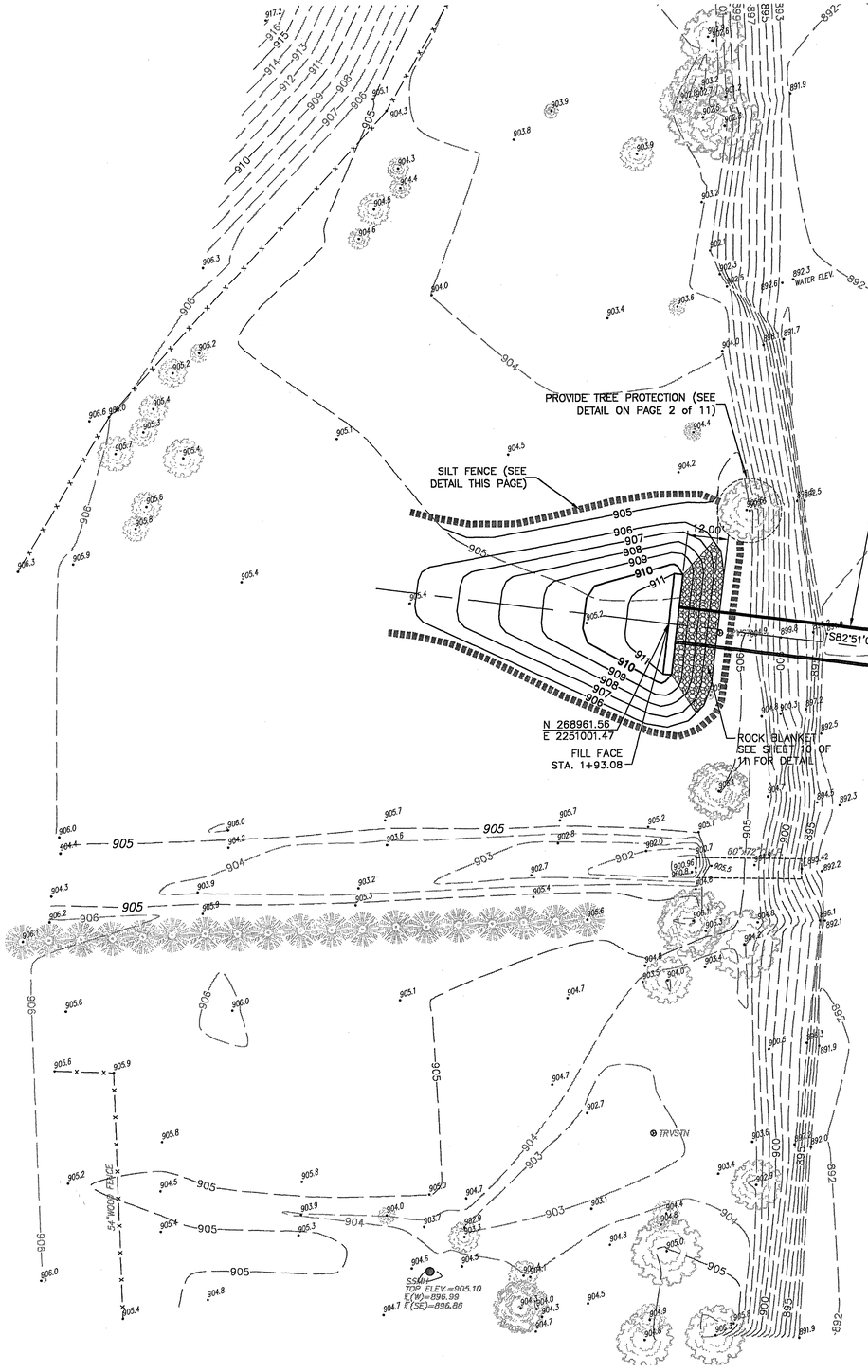
TURKEY CREEK TRAIL
MERRIAM, KANSAS

RETAINING WALL SECTION "B"

SHEET NO.
7 OF 11

AUTOCAD VER. 2000i INFORMATION BLOCK	
DRAWING: 14481-10-10.DWG	BY: MEJ DATE: 2/11/03
XREF DWG1: NONE	XREF DWG2: NONE
XREF DWG3: NONE	XREF DWG4: NONE

KJB00
P:\104481\ACAD\BID SET\14481-11-11.DWG
May 21,2003 9:34am



MERRIAM FARMERS MARKET

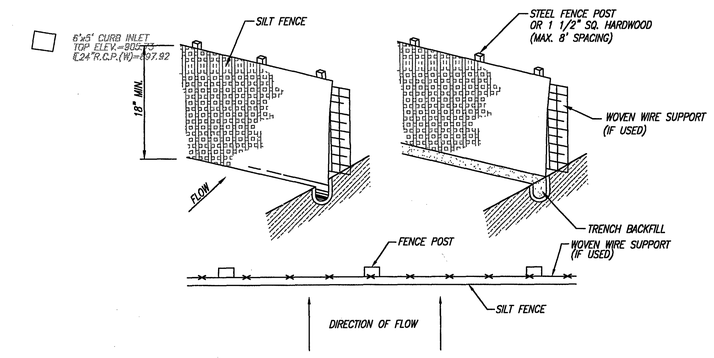
LEGEND

BM	= BENCHMARK
BW	= BOTTOM OF WALL
C	= CONCRETE
——X——	= FENCE LINE
FL	= FLOW LINE
GA	= GUY ANCHOR
LPCB	= LIGHT POLE CONC. BASE
LP	= LIGHT POLE
MS	= METAL SIGN
PPL	= POWER POLE W/ LIGHT
PPT	= POWER POLE W/ TRANSFORMER
PP	= POWER POLE
PRISR	= POWER RISER
SSMH	= SANITARY SEWER MANHOLE
SHRUB	= SHRUB
SCV	= SPRINKLER CONTROL VALVE
TC	= TOP BACK OF CURB
TA	= TOP OF ASPHALT
TW	= TOP OF WALL
TRVSTN	= TRAVERSE STATION
TREE	= TREE
WSH	= WATER SPRINKLER HEAD
WS	= WOOD SIGN

SCALE: 1" = 20'
20 10 0 20

CONSTRUCTION SPECIFICATIONS

1. POSTS WHICH SUPPORT THE SILT FENCE SHALL BE INSTALLED ON A SLIGHT ANGLE TOWARD THE ANTICIPATED RUNOFF SOURCE AND SPACED AT A MAX. 8' APART OR TO SILT FENCE MANUFACTURER SPECIFICATIONS, WHICHEVER IS STRICTER.
2. SILT FENCE SHALL BE TRENCHED IN WITH A SPADE OR MECHANICAL TRENCHER SO THAT THE DOWNSLOPE FACE OF THE TRENCH IS FLAT AND PERPENDICULAR TO THE LINE OF FLOW..
3. THE TRENCH SHOULD BE APPROXIMATELY 8" DEEP 3-4" WIDE TO ALLOW FOR THE SILT FENCE TO BE LAID IN THE GROUND AND BACKFILLED.
4. SILT FENCE SHOULD BE SECURELY FASTENED TO EACH SUPPORT POST OR TO WOVEN WIRE WHICH IS IN TURN ATTACHED TO THE FENCE POSTS, AND SHOULD NOT EXCEED 18" ABOVE THE GROUND SURFACE.
5. WOVEN WIRE SUPPORT SHALL BE A MINIMUM 12 GAUGE WITH A MAXIMUM OF 6"x6" OPENING. WOVEN WIRE SUPPORT SHALL BE INSTALLED FOR FLOWS LONGER THAN 200 FEET.
6. INSPECTION SHALL BE FREQUENT AND REPAIR OR REPLACEMENT SHALL BE MADE PROMPTLY AS NEEDED.
7. SILT FENCE SHALL BE REMOVED WHEN IT HAS SERVED ITS USEFULNESS SO AS NOT TO BLOCK OR IMPEDE STORM FLOW OR DRAINAGE.
8. SEDIMENT TRAPPED BY THIS PRACTICE SHALL BE UNIFORMLY DISTRIBUTED ON THE SOURCE AREA PRIOR TO TOPSOILING.
9. SILT FENCE SHALL BE MIRAFI-100X FILTER FABRIC OR APPROVED EQUAL.



SILT FENCE DETAIL

BENCHMARK #825 **ELEV.=915.129**

Bernsten Aluminum Disk stamped BM 825
Top of 15 inch wingwall at the SE corner of bridge over Turkey Creek on the South side of Johnson Drive.

NO.	DATE	REVISIONS	BY	APP'D
1				
2				
3				
4				
5				

© COPYRIGHT ©-2002-SHAFER, KLINE & WARREN INC.

Designed By KJB
Drawn By KJB
Checked By ABC
Issue Date: 5/22/03
Job No. 104481

DONALD C. JOHNSON 7392 LICENSED PROFESSIONAL ENGINEER STATE OF KANSAS

SHAFER, KLINE & WARREN, INC.
11100 West 91st St. Overland Park, KS 66214
FAX: 913/888-7600 913/888-7660
Kansas City, MO
Macon, MO
OTHER OFFICE LOCATIONS:
Iola, KS
Chillicothe, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

TURKEY CREEK TRAIL
MERRIAM, KANSAS

GRADING AND LAYOUT @ MERRIAM MARKET

SHEET NO.
8 OF **11**

AUTOCAD VER. 2000i INFORMATION BLOCK		
DRAWING: 104481GR1.DWG	BY: KJB	DATE: 10/9/02
XREF 1: 104481SUR	XREF 2:	
XREF 3: NONE	XREF 4: NONE	

K:\BID0
P:\104481\ACAD\BID SET\104481GR1.DWG
May 21,2003 9:34cm

1. GENERAL

1.1. DESIGN CRITERIA:

A. DESIGN LOADING:

1. EARTH 120 LBS/ CU. FT. EQUIVALENT FLUID PRESSURE 45 LBS/ CU. FT.

B. DESIGN UNIT STRESSES:

1. CONCRETE: JOHNSON COUNTY CONCRETE BOARD MIX JCCB5K, f'c = 5000 PSI.

2. REINFORCING STEEL (GRADE 60) fy = 60000 PSI.

3. STEEL PILING A709 GRADE 36 = 9 KSI

4. STRUCTURAL STEEL TUBING HANDRAIL A500 GRADE B

1.2. STRUCTURAL DESIGN DRAWINGS SHOW THE INTENT OF REQUIRED CONSTRUCTION AND SPECIFIC CONSTRUCTION AS NEEDED TO FACILITATE CLEAR DETAILING. FOR SPECIFIC CONDITIONS NOT SHOWN, THE CONTRACTOR SHALL PROVIDE DETAILS OF CONSTRUCTION SIMILAR TO THOSE SHOWN. VERIFY ALL EXISTING CONDITIONS, DIMENSIONS, AND ELEVATIONS PRIOR TO COMMENCING WORK. NOTIFY THE STRUCTURAL ENGINEER IN WRITING OF DISCREPANCIES OR CONTRADICTIONS ENCOUNTERED PRIOR TO EXECUTING ANY AFFECTED CONSTRUCTION.

1.3. CONSTRUCTION PRACTICE AND THE DESIGN, ADEQUACY, AND SAFETY OF TEMPORARY BRACING, SHORING, SUPPORTS, ETC., SHALL REMAIN THE SOLE RESPONSIBILITY OF THE CONTRACTOR.

1.4. BEFORE EXECUTING ANY WORK, THE CONTRACTOR SHALL EXAMINE ACTUAL JOB CONDITIONS AND REPORT TO THE ENGINEER ANY ERROR, OMISSION, OR DISCREPANCY AFFECTING WORK. REFER TO THE ADDITIONAL DRAWINGS IN THIS SET FOR INSERTS, SLEEVES, OPENINGS, EMBEDS, PADS, CURBS, ATTACHMENTS, AND OTHER ITEMS NOT SHOWN HEREIN AFFECTING STRUCTURAL WORK BUT NOT INDICATED ON THE STRUCTURAL PLANS.

1.5. CONTRACTOR SHALL SUBMIT SHOP DRAWINGS FOR REVIEW FOR CONFORMANCE WITH THE DESIGN CONCEPT. THE SHOP DRAWINGS SHALL BE PREPARED UNDER THE DIRECT SUPERVISION OF A LICENSED PROFESSIONAL ENGINEER REGISTERED IN THE STATE WHERE THE PROJECT IS TO BE CONSTRUCTED. SHOP DRAWINGS MUST BE SUBMITTED TO AND BE CHECKED BY THE CONTRACTOR AND SUBCONTRACTOR AND BEAR THE CHECKERS INITIALS BEFORE SUBMISSION TO THE ENGINEER FOR APPROVAL. NO FABRICATION SHALL PROCEED BEFORE SHOP DRAWINGS COVERING THAT WORK HAVE BEEN APPROVED.

1.6. THESE NOTES TO BE READ IN CONJUNCTION WITH ANY WRITTEN SPECIFICATIONS AND THESE DRAWINGS. IN THE EVENT OF A CONFLICT THE ENGINEER SHALL BE NOTIFIED FOR CLARIFICATION.

1.7. THE OWNER OR HIS REPRESENTATIVE RESERVES THE RIGHT TO INSPECT ANY MATERIAL, FABRICATION, OR WORKMANSHIP AT ANY TIME IN FIELD OR SHOP FOR CONFORMANCE TO THE SPECIFICATIONS AND DRAWINGS.

1.8. DO NOT BACKFILL BEFORE 7 DAYS AFTER RETAINING WALL IS IN PLACE.

2. FOUNDATIONS

2.1. SPREAD FOOTINGS SHALL HAVE A DESIGN SOIL BEARING PRESSURE OF 2000 PSF.

2.2. ALL FOOTINGS SHALL BEAR ON UNDISTURBED SOIL AND SHALL BE LOWERED TO FIRM BEARING, IF SUITABLE SOIL IS NOT FOUND AT FOOTING ELEVATIONS INDICATED ON THESE DRAWINGS.

3 CONCRETE

3.1. MIXING AND PLACING OF ALL CONCRETE AND SELECTION OF MATERIALS SHALL BE IN ACCORDANCE WITH THE BUILDING CODE AND THE SPECIFICATIONS.

3.2. PROPORTIONS OF AGGREGATE TO CEMENT SHALL CREATE A DENSE, WORKABLE MIX WHICH CAN BE PLACED WITHOUT SEGREGATION OR EXCESS SURFACE WATER.

3.3. UNLESS SPECIFIED OTHERWISE, ALL TOLERANCES FOR CONCRETE SHALL CONFORM TO ACI 117.

3.4. ALL FORMWORK SHALL BE DESIGNED, ERECTED, SUPPORTED, BRACED AND MAINTAINED IN ACCORDANCE WITH ACI STANDARD 347.

3.5. WALLS AND SLABS MAY BE CORE DRILLED ONLY AS DETAILED OR AS APPROVED IN WRITING BY THE ENGINEER. VERIFY LOCATION OF REINFORCING BEFORE CORE DRILLING; NO REINFORCING SHALL BE CUT.

3.6. THE CONTRACTOR SHALL SUBMIT DRAWINGS SHOWING THE PROPOSED CONSTRUCTION AND CONTROL JOINTS FOR ALL CONCRETE, INCLUDING WALLS, SLABS ON GRADE, ETC., TO THE ENGINEER FOR REVIEW PRIOR TO PLACING ANY CONCRETE.

3.7. ALL CONCRETE WORK SHALL BE IN ACCORDANCE WITH THE CURRENT EDITIONS OF "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE, ACI-318".

3.8. CONCRETE MIXES SHALL BE AS APPROVED BY THE JOHNSON COUNTY CONCRETE BOARD AS MEETING DESIGNATIONS JCCB5K.

3.9. LAP ALL BARS AT SPLICES ACCORDING TO THE ACI CODE REQUIREMENTS BUT NOT LESS THAN 42 BAR DIAMETERS NOR LESS THAN 18 INCHES UNLESS OTHERWISE NOTED. ALL HORIZONTAL WALL BARS SHALL BE DEVELOPED AT CORNERS EITHER BY BENDING NOT LESS THAN 40 BAR DIAMETERS AROUND CORNERS OR WITH PROPERLY PLACED HOOKED AND LAPPED CORNER BARS.

3.10. ALL VERTICAL BARS SHALL LAP NOT LESS THAN 30 BAR DIAMETERS NOR LESS THAN 2'-6" UNLESS OTHERWISE NOTED.

3.11. EXPANSION JOINTS SHALL BE SPACED AT 60'-0" MAXIMUM CENTERS. REINFORCING BARS SHALL NOT BE CONTINUOUS THROUGH THE JOINT.

3.12. ALL CONCRETE WALLS SHALL BE PROPERLY BRACED AND HELD IN LINE UNTIL SUPPORTING SLABS OR FLOORS ARE IN PLACE.

3.13 ALL BAR STEEL SHALL BE PROPERLY SUPPORTED AND HELD ACCURATELY IN PLACE AS RECOMMENDED BY THE CONCRETE REINFORCING STEEL INSTITUTE, EXCEPT THAT MAXIMUM SPACING OF ANY BAR SUPPORT SHALL BE 3'-0". ALL TOP SLAB BARS SHALL BE SUPPORTED BY CONTINUOUS HIGH CHAIRS.

3.14 ALL DOWELS SHALL BE ACCURATELY PLACED AND SECURELY HELD IN POSITION BEFORE CONCRETE IS POURED. STICKING OF DOWELS INTO FRESH OR PARTIALLY HARDENED CONCRETE WILL NOT BE ACCEPTABLE.

3.15. ALL OPENINGS IN SLABS AND WALLS SHALL HAVE ADDITIONAL 2-#5X4'-0" DIAGONAL BARS AT EACH CORNER OF OPENING EXTENDING 2'-0" INTO ADJACENT EXCEPT WHERE SHOWN OTHERWISE.

4 REINFORCING STEEL

4.1. ALL CONCRETE IS REINFORCED CONCRETE UNLESS SPECIFICALLY INDICATED OTHERWISE. REINFORCE ALL CONCRETE NOT SPECIFICALLY DETAILED IN ACCORDANCE WITH TYPICAL DETAILS AND OTHER, SIMILAR CONDITIONS.

4.2. MILD REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60.

4.3. DEFORMED BAR ANCHORS ARE TO BE RESISTANCE WELDED, CONFORMING TO ASTM A496, WITH A MINIMUM YIELD STRENGTH OF 70,000 PSI.

4.4. CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH SHALL HAVE A MINIMUM COVER OF 3 INCHES.

4.5. CONCRETE EXPOSED TO EARTH WATER (RAW OR TREATED) OR WEATHER SHALL HAVE A MINIMUM COVER OF 1 1/2 INCHES FOR #5 BARS OR SMALLER, AND 2 INCHES FOR #6 BAR OR LARGER.

4.6. REINFORCING STEEL SHALL NOT BE TACK WELDED FOR ANY REASON. NO REINFORCING BARS SHALL BE WELDED (EXCEPT DEFORMED ANCHORS).

5. PRE ENGINEERED BRIDGE SPECIFICATIONS

5.1. THE PEDESTRIAN BRIDGE TO BE A LOW-PROFILE BRIDGE UTILIZING A PONY-TRUSS DESIGN, ALSO KNOWN AS A HALF-TRUSS DESIGN, U-FRAME STIFFNESS.

5.2. THE BRIDGE SHALL HAVE AN INSIDE NOMINAL DECK WIDTH OF 10 AND A CLEAR SPAN LENGTH OF 137 FEET.

5.3. THE STANDARD BRIDGE SHALL BE DESIGNED FOR A MINIMUM LIVELOAD OF 60 POUNDS PER SQUARE FOOT.

5.4. BRIDGE APPLICATIONS, SHALL BE DESIGNED IN ACCORDANCE WITH THE "SPECIFICATIONS FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS BY THE AMERICAN INSTITUTE OF STEEL" (AISC) - LATEST EDITION.

5.5. WELDED TUBULAR STRUCTURAL DESIGN SHALL BE IN ACCORDANCE WITH THE STRUCTURAL WELDING CODE (ANSI/AWS D1.1--8)-CHAPTER 10. TUBULAR STRUCTURES.

5.6. RAILING HEIGHT SHALL BE 54 INCHES ABOVE THE DECK TO THE TOP OF TRUSS TOP CHORD.

5.7. ONE DIAGONAL STEEL MEMBER SHALL BE INSTALLED PER PANEL.

5.8. THE BRIDGE CAMBER AT CENTER OF BRIDGE SPAN SHALL BE 2 1/2% OF THE TOTAL BRIDGE SPAN. THE LOCALIZED DECK SLOPE SHALL NOT EXCEED 8.33%.

5.9. THE BRIDGE SHALL BE FABRICATED FROM HIGH STRENGTH, SELF-WEATHERING, LOW ALLOW, ATMOSPHERIC CORROSION RESISTANT ASTM A847 COLD-FORMED WELDED SQUARE AND RECTANGULAR TUBING, AND ASTM A588, ASTM A606, OR ASTM A242 PLATE AND STRUCTURAL SHAPES (FY=50,000 PSI)

5.10. FIELD SPLICES SHALL BE FULLY BOLTED WITH ASTM A325 TYPE 3 HIGH STRENGTH BOLTS IN ACCORDANCE WITH "SPECIFICATIONS FOR STRUCTURAL JOINTS USING ASTM A325 OR A490 BOLTS".

5.11. ALL WELDING SHALL UTILIZE E80 SERIES ELECTODES WHICH HAVE THE SAME WEATHERING CHARACTERISTICS AS CORROSION-RESISTANT STEEL.

5.12. ALL DECKING MATERIAL SHALL BE PRESSURE TREATED SOUTHERN YELLOW PINE SELECT STRUCTURAL PLANKS GRADED ACCORDING TO WCLB STANDARD GRADING. DECKING TO BE TREATED TO AWPA STANDARD P-5. PRESERVATIVES UTILIZED SHALL BE EITHER AMMONIACAL COPPER ARSENATE (CHEMONITE OR ACA) OR CHROMATED COPPER ARSENATE (CCA). DECKING SHALL BE TREATED TO A TOTAL ABSORPTION OF 0.40 POUNDS PER CUBIC FOOT OF WOOD, OR TO REFUSAL.

5.13. THE DECK PLANKS SHALL BE NOMINAL SIZE OF 3" X 12" FOR EQUESTRIAN AND 10,000 POUND VEHICLE LOADS.

5.14. SAFETY RAILS SHALL BE CONTINUOUS STEEL WITH A CLEAR OPENING OF 4 INCHES. CONTINUOUS 6 INCH HIGH TOE PLATE, 1 1/2 INCHES ABOVE THE DECK LEVEL.

5.15. NOMINAL 2" X 6" SOUTHERN YELLOW PINE RUB RAIL SHALL BE ON THE INSIDE OF THE BRIDGE, PLACED 32 INCHES ABOVE THE TOP OF THE DECK.

5.16. WORKMANSHIP, FABRICATION AND SHOP CONNECTIONS SHALL BE IN ACCORDANCE WITH AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS SPECIFICATIONS (AASHTO).

5.17. WELDING OPERATORS SHALL BE EXPERIENCED IN THE TYPE OF WELDING REQUIRED TO ASSEMBLE THE PEDESTRIAN BRIDGE AND SHALL MEET ALL CITY REQUIREMENTS AND CERTIFICATIONS.

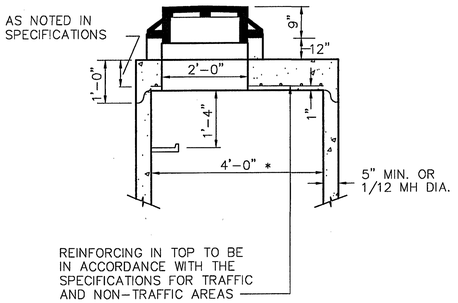
5.18. TO AID IN PROVIDING A UNIFORMLY "WEATHERED" APPEARANCE, ALL EXPOSED SURFACES OF STEEL SHALL BE BLAST CLEANED IN ACCORDANCE WITH STEEL STRUCTURES PAINTING COUNCIL SURFACE PREPARATION SPECIFICATIONS NO. 7 BRUSH-OFF BLAST CLEANING, SSPC-SP7 LATEST EDITION. EXPOSED SURFACES OF STEEL SHALL BE DEFINED AS THOSE SURFACES SEEN FROM THE DECK AND FROM OUTSIDE OF THE STRUCTURE. STRINGERS, FLOOR BEAMS, LOWER BRACE DIAGONALS AND THE INSIDE FACE OF THE TRUSS BELOW DECK AND BOTTOM FACE OF THE BOTTOM CHORD SHALL NOT BE BLASTED.

5.19. ALL CONCRETE FOR THE BRIDGE ABUTMENTS SHALL BE JCCB5K 5000 PSI.

6. STRUCTURAL STEEL TUBE HANDRAIL

6.1. HANDRAIL AT THE EAST AND WEST ENDS OF THE PEDESTRIAN BRIDGE SHALL BE GIVEN 1 SHOP COAT OF PRIMER AND THEN TWO COATS OF PAINT, THE COLOR TO BE DETERMINED BY THE OWNER.

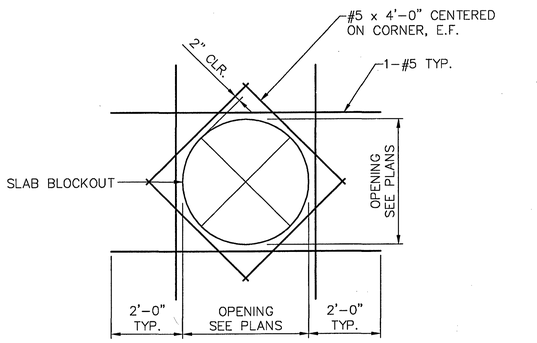
6.2. THE HANDRAIL ON TOP OF THE 10" RETAINING WALLS SHALL BE HOT DIPPED GALVANIZED IN ACCORDANCE WITH ASTM 123.



TO BE USED ONLY WHEN SPECIFIED IN PLANS

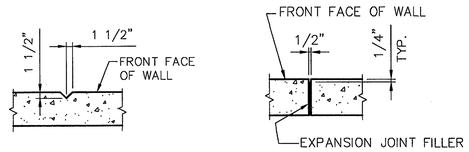
STANDARD PRECAST MANHOLE (SHALLOW TYPE)

NOT TO SCALE



TYPICAL SLAB/WALL OPENING REINFORCING

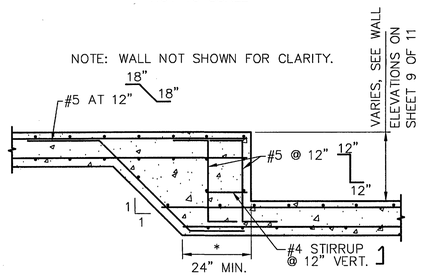
NOT TO SCALE



WALL CONTROL JOINT — TYP. WALL EXPANSION JOINT

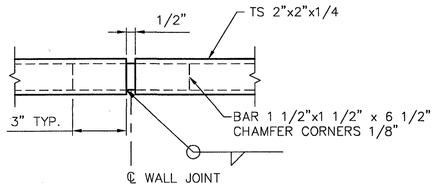
WALL JOINT DETAILS

NOT TO SCALE



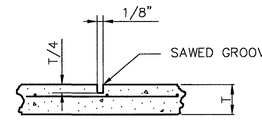
FOOTING STEP DETAIL

NOT TO SCALE

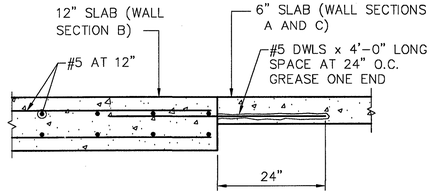


RAIL SLIP JOINT AT WALL EXPANSION JOINT

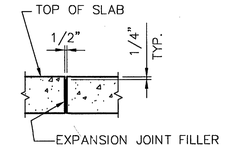
NOT TO SCALE



SAWED CONTROL JOINT DETAIL



SLIP JOINT DETAIL



SLAB EXPANSION JOINT

SLAB JOINT DETAILS

NOT TO SCALE

NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3				
2				
1				

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

Designed By CDS
Drawn By SRH
Checked By CDS
Issue Date: 5/22/03
Job No 104481

SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS:
Ida, KS
Chillicothe, MO
Kansas City, MO
Macon, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

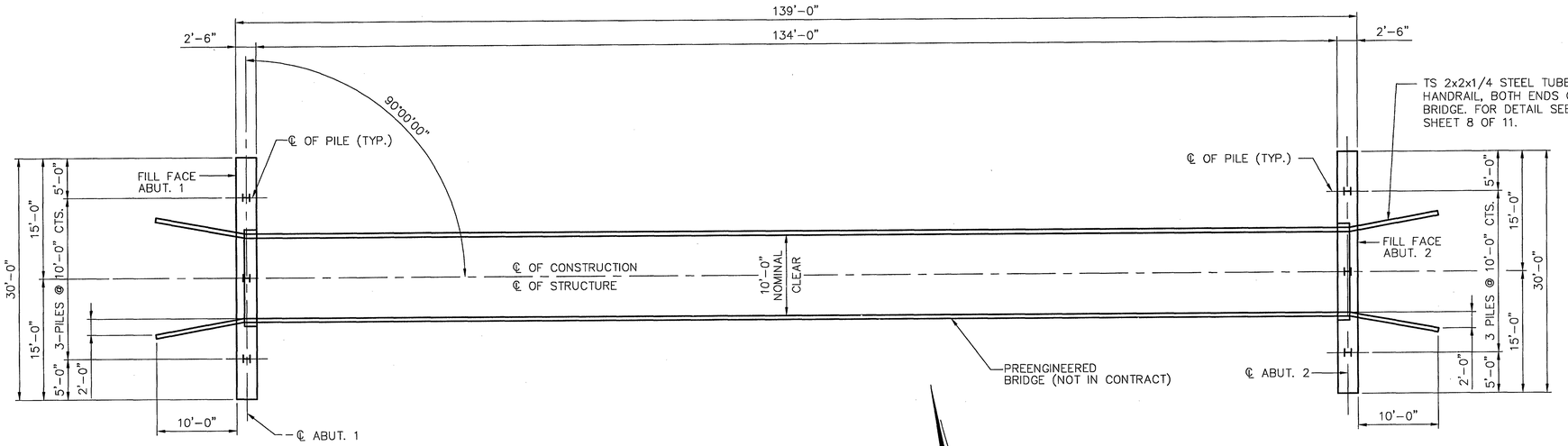
TURKEY CREEK TRAIL
MERRIAM, KANSAS

GENERAL NOTES AND STANDARD DETAILS

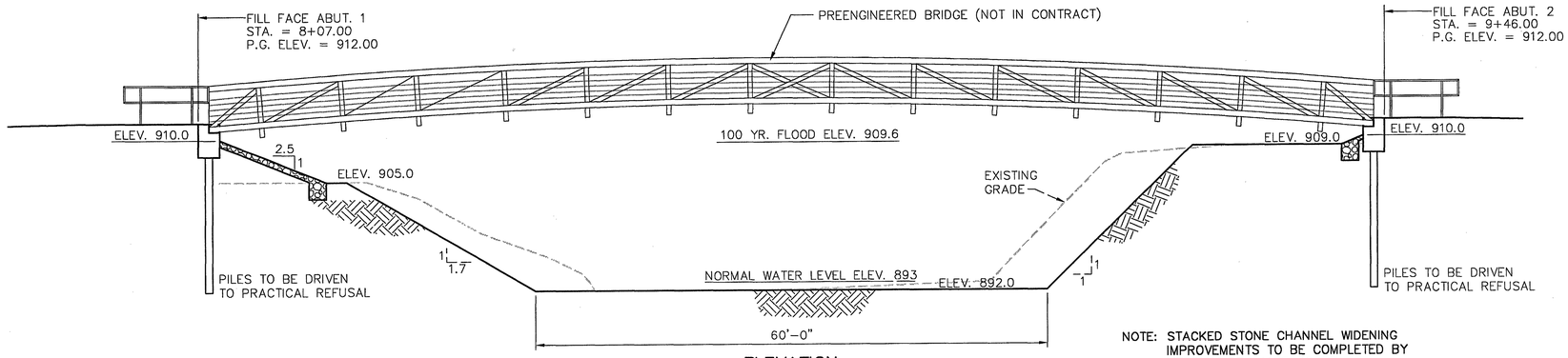
SHEET NO.
9 OF 11

AUTOCAD VER. 2000i INFORMATION BLOCK	
DRAWING: 14481-6-10.DWG	BY: MEJ / DATE: 2/12/03
XREF DWG1: NONE	XREF DWG2: NONE
XREF DWG3: NONE	XREF DWG4: NONE

K:\BDD
P:\104481\ACAD\BID SET\14481-6-11.DWG
May 21,2003 9:34am

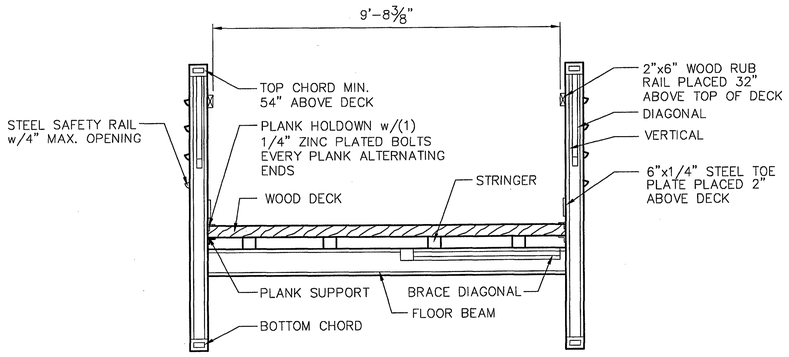


PLAN
NOT TO SCALE

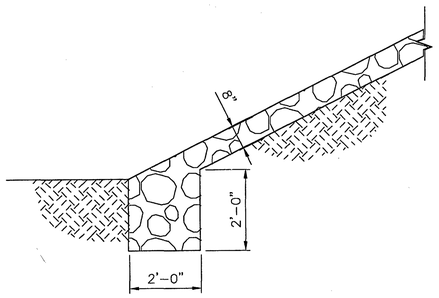


ELEVATION
NOT TO SCALE

NOTE: STACKED STONE CHANNEL WIDENING IMPROVEMENTS TO BE COMPLETED BY MERRIAM DRAINAGE BOARD.



BRIDGE SECTION
NOT TO SCALE



TYPE 2 ROCK BLANKET DETAIL
NOT TO SCALE

NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3				
2				
1				

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

Designed By GDS
Drawn By SRH
Checked By GDS
Issue Date: 5/22/03
Job No. 104481

GERALD G. JOHNSON 73924 STATE OF KANSAS REGISTERED PROFESSIONAL ENGINEER 5/21/03

SHAFER, KLINE & WARREN, INC.
11100 West 91st St. Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS: Kansas City, MO; Idaia, KS; Macon, MO; Chillicothe, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

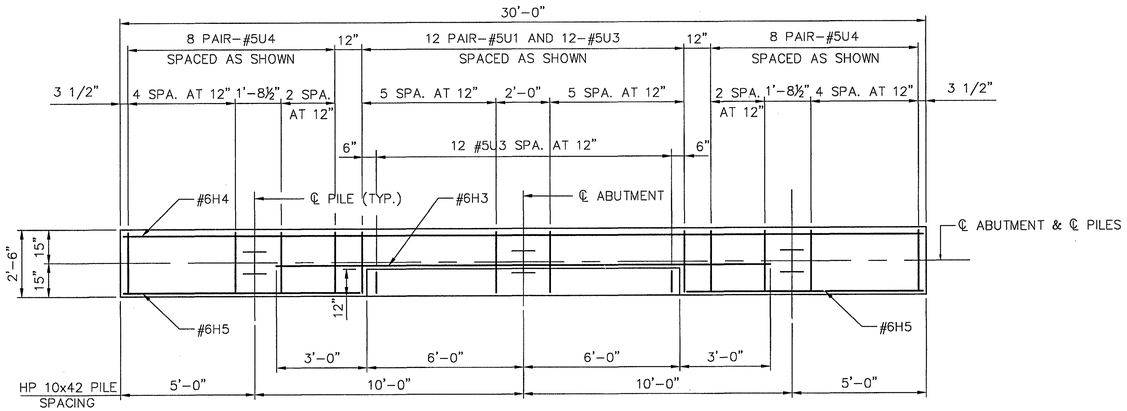
TURKEY CREEK TRAIL
MERRIAM, KANSAS

PEDESTRIAN BRIDGE PLAN AND ELEVATION

SHEET NO.
10 OF 11

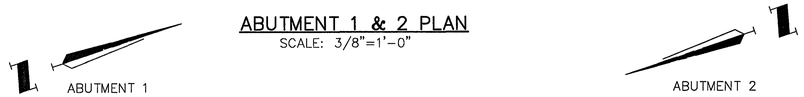
AUTOCAD VER. 2000i INFORMATION BLOCK		
DRAWING: 14481-7-10.DWG	BY: MEJ	DATE: 2/11/03
XREF DWG1: NONE	XREF DWG2: NONE	
XREF DWG3: NONE	XREF DWG4: NONE	

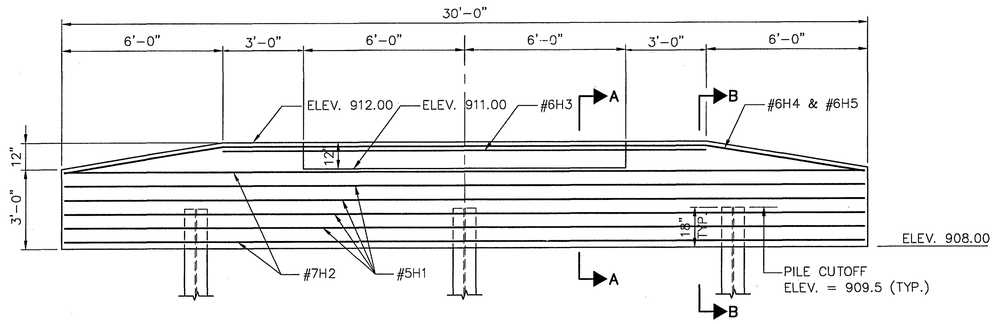
K.BROD
P:\10448\ACAD\SET\14481-7-11.DWG
May 21,2003 9:34am



ABUTMENT 1 & 2 PLAN

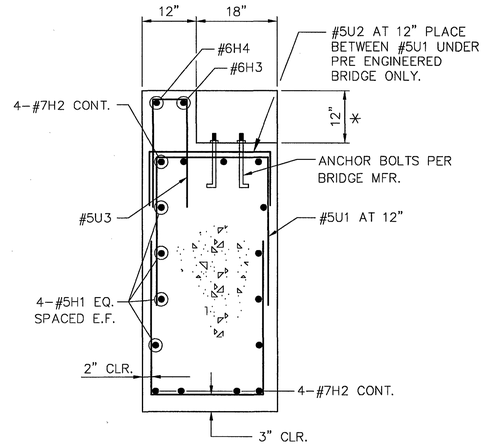
SCALE: 3/8"=1'-0"





ABUTMENT 1 & 2 ELEVATION

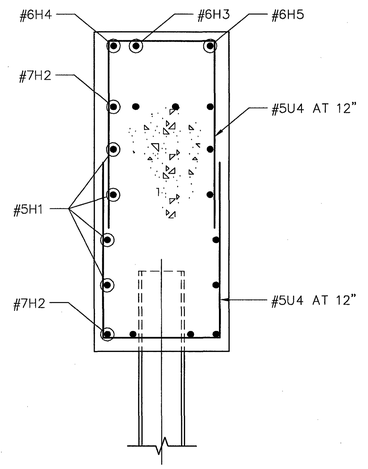
SCALE: 3/8"=1'-0"



* VERIFY DIMENSION WITH BRIDGE MFR.

SECTION A-A

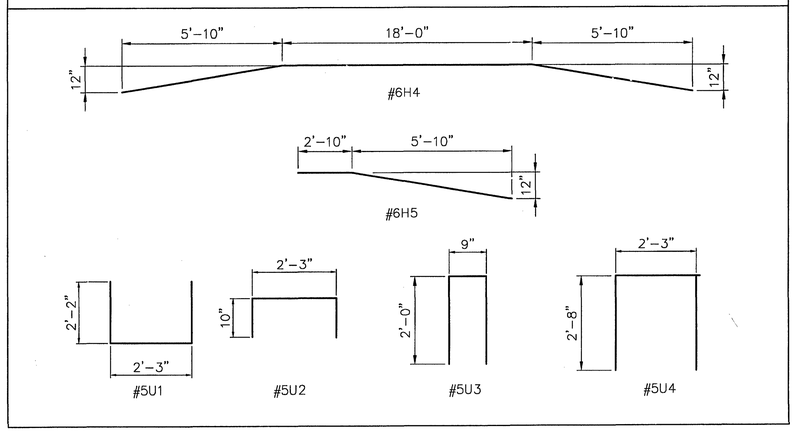
SCALE: 3/4"=1'-0"

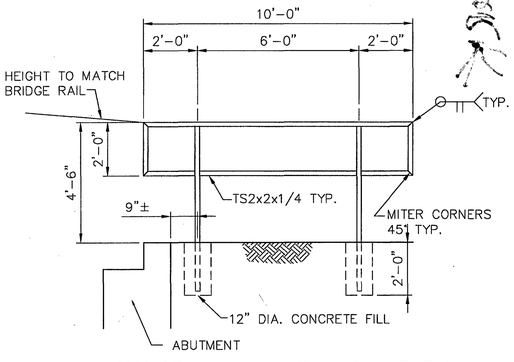


SECTION B-B

SCALE: 3/4"=1'-0"

BAR BENDING DIAGRAM





HANDRAIL ELEVATION ABUTMENT 2

SCALE: 3/8"=1'-0"

NO.	DATE	REVISIONS	BY	APP'D
5				
4				
3				
2				
1				

COPYRIGHT © -2002-SHAFER, KLINE & WARREN INC.

Designed By GDS
Drawn By SRH
Checked By GDS
Issue Date: 5/22/03
Job No. 104481

GERALD C. JOHNSON LICENSED PROFESSIONAL ENGINEER 7392 KANSAS 5/21/03

SHAFER, KLINE & WARREN, INC.
11100 West 91st St., Overland Park, KS 66214
913/888-7800 FAX: 913/888-7868
OTHER OFFICE LOCATIONS:
Iola, KS
Chillicothe, MO
Kansas City, MO
Macon, MO
CIVIL ENGINEERS ELECTRICAL ENGINEERS MECHANICAL ENGINEERS LANDSCAPE ARCHITECTS LAND SURVEYORS LAND PLANNERS

TURKEY CREEK TRAIL
MERRIAM, KANSAS

PEDESTRIAN BRIDGE DETAILS

SHEET NO.
11 OF 11

AUTOCAD VER. 2000i INFORMATION BLOCK
DRAWING: 14481-B-10.DWG BY: MEJ DATE: 2/11/03
XREF DWG1: NONE XREF DWG2: NONE
XREF DWG3: NONE XREF DWG4: NONE

K:\B00
P:\104481\ACAD\BID SET\14481-B-11.DWG
May 21,2003 9:34am

Proj. Stage	State	Proj. No.	Sheet Year	Sheet No.	Total Sheets
S	Kansas	169-46 K1228 (1)	1965	12	23

(1)

30'-0" End of Wearing Surface to ℄ 6 ℄ Panels @ 8'-3" = 53'-7½"

Cr. Gr.

24'-6" ℄ of Abut #1 & Piers

12:1 Slope

4" Tile Drain

PC (Outside)

3'-3"

17'-6"

Ground Line

Flow Line El 839.60

1'-6"

2'-0"

Ftg El 887.5

5'-0"

HALF ELEVATION

1'-6"

3'-0"

Ftg El 887.5

Symmetrical about ℄

12'-6"

25'-0"

5'-3"

5'-3"

12"

2" 3'-6"

Cr. Gr. El 912.0

PC

3'-6"

Const Joint (Horizontal)

Const Joint

3"

#b₁ @ 24" ctrs Splice to b₃

#b₂ @ 12" ctrs Each face

#Ab₁ @ 24" ctrs Splice to Ab₃

#Ab₂ @ 6" ctrs Abut

#Ab₃ @ 12" ctrs Splice to Ab₂

#Ab₄ @ 12" ctrs

17'-6"

2"

Const Joint

Const Joint

Ftg El 887.5

3'-0"

5'-0"

Ftg El 887.5

HALF LONGITUDINAL SECTION AT ℄

#C₃₋₈ Bars 37 Sp @ 12" = 37'-0"

#a₁₁ 65 Sp @ 3'-0"

#a₁₁ 65 Sp @ 3'-0"

#b₄ @ 12" ctrs

2'-8" 2'-0" 2'-8"

25'-0"

12'-0"

5'-0" Sidewalk

34'-0"

10'-0"

24'-0"

30'-0"

10'-0"

70'-0"

12'-0"

5'-0"

9'

35'-0"

℄ Semi Circular Depression at Drains

Sec I

Sec II

#b₅ @ 6" ctrs Splice to #b₅ @ 6" ctrs

#b₆ @ 12" ctrs Splice to #b₆ @ 12" ctrs

#b₇ @ 12" ctrs

#b₈ @ 12" ctrs

35'-0"

8'-6"

#b₉ @ 12" ctrs

0'-6" 9'-0" 8'-6"

HALF PLAN

#a₁₀ Spacing 35 Sp @ 12" = 3'-0" 35 Sp @ 2'-0" 35 Sp @ 12" = 12'-0" 35 Sp @ 2'-0"

Stirrups in Curb @ 12" ctrs #a₉ #a₁₀ #a₁₁ #a₁₂

12'-6" 25'-0"

HALF PLAN SHOWING REINFORCING

Cr. Gr. El 912.0

5'-0"

Cant 6" @ ℄ of Abut #2

Bent Steel in Top Slab

Reinf Steel in Bottom Slab

#a₁ #a₂ #a₃ #a₄ #a₅ #a₆ #a₇ #a₈ #a₉ #a₁₀ #a₁₁ #a₁₂

℄ Span 12'-6"

℄ Wall

2'-0" 2'-0" 2'-0" 12" 12" 12" 12" 12" 12" 6"

Top Floor Slab

Ordinates: y = ax²

R = 10 Ft

9" 1'-6" 2'-3" 5'-0" @ ℄

Const Joint

HAUNCH ORDINATES

5'-0"

1'-6" 12" 2'-6"

#Af₆ #Af₅ #Af₄ #Af₃ #Af₂ #Af₁

Const Joint

2'-0"

5'-0"

4 Sp @ 12" = 4'-0"

#Af₆ Splice to Af

FOOTING DETAILS AT ℄ OF ABUTMENT

3'-0"

12" 12" 12"

#f₂

Const Joint

#f₁

9 Sp @ 10" = 3'-6" #f₁ Splice to f₄

3'-0"

FOOTING DETAILS AT ℄ OF PIER

NO.	DATE	REVISIONS	BY	APP'D

STATE HIGHWAY COMMISSION OF KANSAS

BR NO 169-46-242 STA 56+06

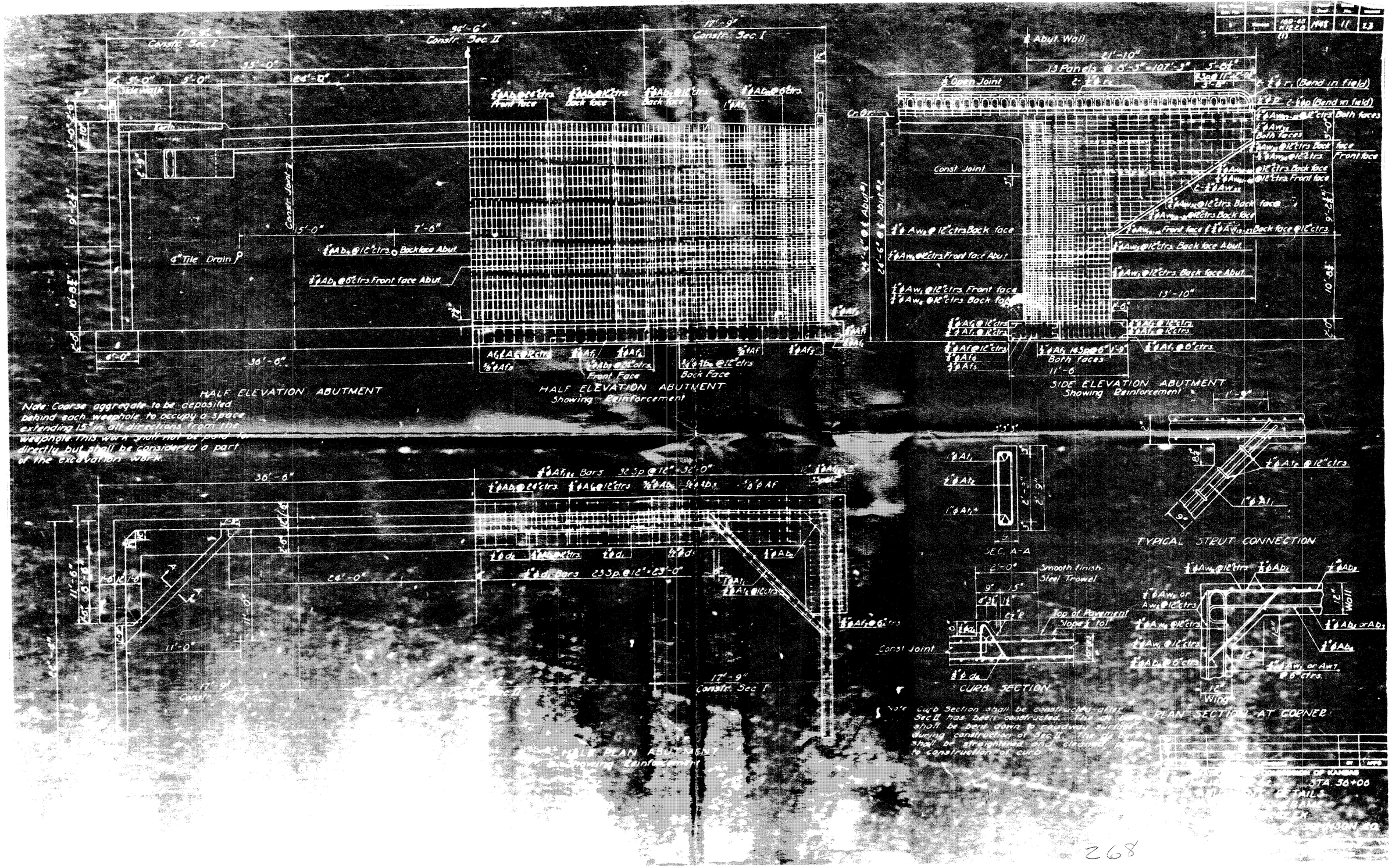
3 @ 24' RIGID FRAME TURKEY CREEK

169-46-K1228 (1) JOHNSON CO

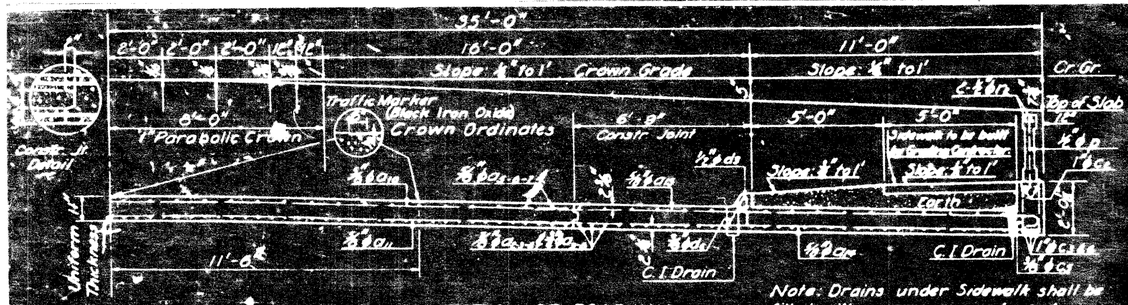
SHEET NO 6 OF 23 SHEETS SCALE ¼" = 1'-0"

DESIGNED BY Z.E.W. DATE 12-47 DETAILED BY R.E.W. TRACED BY

CHECKED BY W.J.A. DATE 2-48 APPROVED BY DATE

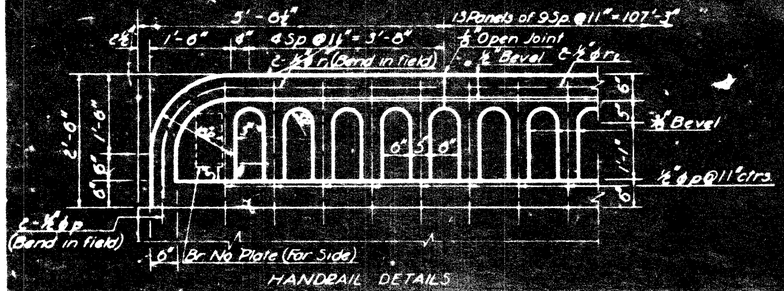

HALF ELEVATION ABUTMENT
HALF ELEVATION ABUTMENT
Showing Reinforcement
SIDE ELEVATION ABUTMENT
Showing Reinforcement
Note: Coarse aggregate to be deposited behind each weephole to occupy a space extending 15" in all directions from the weephole. This work shall not be paid for directly but shall be considered a part of the excavation work.
TYPICAL STRUT CONNECTION
SEC. A-A
CURB SECTION
PLAN SECTION AT CORNER
HALF PLAN ABUTMENT
Showing Reinforcement
Note: Curb Section shall be constructed after Sec II has been constructed. The dr. bars shall be bent down to roadway surface during construction of Sec II. The dr. bars shall be straightened and cleaned prior to construction of curb.
STA. 36+00
Const. Joint
Const. Sec. I
Const. Sec. II
Abut. Wall
Open Joint
Tile Drain
Front Face
Back Face
Both faces
Smooth finish Steel Trowel
Top of Pavement Slope ¼ to 1
Wing
Wall

268

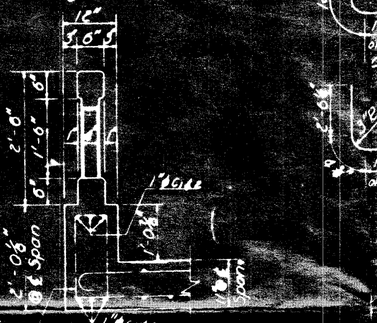


HALF SECTION OF ROADWAY AT ℄ OF SPANS

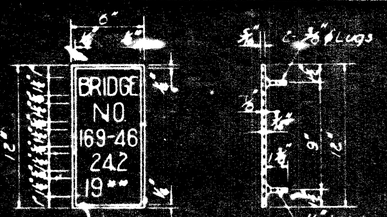
Note: Drains under Sidewalk shall be filled with coarse aggregate and covered with fine gravel to a depth of 2" and a distance of 18" around the drain.



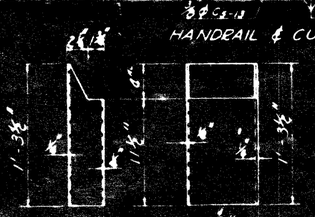
HANDRAIL DETAILS



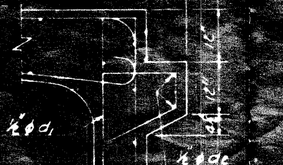
HANDRAIL & CUBB DETAILS



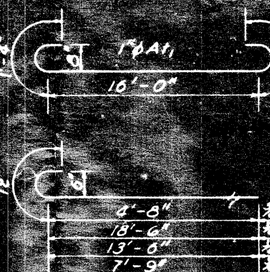
BRIDGE NUMBER PLATE
2 Required



CAST IRON DRAIN
Computed Wt. 190 No. Req'd 6

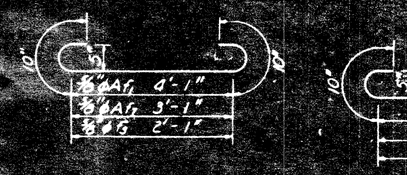


PAVING REST DETAILS

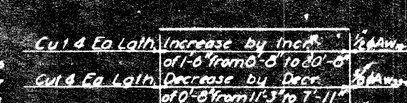


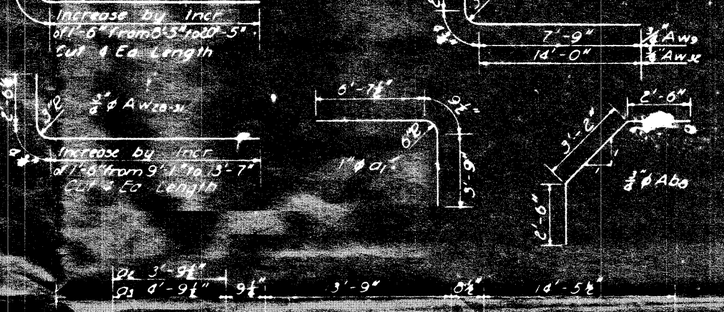
4'-8"	½"ø Af5
18'-6"	½"ø Af6
13'-6"	½"ø Af7
7'-9"	½"ø Aw8
21'-7"	½"ø Aw9
4'-2"	½"ø fc

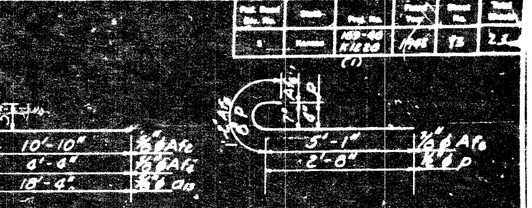




Cut & Ea. Lgth.	Increase by Incr. of 1'-6" from 0'-8" to 20'-8"
Cut & Ea. Lgth.	Decrease by Decr. of 0'-8" from 11'-3" to 7'-11"
Cut & Ea. Lgth.	Decrease by Decr. of 0'-8" from 11'-3" to 7'-11"
Cut & Ea. Lgth.	Decrease by Decr. of 0'-8" from 7'-3" to 3'-5"







BILL OF REINFORCING STEEL

Bar	No.	Size	Shape	Length	Bar	No.	Size	Shape	Length
Af1	32	⅝	—	20'-5"	Af4	16	1"	—	18'-8"
Af2	130	⅝	—	5'-9"	Af8	60	½	—	6'-0"
Af3	16	⅝	—	11'-6"					
Af4	16	⅝	—	11'-0"	f1	8	⅝	—	36'-4"
Af5	130	⅝	—	5'-2"	f2	280	⅝	—	5'-2"
Af6	262	⅝	—	3'-6"	f3	142	⅝	—	3'-9"
Af7	134	⅝	—	6'-3"	f4	16	⅝	—	12'-3"
Af8	56	⅝	—	4'-9"	b	40	½	—	36'-4"
Af10	16	⅝	—	36'-7"	b1	280	½	—	22'-2"
Ab5	22	⅝	—	36'-4"	b2	80	½	—	18'-6"
Ab6	274	⅝	—	6'-8"	a1	136	1"	—	11'-2"
Ab7	44	⅝	—	10'-6"	a2	68	⅝	—	36'-1"
Ab8	138	⅝	—	18'-8"	a3	66	⅝	—	35'-1"
Ab9	44	⅝	—	36'-4"	a4	60	⅝	—	6'-11"
Ab10	60	⅝	—	19'-6"	a5	68	⅝	—	13'-6"
Ab11	82	⅝	—	14'-6"	a6	50	½	—	18'-2"
Ab12	172	⅝	—	8'-2"	a7	67	½	—	17'-10"
					a8	136	⅝	—	27'-3"
Aw1	40	⅝	—	8'-3"	a9	67	½	—	27'-0"
Aw2	36	⅝	—	22'-9"	a10	27	½	—	36'-4"
					a11	53	⅝	—	36'-4"
Aw4	40	⅝	—	6'-3"					
Aw5	32	⅝	—	12'-5"	c	28	1"	—	18'-0"
					c1	12	½	—	19'-0"
Aw7	28	⅝	—	16'-5"	c2	12	1"	—	20'-8"
					c3	12	1"	—	18'-0"
Aw9	40	⅝	—	10'-0"	c4	*	⅝	—	*
Aw9-10	30	⅝	—	*	c5	*	⅝	—	*
Aw9-11	36	⅝	—	*	d1	90	½	—	4'-9"
Aw9-12	16	⅝	—	*	d2	4	½	—	36'-4"
Aw10	20	⅝	—	16'-11"	d3	8	½	—	28'-4"
Aw11	12	⅝	—	22'-7"	d4	54	⅝	—	6'-0"
Aw12	16	⅝	—	21'-10"	d5	8	½	—	7'-6"
Aw13-14	24	⅝	—	*	r	8	½	—	6'-0"
Aw13-15	24	⅝	—	*	m	52	½	—	6'-0"
Aw14	56	⅝	—	*					
Aw15	8	⅝	—	2'-7"	p	262	½	→	3'-4"
Aw16	8	⅝	—	19'-0"					
					d6	54	½	→	19'-2"
					d14	106	½	—	18'-8"

* See Bending Diagrams

SUMMARY OF QUANTITIES

Class I Excavation	610 Cu. Yds.
Class II Excavation	847 Cu. Yds.
Class AAA Concrete	583.8 Cu. Yds.
Reinforcing Steel	97490 Lbs.
Cast Iron	198 Lbs.
Removal of Existing Structure	Lump Sum

Note: Round bars of equivalent cross-sectional area may be substituted for ℄a bars.

GENERAL NOTES

Class AAA concrete shall be used throughout. All concrete shall be vibrated.

Bevel all exposed edges with a ¾" ∆ molding unless otherwise noted.

Construction Joints shall be made only at points indicated or as approved by the Engineer.

Falsework shall remain in place under each section of superstructure until the concrete has attained its design strength. The Handrail shall be constructed after falsework has been removed.

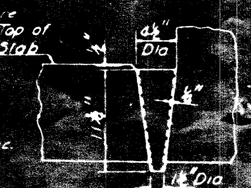
All parts of Cast Iron Drains accessible after erection shall receive one coat of tinted aluminum paint followed by one coat aluminum paint.

The Bridge Contractor shall remove existing structure (64' Steel Truss Span) after new has been constructed & opened to traffic.

Loading H20-44 (A.A.S.H.O. Specifications)

Basic Stresses: f'c = 3000 p.s.i. fs = 20,000 p.s.i. (Reinf.)

2 Year Bridge is completed



CAST IRON CONICAL DRAIN
Computed Wt. 0.0 No. Req'd 16

BENDING DIAGRAMS
All dimensions to ℄ of Bars

Bar	No.	W	Lgth.	Bar	No.	W	Lgth.
c3	56	1'-3"	5'-0"	Cm	12	2'-0"	5'-6"
c4	12	1'-3"	5'-2"	Cn	12	2'-4"	5'-9"
c5	12	1'-0"	5'-2"	Cp	12	2'-8"	5'-11"
Cr	12	1'-0"	5'-5"	Cr	12	3'-0"	6'-2"
Cs	12	1'-3"	5'-5"				

STATE HIGHWAY COMMISSION OF KANSAS
BR. NO. 169-46-24.2 STA. 56+06
3@24' RIGID FRAME
TURKEY CREEK
169-46 K1228 (1) JOHNSON CO.
Varies

270

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	1	9
FED. AID NO.		ER-N015(601)			

STATE OF KANSAS
FEDERAL AID PROJECT

CITY OF MERRIAM

Traffic to be carried through construction.
The west lane of southbound traffic may temporarily be closed to facilitate construction.

INDEX OF SHEETS

1. TITLE SHEET
2. PLAN, GENERAL NOTES & SUMMARY OF QUANTITIES
3. WEST CONCRETE RAILING & SIDEWALK DETAILS
4. METAL HANDRAIL DETAILS
5-9. TRAFFIC CONTROL

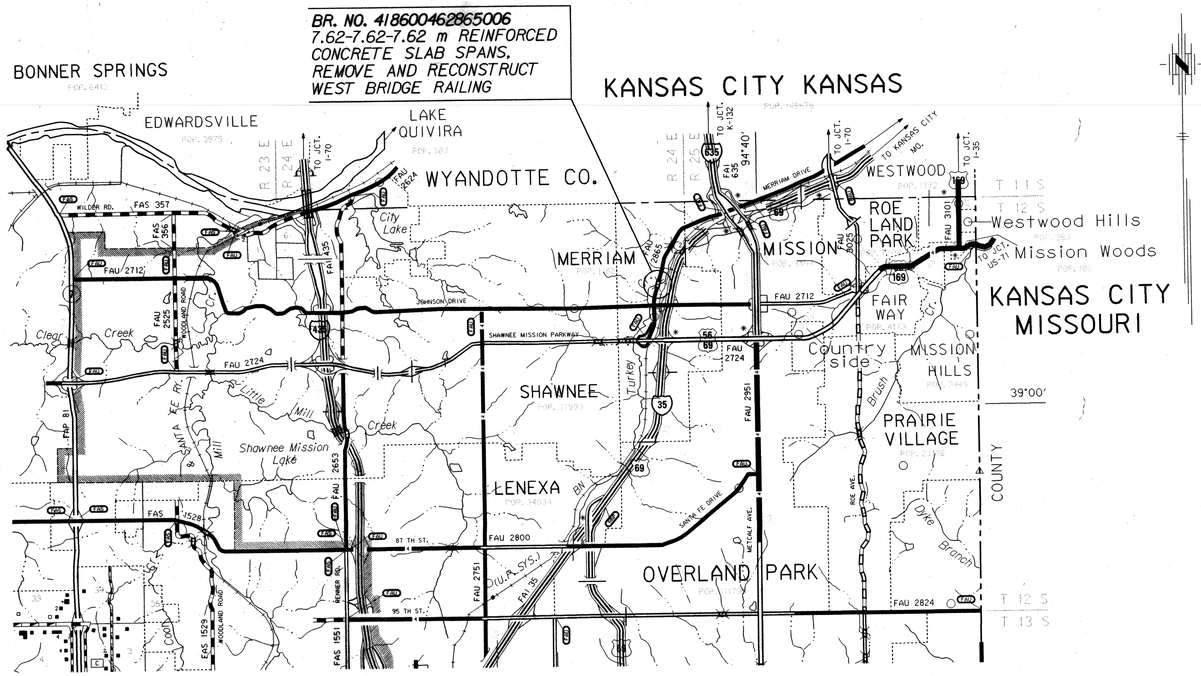
KANSAS DEPARTMENT OF TRANSPORTATION PROJECT NUMBER 46N-0156-01

1992 TRAFFIC CENSUS 10,800 VEHICLES PER DAY

DESIGN SPEED 55 km/h

BRIDGE REPAIR

MERRIAM DRIVE BRIDGE OVER TURKEY CREEK



PREPARED & SUBMITTED BY

HNTB
ARCHITECTS ENGINEERS PLANNERS
OVERLAND PARK, KANSAS

Claude A. Perdue — 2/29/2000
CLAUDE A. PERDUE, P.E. — DATE

CLAUDE A. PERDUE LICENSED 11912 KANSAS PROFESSIONAL ENGINEER

APPROVED BY
CITY OF MERRIAM, KANSAS

CITY OF MERRIAM — 3/2/2000 DATE

CONVENTIONAL SIGNS

COUNTY LINE		CENTER LINE OF PROJECT	4+000 +100
CITY LIMITS		TERRACE	
STATE OR NATIONAL LINE		CULVERTS	
TOWNSHIP, SECTION or GRANT LINE		DROP INLET & STORM SEWER	
PROPERTY LINE		ACCESS CONTROL	
HIGHWAY FENCE		POWER POLE	
EXISTING FENCE		TELEPHONE POLE	
GUARD FENCE		MARSH	
CONSTRUCTION LIMITS		HEDGE	
RIGHT OF WAY LINE		TREES	
TRAVELED WAY		PROFILE ELEVATION	1172.18 / 503
EXISTING PAVEMENT		STREAM or CREEK	
RAILROADS			

GROSS LENGTH OF PROJECT	36.2 m
EXCEPTIONS	NONE
ADDITIONS	NONE
NET LENGTH OF PROJECT	36.2 m
NET LENGTH OF BRIDGES	23.2 m
NET LENGTH OF ROAD	13.0 m

This design specifically prepared for use at the location shown. Use in any other manner exceeds the intended purpose of these drawings.

KDOT METRIC

All dimensions shown without an SI suffix are in millimeters. All elevations shown are in meters.

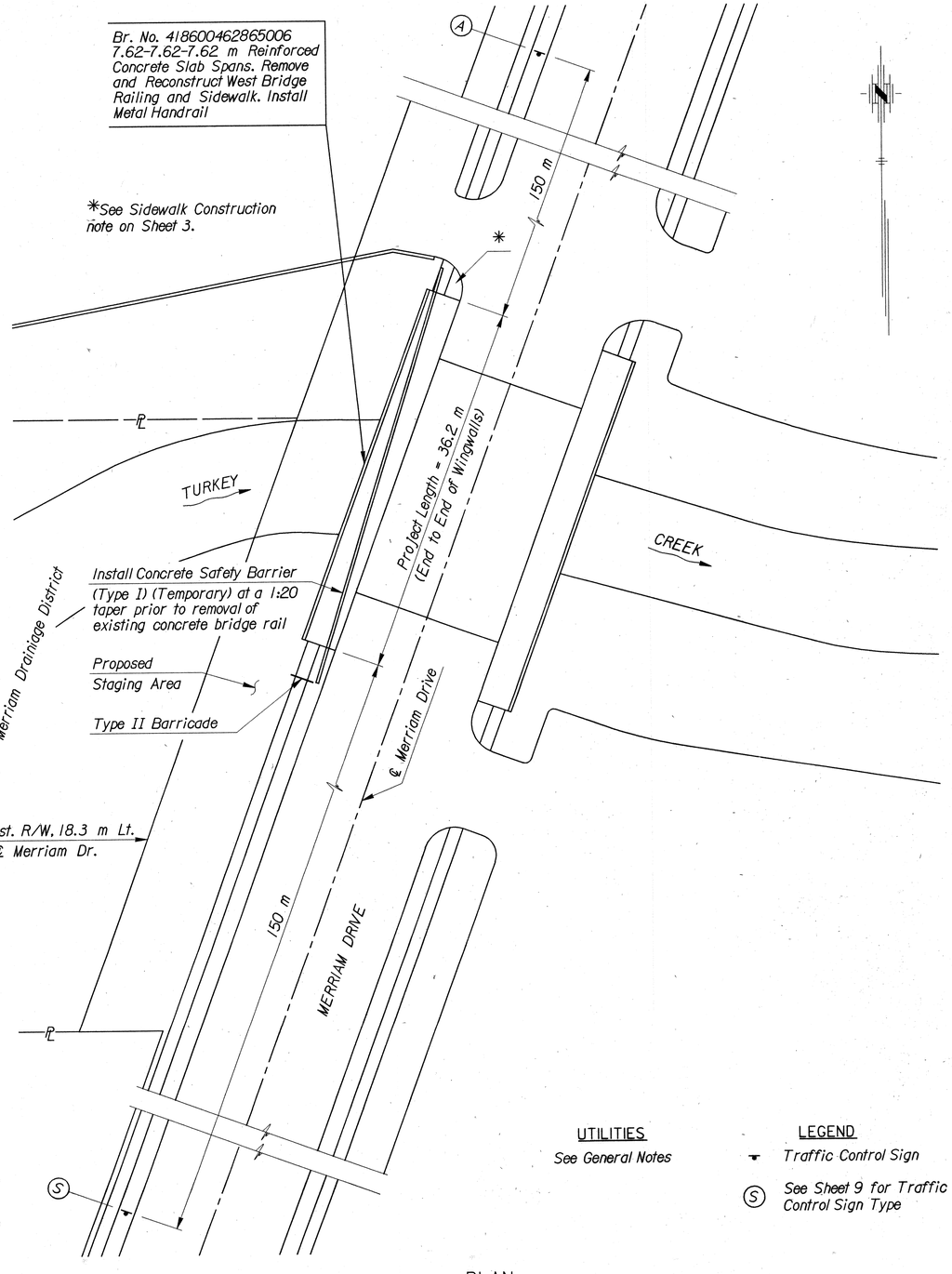
RECOM. FOR APPROVAL-DATE

CHIEF, BUREAU OF LOCAL PROJECTS
KANSAS DEPARTMENT OF TRANSPORTATION

APPROVED - DATE

STATE TRANSPORTATION ENGINEER
KANSAS DEPARTMENT OF TRANSPORTATION

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	2	9



PLAN

UTILITIES
See General Notes

LEGEND
Traffic Control Sign
(S) See Sheet 9 for Traffic Control Sign Type

HNTB
ARCHITECTS ENGINEERS PLANNERS

GENERAL NOTES

CONSTRUCTION SPECIFICATIONS: Kansas Department of Transportation, Standard Specifications for State Road and Bridge Construction, 1990, Metric Version, and Special Provisions.

DESIGN SPECIFICATIONS: AASHTO Standard Specifications for Highway Bridges, 1996 Edition, plus 1997 & 1998 interims.

BROKEN CONCRETE: Waste the broken concrete from the existing bridge on sites provided by the Contractor and approved by the Engineer. All necessary permits or agreements required to dispose of waste material will be the responsibility of the Contractor.

REMOVAL OF EXISTING CONCRETE: Removal of existing concrete is included in the bid item, "Removal of Existing Structures", Lump Sum. All materials removed from the existing structure shall become the property of the Contractor. Remove portions of the existing structure, to limits shown on the plans, to clear the new construction. Remove this material from the site.

CONCRETE: All concrete is bid as Concrete (Grade 30)(AE). Bevel all exposed edges of all concrete with a 20 mm triangular molding, except as otherwise noted on the plans. The Contractor may place the concrete rail continuously from one end of the bridge to the other.

EPOXY GROUTING: This item shall consist of drilling and grouting reinforcing steel into the existing concrete, where required by the Engineer, with an epoxy grout. Locate each hole with the aid of a pachometer to miss the existing reinforcing steel. Drill the holes to the specifications required by the grout manufacturer and in such a manner as not to damage adjacent concrete or bars. After the hole is drilled, remove all loose material by using a wire brush to free the dust from the side of the hole and then vacuuming to remove material and dust. Fill the hole 30% to 50% full of epoxy grout and insert the bar. Then fill the hole to 5 mm from the top of the hole. Follow the manufacturer's directions for mixing, application and curing. The tools, materials, labor and incidentals necessary to complete the work shall be subsidiary to Concrete (Grade 30)(AE).

REINFORCING STEEL: All reinforcing steel dimensions are to the centerline of bars unless otherwise noted. All new reinforcing steel shall conform to the requirements of ASTM A615M-96, Grade 420. All existing reinforcing steel shall remain in place unless otherwise noted or it is sufficiently deteriorated. Any reinforcing steel cut or damaged by the Contractor during concrete removal shall be replaced at no additional cost to the State.

CONSTRUCTION JOINTS: Construction Joints are optional with the Contractor. If used, place the construction Joints at locations approved by the Engineer.

DIMENSIONS: All dimensions shown on the design plans are horizontal dimensions unless otherwise noted.

QUANTITIES: Items not listed separately in the Summary of Quantities are subsidiary to other items in the proposal.

UTILITIES: There are no known utilities within the project limits. Contractor shall verify prior to starting construction.

STAGING AREA: The Contractor shall confine the storage of equipment and materials to an area in the southwest quadrant of the project. This storage area shall be approved by the City and shall be contained within temporary fencing. All areas damaged by the Contractor shall be restored to a condition equal to or better than that existing before damage occured. There shall be no separate payment for this work.

SUMMARY OF ESTIMATED QUANTITES

ITEM	UNITS	QUANTITY
CONCRETE (GRADE 30)(AE)	m^3	6.0
REINFORCING STEEL (GRADE 420)	kg	720
DRILLING AND GROUTING	Each	260
SIDEWALK CONSTRUCTION(100 mm)(AE)(REINF.)	m^2	113
BRIDGE METAL HANDRAIL (560 mm)	m	35.2
REMOVAL OF EXISTING STRUCTURES	Lump Sum	L.S.
TRAFFIC CONTROL	Lump Sum	L.S.
CONCRETE SAFETY BARRIER (TYPE I)(TEMPORARY)	m	42
MOBILIZATION	Lump Sum	L.S.
MOBILIZATION (DBE)	Lump Sum	L.S.
FLAGGER (SET)	Hour	1

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

CITY OF MERRIAM

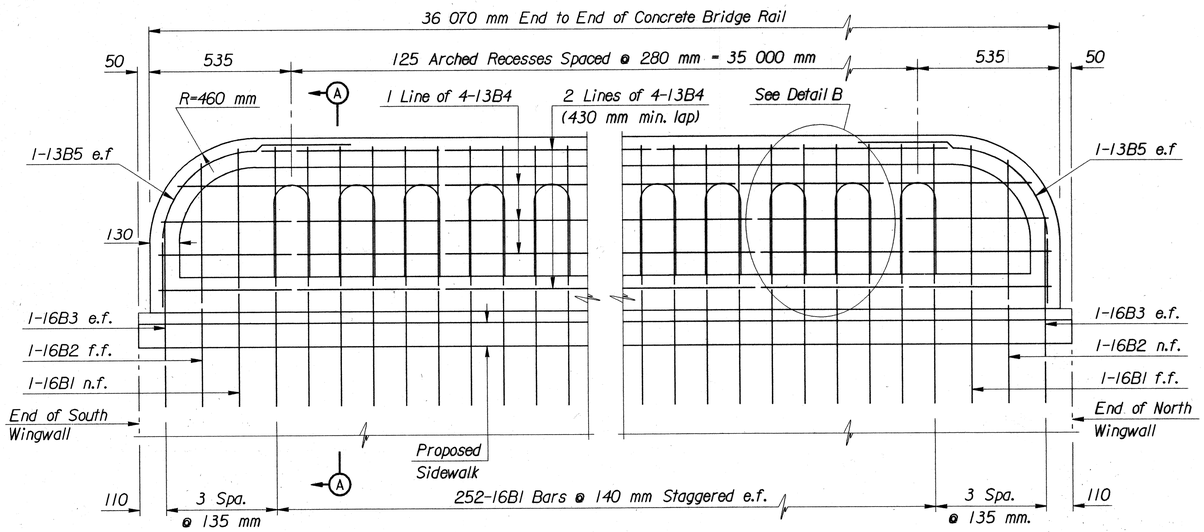
MERRIAM DRIVE OVER TURKEY CREEK
PLAN, GENERAL NOTES & QUANTITIES

SHEET NO. OF	SCALE	APP'D		
DESIGNED	DETAILED RRG	QUANTITIES RRG	TRACED	
DESIGN CK.	DETAIL CK. SAS	QUAN.CK. SAS	TRACE CK.	

Design Filename: h:\30304\details\layout.dgn
Plotted on Tuesday February 29, 2000 10:31 AM Prounle

Plotted on Tuesday February 29, 2000 10:31 AM
Prounle Plot Queues: \\DWPLOT\BRO\xerox
Plot Scale: 0.025400\0.000000 Pen Table: \\ovpw00\bridges\30282\plot\plotpen\Fullsize.pen
Design Filename: h:\30304\details\layout.dgn

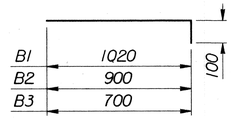
FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	3	9



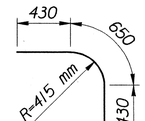
WEST BRIDGE RAIL ELEVATION
(Looking West)

BILL OF REINFORCING							
STRAIGHT BARS				BENT BARS			
MARK	SIZE	NO.	LENGTH	MARK	SIZE	NO.	LENGTH
B4	13	28	9360	B1	16	254	1120
				B2	16	2	1000
				B3	16	4	800
				B5	13	4	1510

BAR BENDING DIAGRAMS

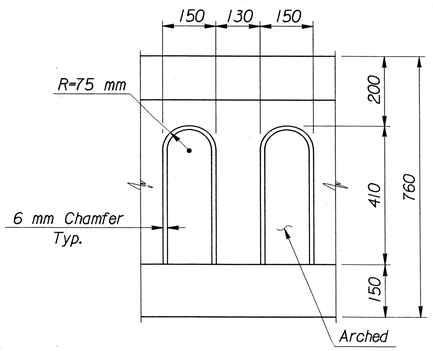


B1, B2, B3

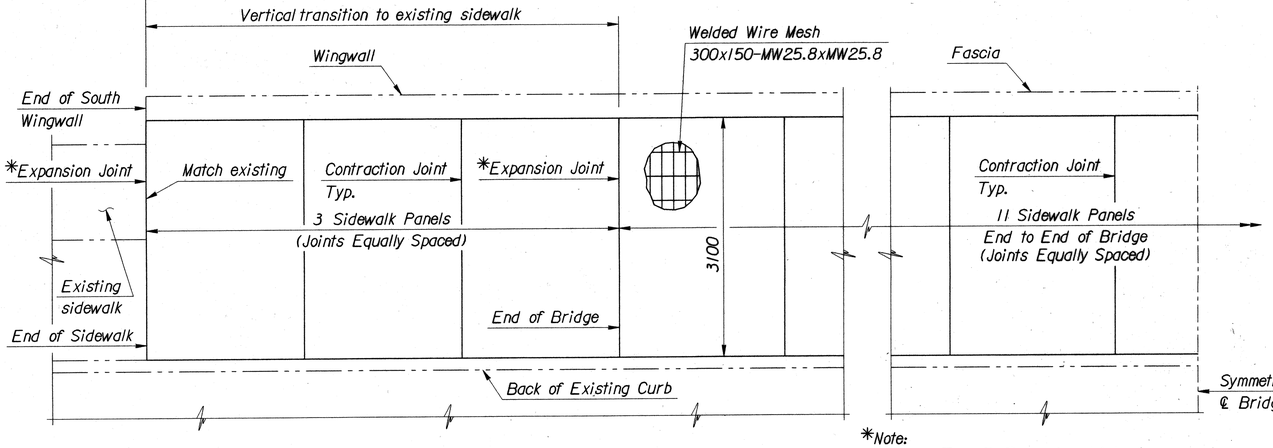


B5

Bar dimensions are out to out.
Radius is to ℄ bar.

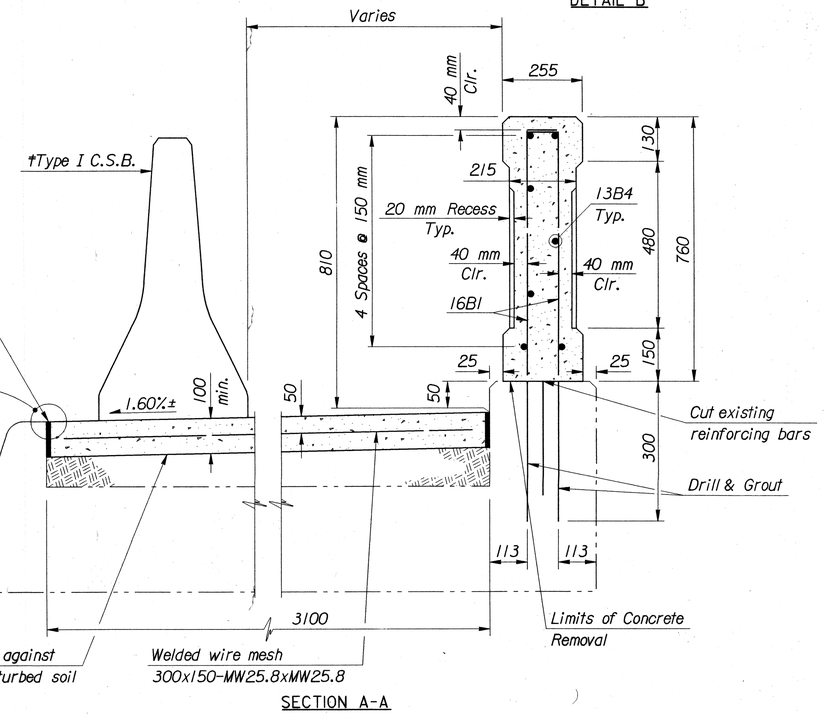


DETAIL B



WEST SIDEWALK PLAN

*Note:
13 mm Type B Expansion Joint
Filler with Silicone Joint Sealant.



SECTION A-A

Notes:

Remove the existing west sidewalk from end to end of wingwall. Saw cut the existing sidewalk at the ends of the wingwalls to neat lines. Repair damage to the existing sidewalk beyond the end of wingwalls at no cost to the project. Removal of existing sidewalk is included in the bid item "Removal of Existing Structures".

Sidewalk Construction will be paid for in m^2. Use Concrete (Grade 30)(AE) in the sidewalk. The excavation, tools, materials, labor and incidentals necessary to complete the work will be subsidiary to Sidewalk Construction. Limits of Sidewalk Construction extend from end to end of existing wingwalls and include a triangular portion between the end of the north wingwall and the parking lot entrance north of the bridge (see Plan on Sheet 2).

†Place Concrete Safety Barrier (Type I)(Temporary) at the location shown prior to the removal of the existing west concrete bridge railing. For the limits of the Type I C.S.B. see Sheet 2.

The length (36 170 mm) from end of the south wingwall to the end of the north wingwall along the west railing is based on field measurements. Verify this dimension by field measurement and submit such verification in writing to the Engineer prior to starting construction.

Bevel all exposed edges with a 20 mm triangular molding except as noted.
n.f. denotes near face.
f.f. denotes far face.
e.f. denotes each face.

HNTB
ARCHITECTS ENGINEERS PLANNERS

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

CITY OF MERRIAM

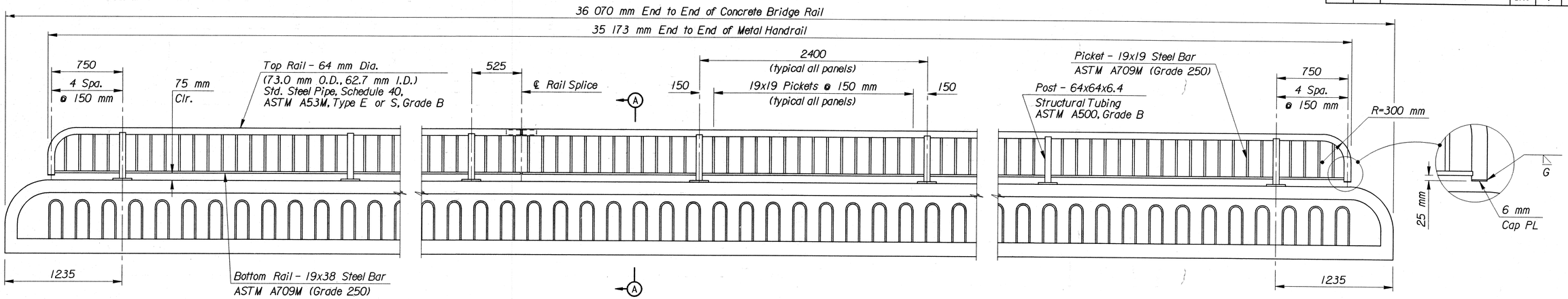
MERRIAM DRIVE OVER TURKEY CREEK
WEST CONCRETE BRIDGE RAILING & SIDEWALK DETAILS

SHEET NO. OF	SCALE		APP'D		
DESIGNED	DETAILED	RRG	QUANTITIES	RRG	TRACED
DESIGN CK.	DETAIL CK.	SAS	QUAN.CK.	SAS	TRACE CK.

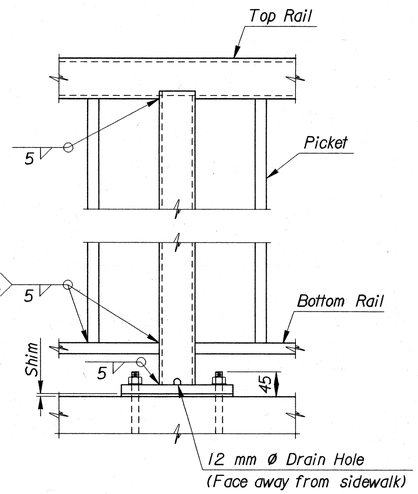
Design Filename: h:\30304\details\barrier.dgn
Plotted on: Tuesday February 29, 2000 10:32 AM Proufle

Plotted on: Tuesday February 29, 2000 10:32 AM
Proufle Plot Queue: \\DNPPLOT1\BRGWIDE1
Plot Scale: 1.000000 x 1.000000 Pen Table: \\cvpw00\bridges\30282\plot\plotpenVTwidth2.pen
Design Filename: h:\30304\details\barrier.dgn

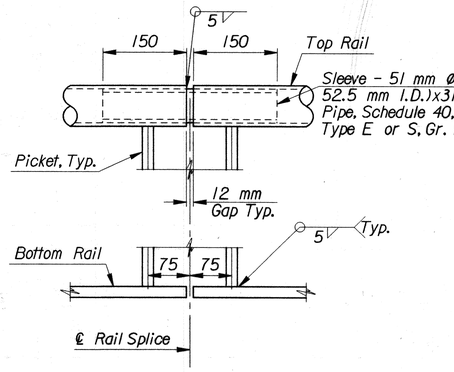
FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	4	9



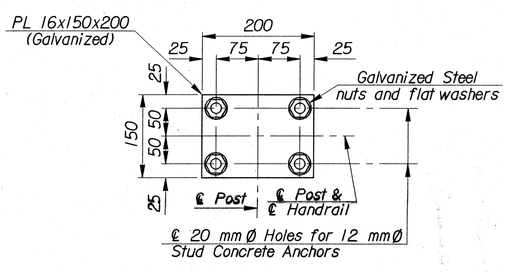
ELEVATION



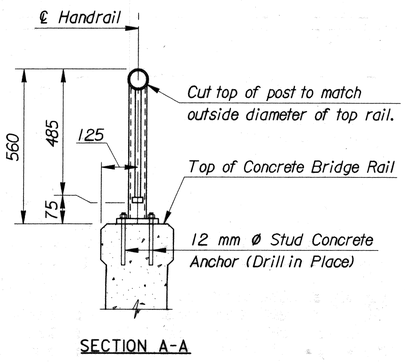
POST DETAIL



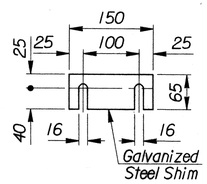
RAIL SPLICE DETAIL



BASE PLATE DETAIL

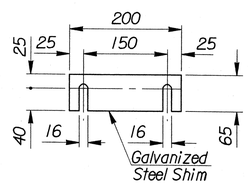


SECTION A-A



SHIM DETAIL

Note:
Provide 1-1.5 mm x 150 mm galvanized steel shim for each post, to be used as required.



SHIM DETAIL

Note:
Provide 1-1.5 mm x 200 mm galvanized steel shim for each post, to be used as required.

METAL HANDRAIL NOTES

Top rails shall be 64 mm Dia. standard steel pipe, Schedule 40 and shall conform to ASTM A53M, Type E or S, Grade B. Posts shall be 64x64x6.4 structural tubing and shall conform to ASTM A500, Grade B. Bottom rails, pickets and base plates shall conform to ASTM A709M (Grade 250).

Galvanize all rails, posts, pickets and base plates after fabrication. Galvanize in accordance with the requirements of the KDOT Standard Specifications.

Construct handrail according to Section 709 of the KDOT Standard Specifications.

Set rails parallel to the top of the concrete rail. Set all posts and pickets vertical in both planes of the handrail. Shims may be used between concrete and base plate of steel post.

Grind smooth all top and bottom rail-to-post welded connections. Field welding is not permitted. Submit shop drawings to the Engineer for approval prior to fabrication of the handrail.

Fabricate handrail in lengths to include a minimum of two and a maximum of three panels. Attach top rail members to at least two posts. Locate center of rail splices 525 mm from centerline of posts unless shown otherwise.

All material, labor, splices, shims, and installation shall be paid for under the bid item "Bridge Metal Handrail (560 mm)". Bid the handrail on a per meter basis measured from end to end of handrail.

Galvanize the stud concrete anchors and provide a minimum pullout strength of 35 kN based on 30 MPa concrete. Length of embedment into concrete shall conform to manufacturer's recommendations.

HNTB
ARCHITECTS ENGINEERS PLANNERS

NO.	DATE	REVISIONS	BY	APP'D
3				
2				
1				

CITY OF MERRIAM

MERRIAM DRIVE OVER TURKEY CREEK

METAL HANDRAIL DETAILS

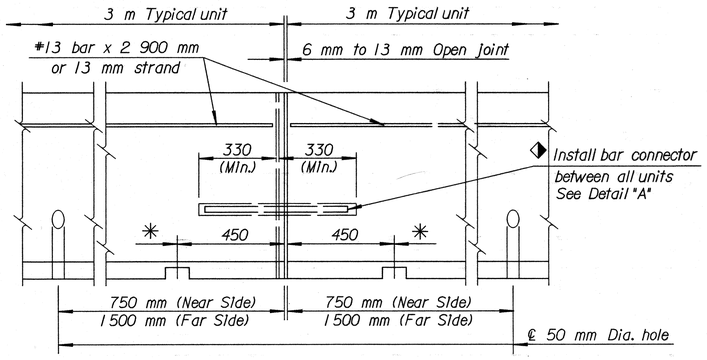
SHEET NO. OF	SCALE		APP'D	
DESIGNED	DETAILED	RRG	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	SAS	QUAN.CK.	TRACE CK.

Design Filename: h:\30304\details\handrail.dgn
Plotted on: Tuesday February 29, 2000 10:33 AM PYounie

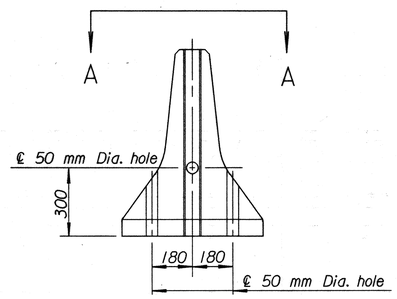
Plotted on: Tuesday February 29, 2000 10:33 AM
PYounie Plot Queue: \\WPPLOT\BROXerox
Plot Scale: 0.025400x0.000000 Pen Table: \\wpw00\bridges\30282\plot\plotpen\Fullsize.pen
Design Filename: h:\30304\details\handrail.dgn

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	5	9

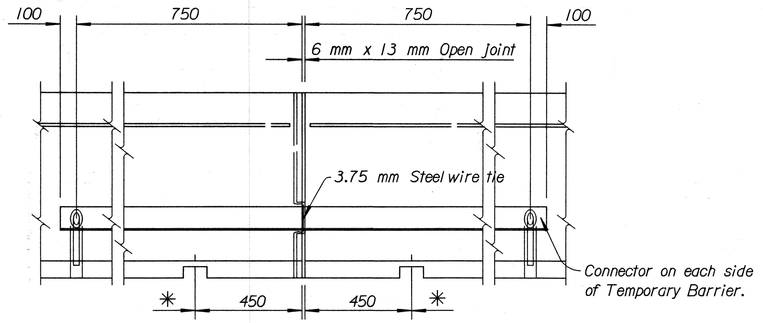
✳ ℄ 50 mm x 100 mm (Nominal) lifting slots at each end of section.



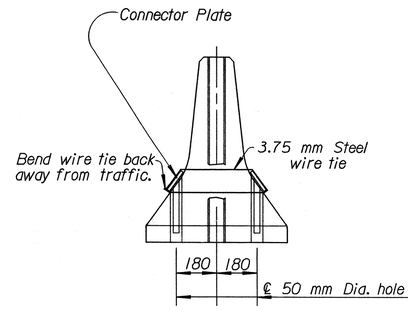
SIDE ELEVATION
(With dowel bar connector)



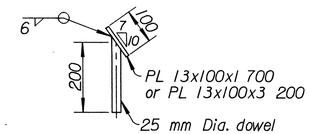
END ELEVATION
(With dowel bar connector)



SIDE ELEVATION
(With plate connector)

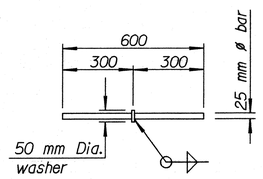


END ELEVATION
(With plate connector)

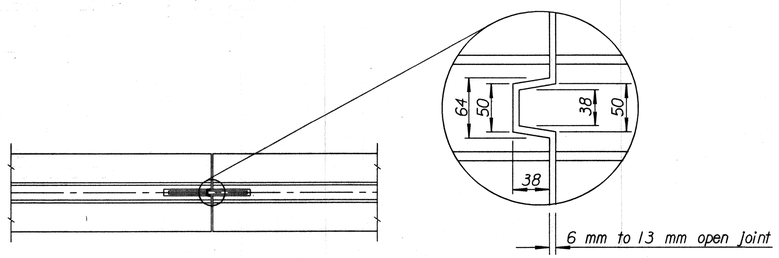


SECTION THRU CONNECTOR

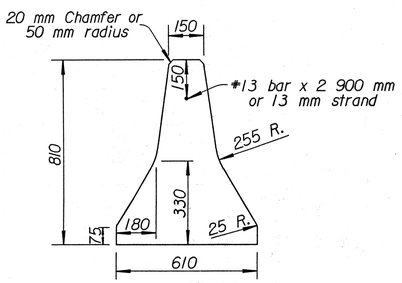
◆ For the convenience of the Contractor in replacing broken units, plate connectors as shown at the right may be used. At the Contractor's option, plate connectors may be used every 15 m in lieu of bar connectors. The Contractor shall use plates on both sides. When used, the Contractor may use either a 3 200 mm plate on the one side of the installed barrier unit or drill appropriate holes in the far side for dowels on the 1700 mm plate connector. This work is subsidiary to installing the barrier.



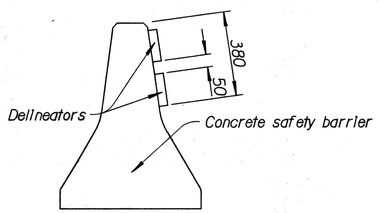
DETAIL "A"
(Showing 25 mm Φ bar & washer)



VIEW A-A



TYPICAL END SECTION



DELINEATOR DETAILS

MATERIAL: Use Concrete Grade 25 (AE) throughout. All structural steel shall be merchant grade. Reinforcing steel shall be Grade 300 or Grade 420 billet steel and shall meet the requirements of the KDOT Standard Specifications.

SECTION: The section furnished must generally comply with dimensions shown. Requests for minor variations in section geometry and attachments may be submitted to the Engineer for approval.

LIFTING SLOTS: Lifting slots shall be constructed where specified on the plans to facilitate the drainage of water after installation on the roadway. Approximate mass of one unit equals 1800 kg.

TEMPORARY CONCRETE SAFETY BARRIER: Furnishing and placing of all materials when required and all labor and equipment required to position the temporary barrier shall be included in the Contract unit price bid for "Concrete Safety Barrier (Type 1) (Temporary)". Any relocation of the barrier required for the project shall be paid in accordance with the KDOT Special Provisions under the bid item "Concrete Safety Barrier (Type 1) (Temp.- Relocate)". Unless otherwise noted on the Plans the Temporary Safety Barrier shall become the property of the Contractor and shall be removed from the site upon acceptance of the completed project.

SURFACE PREPARATION: Temporary concrete barriers shall be placed on either a concrete or bituminous surface. All loose dirt and sand shall be removed from the roadway surface just prior to placement of the safety barrier. Temporary barriers shall not be placed directly on soil or gravel.

SIGNING: For sign spacing, details of other traffic control devices and reference notes, see other sheets.

DELINEATION: Delineators shall be spaced on 30 m centers, except through curves having 580 m radius or less where they shall be spaced on 6 m centers.

All delineators shall be one of the following: Stimsonite 965 Y or W, Astro-Optics Model JD-1 or 2, Duraflex Model Flexx 2020, or an approved equivalent to those listed.

The delineation shall be mounted on the side of the Temporary Concrete Safety Barrier with two delineators at each location. Each delineator shall have a minimum height-to-width ratio of 1.75, and a minimum reflective surface area of 4 500 mm². The delineators shall be affixed to the Temporary Concrete Safety Barrier as recommended by the manufacturer.

Delineators shall be attached to bridge rail or other structures in construction zones when roadway is narrowed and traffic is adjacent to the structure. The method and location of placement shall be similar to temporary barrier delineation. All work and materials required for this installation shall be subsidiary to the bid item "Concrete Safety Barrier (Type ◆) (Temporary)".

When traffic flow is in one direction, the delineators shall be yellow when used on the left, and white when used on the right. When traffic flow is in both directions, delineators shall be placed back-to-back, and shall correspond to the color of the edge line.

The work and materials required for the installation of delineators shall be subsidiary to the Temporary Safety Barrier bid item.

TEMPORARY BARRIERS: Barriers constructed to the details of this drawing shall not be used in permanent installations.

◆ Type shall be either Type I or Type IV.

NO.	DATE	REVISIONS	BY	APP'D.
4	11-12-98	Revised concrete grade	R.J.S.	J.O.B.
3	4-10-98	Added Relocation Bid Item	R.J.S.	J.O.B.
2	1-29-97	Revised reinf. steel grade	R.J.S.	J.O.B.

KANSAS DEPARTMENT OF TRANSPORTATION

TEMPORARY CONCRETE SAFETY BARRIER TYPE I

RD640 SI

FHWA APPROVAL	2-8-99	APP'D. James O. Brewer	
DESIGNED	DETAILED	QUANTITIES	TRACED Bowser
DESIGN CK.	DETAIL CK.	QUAN.CK.	TRACE CK. Seitz

Design Filename: h:\30304\standards\rd640sl.dgn

Plotted on: Tuesday February 29, 2000 10:34 AM

PYcuntle

Plot Scale: 2.540000\1.000000 Pen Tables: \\corpw00\bridges\30282\plot\plotpen\kdot-full.pen

Plot Queue: \\DWPPLOT\BROXerox

Design Filename: h:\30304\standards\rd640sl.dgn

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	6	9

GENERAL NOTES

1. CHANNELIZING DEVICES: "DEVICES" INCLUDES BUT IS NOT LIMITED TO BARRICADES, BARRIERS, CONES, DRUMS AND VERTICAL PANELS.

   (A) THE MAXIMUM SPACING BETWEEN DEVICES IN THE TAPER SHOULD NOT EXCEED A DISTANCE IN METERS EQUAL TO 0.3 TIMES THE SPEED LIMIT (IN MPH) PRIOR TO WORK.

   (B) THE SPACING BETWEEN DEVICES IN THE WORK ZONE SHOULD NOT EXCEED A DISTANCE IN METERS OF 0.6 TIMES THE POSTED SPEED (MPH).

   (C) WHERE EXISTING CONDITIONS WARRANT, THE ENGINEER MAY REQUIRE A DECREASE IN THE SPACING STIPULATED ABOVE. WHEN TRAFFIC CONTROL IS BID LUMP SUM, ADDITIONAL DEVICES AND WARNING LIGHTS WILL BE PAID FOR AS EXTRA WORK.

   (D) TRAFFIC CONES MAY BE UTILIZED AS CHANNELIZING DEVICES FOR DAY-TIME TRAFFIC CONTROL OPERATIONS. THE ENGINEER MAY REQUIRE THAT CONES BE SUPPLEMENTED BY OTHER TRAFFIC CONTROL DEVICES IN CERTAIN SITUATIONS SUCH AS OPEN TRENCHES.

2. COVERING OF SIGNS NOT IN USE:

   WHEN NO WORK IS IN PROGRESS NOR IS ANY EXPECTED TO BE FOR AN EXTENDED PERIOD OF TIME, AND THE ROADWAY IS UNRESTRICTED TO THE TRAVELING PUBLIC, TRAFFIC CONTROL SIGNS SHALL BE REMOVED OR COMPLETELY COVERED WITH ADEQUATE OPAQUE WATERPROOF MATERIAL. TAPE SHALL NOT BE APPLIED TO THE FACE OF A SIGN.

3. CLEAR ZONE:

   WHENEVER PRACTICAL, ALL CONSTRUCTION EQUIPMENT, MATERIALS, AND DEBRIS SHALL BE STORED OUT OF THE CLEAR ZONE. WHERE THIS CANNOT BE ACHIEVED, THE CONTRACTOR SHALL PLACE APPROPRIATE SIGNS AND/OR BARRICADES AS DESIGNATED BY THE ENGINEER. TRAFFIC CONTROL DEVICES NEEDED FOR THIS CONDITION SHALL BE CONSIDERED SUBSIDIARY TO OTHER BID ITEMS.

4. MAINTENANCE:

   THE CONTRACTOR SHALL MAINTAIN ALL SIGNS AND DEVICES IN THEIR PROPER POSITION AND CLEAN AND/OR REPLACE ANY DAMAGED OR ILLEGIBLE SIGN OR DEVICE AS DIRECTED BY THE ENGINEER.

5. TAPER FORMULAS:

   $L = WS$ FOR SPEEDS OF 45 MPH OR MORE

   $L = WS^2/60$ FOR SPEEDS OF 40 MPH OR LESS

   WHERE: L = MINIMUM LENGTH OF TAPER IN METERS
   S = NUMERICAL VALUE OF POSTED SPEED PRIOR TO WORK IN MPH
   W = WIDTH OF OFFSET IN METERS

6. PAVEMENT MARKINGS:

   (A) TYPE I TAPE IS TO BE APPLIED TO A FINAL SURFACE AS A TEMPORARY MARKING WHEN THE PATTERN OF THE PERMANENT MARKINGS WILL BE DIFFERENT THAN THE TEMPORARY MARKING.

   TYPE II TAPE IS TO BE APPLIED TO A TEMPORARY SURFACE THAT IS TO BE REMOVED OR COVERED BY FUTURE CONSTRUCTION. TYPE II TAPE IS ALSO TO BE APPLIED TO A FINAL SURFACE WHERE IT IS KNOWN THERE WILL BE PERMANENT MARKINGS PLACED IN THE SAME PATTERN AS THE TEMPORARY TYPE II TAPE.

   (B) ALL CONFLICTING PAVEMENT MARKINGS SHALL BE REMOVED AND ALL TRANSITION TAPERS, CROSSOVERS, AND EDGE LINES ALONG CHANNELIZING DEVICES SHALL BE MARKED WITH SOLID 100 mm WIDE PAVEMENT MARKING TAPE WHEN THE WORK WILL OCCUPY A LOCATION MORE THAN THREE DAYS.

7. TEMPORARY AND POST MOUNTED SIGNS:

   TRAFFIC CONTROL PLANS THAT ARE ANTICIPATED TO REMAIN IN PLACE DURING DAYLIGHT HOURS ONLY ARE CONSIDERED "TEMPORARY". TEMPORARY SIGNS SHALL BE MOUNTED AT A MINIMUM HEIGHT OF 300 mm. TRAFFIC CONTROL PLANS OF LONGER DURATION WILL REQUIRE THAT ALL SIGNS BE POST MOUNTED AT A MINIMUM HEIGHT OF 2.1 m.

8. EXISTING SIGNS:

   IF EXISTING SIGNS THAT ARE TO REMAIN (OR THAT ARE TO BE REMOVED AND RESET) INTERFERE WITH CONSTRUCTION WORK, THE CONTRACTOR SHALL REMOVE, STORE, AND RESET THE SIGNS. THIS SHALL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS.

9. CONFLICTING SIGNS:

   ALL PERMANENT AND TEMPORARY SIGNING WHICH IS IN CONFLICT WITH THE TRAFFIC CONTROL PLAN SHALL BE REMOVED OR COVERED.

10. MINIMUM LANE WIDTHS:

    LANE WIDTHS SHALL BE A MINIMUM OF 3.4 m (MEASURED BETWEEN CENTERLINES OF PAVEMENT MARKINGS), OR AS SHOWN ON THE PLANS, OR AS DIRECTED BY THE ENGINEER. A LANE WIDTH LESS THAN 3.4 m MAY REQUIRE RESTRICTED ROADWAY WIDTH SIGNING.

11. FLAGGER:

    A MINIMUM OF ONE FLAGGER SHALL BE STATIONED WITHIN EACH INTERSTATE WORK AREA WHERE WORK IS ADJACENT TO TRAFFIC.

12. BARRICADES:

    TYPE III BARRICADES PLACED ACROSS A ROADWAY SHALL BE SUITABLY DISTRIBUTED ACROSS THE ROADWAY TO EFFECTIVELY CLOSE THE ROADWAY. WHERE PROVISION IS MADE FOR ACCESS BY AUTHORIZED VEHICLES, THE BARRICADES SHALL CLOSE THE ROAD AT THE END OF THE WORK DAY. WHEN ACCESS MUST BE ALLOWED FOR LOCAL TRAFFIC, THE TYPE III BARRICADES SHOULD BE STAGGERED.

SIGN SPACING (IN METERS):

	A	B	C
URBAN (40 MPH OR LOWER)	60	60	60
URBAN (45 MPH OR HIGHER)	115	115	115
RURAL	150	150	150
EXPRESSWAY/FREEWAY	300	525	800

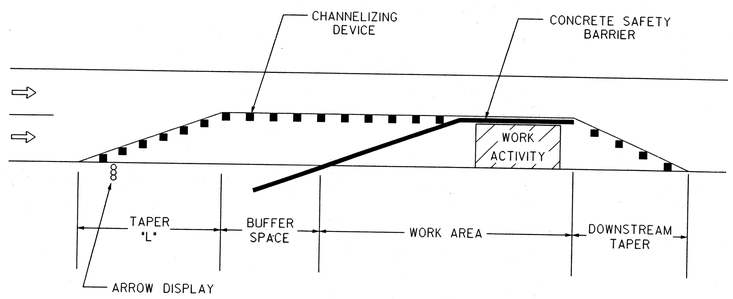
IN NO CASE SHALL THE SPACING BETWEEN SIGNS BE LESS THAN 30 m.

BUFFER SPACE:

SPEED (MPH)	20	25	30	35	40	45	50	55	60	65	70
LENGTH (m)	10	15	25	35	50	65	85	100	125	150	175

NEITHER WORK ACTIVITY NOR STORAGE OF EQUIPMENT, VEHICLES, OR MATERIAL SHOULD OCCUR IN THE BUFFER SPACE. WHEN A PROTECTION VEHICLE IS PLACED IN ADVANCE OF THE WORK SPACE, ONLY THE SPACE UPSTREAM OF THE VEHICLE CONSTITUTES THE BUFFER SPACE.

IF TEMPORARY CONCRETE SAFETY BARRIER IS USED TO SEPARATE APPROACHING TRAFFIC FROM THE WORK ACTIVITY, THE BARRIER SHALL BE CONSIDERED PART OF THE WORK AREA. A FULL LANE WIDTH SHOULD BE AVAILABLE THROUGHOUT THE LENGTH OF THE BUFFER SPACE. FOR EXAMPLE:



NO.	DATE	REVISION	BY	APP'D
3				
2	6-21-96	ADDED TO BARRICADE NOTES & BUFFER CHART	WPR	NAB
1	1-4-95	ADDED FLAGGER NOTES	RM	NAB

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL
GENERAL NOTES

TE735SI 7/1/94

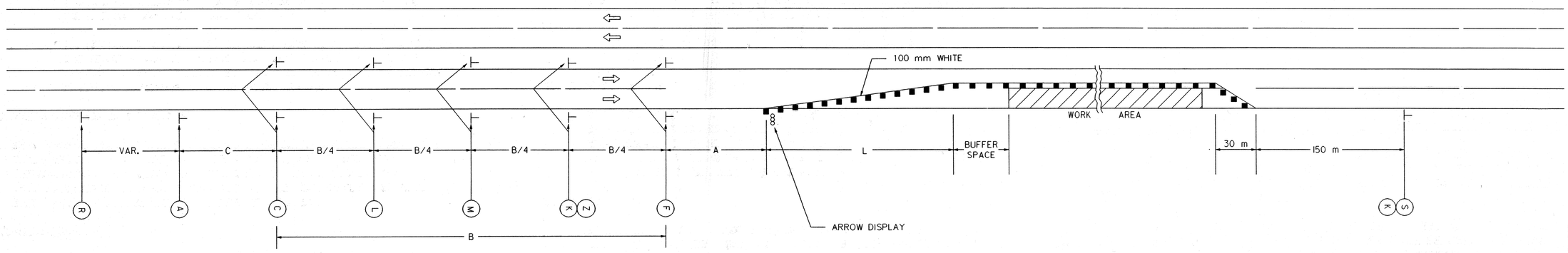
FHWA APPROVAL 8-1-96		APP'D	Nelda A. Buckley
DESIGNED	DETAILED	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Design Filename: h:\30304\standards\te735si.dgn
Plotted on: Tuesday, February 29, 2000 10:34 AM Pycuna

Plotted on: Tuesday February 29, 2000 10:34 AM
Pycuna Plot Queue: \DWPPLOT\BROKER
Plot Scale: 0.025400\0.000000 Pen Table: \\ovpw00\bridges\3028z\plot\plotpen\kdot-full.pen
Design Filename: h:\30304\standards\te735si.dgn

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	7	9

REFER TO STD. TE755SI FOR IDENTIFICATION OF SIGNS FOR HANDLING TRAFFIC THROUGH CONSTRUCTION.
REFER TO STD. TE735SI FOR DIMENSIONS.
■= CHANNELIZING DEVICES



LEFT-SIDE SIGNS SHALL BE OMITTED FOR A FOUR-LANE UNDIVIDED HIGHWAY.

A TRAILER OR TRUCK WITH AN ARROW DISPLAY MAY BE USED DURING DAYTIME IN PLACE OF THE ARROW DISPLAY. THE TRUCK SHOULD BE LOCATED BETWEEN THE BUFFER SPACE AND THE WORK AREA.

NO.	DATE	REVISION	BY	APP'D
3				
2				
1	10/10/96	ADDED M4-20 AND END OF WORK ZONE R2-1 SIGNS	WPR	NAB

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL TRAFFIC CONTROL FOUR-LANE HIGHWAY ONE LANE CLOSED

TE744SI 7/1/94

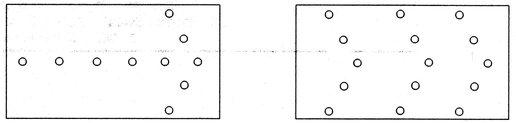
FHWA APPROVAL 12-12-96		APP'D	Nelda A. Buckley
DESIGNED	DETAILED	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Design Filename: h:\30304\standards\te744sl.dgn
Plotted on Tuesday, February 29, 2000 10:34 AM PYounie

Plotted on Tuesday February 29, 2000 10:34 AM
PYounie Plot Queue: \\WPPLOT\BRGXerox
Plot Scale: 0.025400\.000000 Pen Table: \\orpw00\bridges\30282\plot\plotpen\kdot+fullpen
Design Filename: h:\30304\standards\te744sl.dgn

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	8	9

ARROW DISPLAY

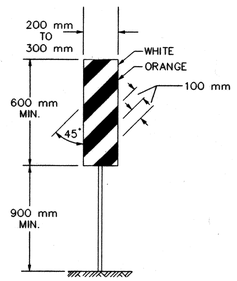


FLASHING ARROW

SEQUENTIAL CHEVRON

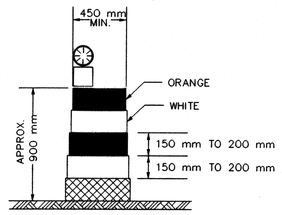
ARROW DISPLAY ELEMENTS SHALL BE CAPABLE OF A MINIMUM 50 PERCENT DIMMING FROM THEIR FULL-RATED LAMP VOLTAGE. FULL LAMP VOLTAGE SHOULD BE USED FOR DAY AND DIMMED MODE SHALL BE USED FOR NIGHT.

VERTICAL PANEL

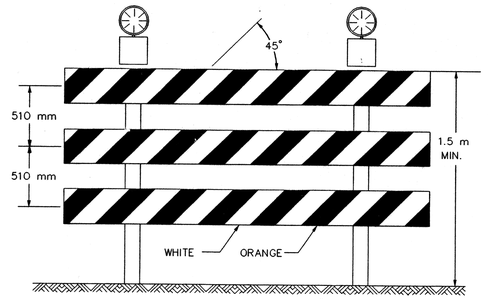


THE ENTIRE AREA OF VERTICAL PANELS SHALL BE FULLY REFLECTORIZED WITH TYPE II HIGH PERFORMANCE RETROREFLECTIVE SHEETING. VERTICAL PANELS USED ON ROADWAYS WITH A SPEED LIMIT OF 45 MPH OR GREATER SHALL HAVE A MINIMUM OF 0.175 SQ m OF RETROREFLECTIVE AREA FACING TRAFFIC.

NON-METALLIC REFLECTORIZED DRUM WITH LIGHT

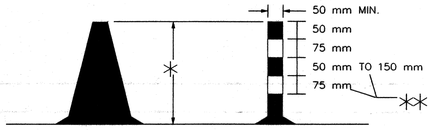


DRUMS SHALL BE REFLECTORIZED FULL CIRCUMFERENCE WITH AT LEAST TWO ORANGE AND TWO WHITE 150 TO 200 mm WIDE STRIPES OF TYPE II HIGH PERFORMANCE RETROREFLECTIVE SHEETING. ADDITIONAL ORANGE AND WHITE STRIPES MAY BE NON-REFLECTIVE. IF THERE ARE NON-REFLECTIVE SPACES BETWEEN ADJACENT STRIPES, THEY SHALL BE NO MORE THAN 50 mm WIDE.



TYPE III BARRICADE WITH LIGHTS

A MINIMUM OF TWO TYPE A LIGHTS SHALL BE USED AT EACH LOCATION WHERE A TYPE III BARRICADE OR BARRICADES ARE USED. A LIGHT SHALL BE MOUNTED ON THE OUTSIDE CORNER OF THE END BARRICADES WHEN MORE THAN ONE IS USED.



ORANGE CONES TUBULAR MARKERS

* CONES AND TUBULAR MARKERS SHALL HAVE A MINIMUM HEIGHT OF 710 mm WHEN USED ON 45 MPH OR GREATER ROADWAYS. THE MINIMUM HEIGHT MAY BE REDUCED TO 450 mm ON ROADWAYS OF 40 MPH AND LOWER.

** FOR 710 mm TUBULAR MARKERS

TRAFFIC CONES MAY BE USED FOR DAYTIME OPERATIONS ONLY. IN CERTAIN SITUATIONS, SUCH AS OPEN TRENCHES, THE ENGINEER MAY REQUIRE THAT CONES BE SUPPLEMENTED BY OTHER TRAFFIC CONTROL DEVICES.

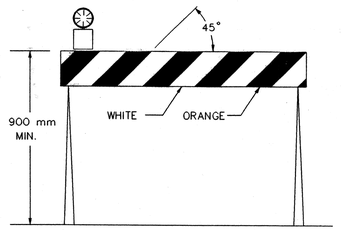
WARNING LIGHTS

WARNING LIGHTS SHALL COMPLY WITH THE CURRENT ITE PURCHASE SPECIFICATIONS FOR FLASHING AND STEADY BURN WARNING LIGHTS. THE LENS SHALL BE A MINIMUM OF 175 mm IN DIAMETER AND BE MOUNTED AT A MINIMUM HEIGHT OF 750 mm.

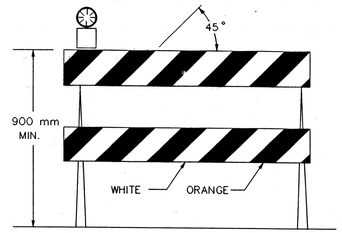
WARNING LIGHTS USED ON INDIVIDUAL CHANNELIZING DEVICES SHALL BE TYPE A FLASHING. THOSE USED ON DEVICES PLACED IN SERIES SHALL BE TYPE C STEADY BURN, WITH THE EXCEPTION THAT WARNING LIGHTS ARE NOT TO BE USED ON REFLECTORIZED DRUMS PLACED IN SERIES.

TYPE B HIGH INTENSITY LIGHTS ARE TO BE USED ON CONSTRUCTION WARNING SIGNS AS DIRECTED BY THE ENGINEER.

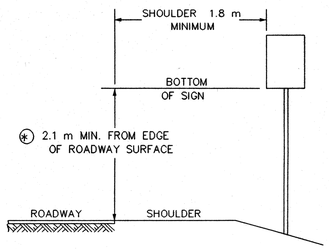
TYPE A LOW INTENSITY FLASHING WARNING LIGHTS AND TYPE C STEADY BURN WARNING LIGHTS SHALL BE MAINTAINED SO AS TO BE CAPABLE OF BEING VISIBLE ON A CLEAR NIGHT FROM A DISTANCE OF 1000 m. TYPE B HIGH INTENSITY FLASHING WARNING LIGHTS SHALL BE MAINTAINED SO AS TO BE CAPABLE OF BEING VISIBLE ON A SUNNY DAY FROM A DISTANCE OF 300 m WHEN VIEWED WITHOUT THE SUN DIRECTLY ON OR BEHIND THE DEVICE.



TYPE I BARRICADE WITH LIGHT



TYPE II BARRICADE WITH LIGHT



SUPPORTS USED FOR MOUNTING SIGNS OR DEVICES FOR TEMPORARY CONDITIONS SHALL BE CONSTRUCTED TO YIELD UPON IMPACT. ADDITIONAL SUPPORTS MAY BE PLACED ON THE BACK SIDE OF SIGNS IN THE FORM OF BRACING FOR RESISTING WIND CURRENTS. GUY WIRES AND TIE-DOWNS WILL NOT BE ALLOWED.

SIGNS MOUNTED ON TYPE III BARRICADES SHOULD NOT COVER MORE THAN 50 PERCENT OF THE TOP TWO RAILS OR 33 PERCENT OF THE TOTAL AREA OF THE THREE RAILS.

* THE HEIGHT TO THE BOTTOM OF A SECONDARY SIGN MOUNTED BELOW ANOTHER SIGN MAY BE 2.0 m.

NOTE:

ALL TRAFFIC CONTROL DEVICES, THEIR INSTALLATION AND MAINTENANCE SHALL COMPLY WITH THE REQUIREMENTS OF THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES FOR STREETS AND HIGHWAYS. WHENEVER THIS STANDARD CONFLICTS WITH THE MANUAL, THIS STANDARD SHALL GOVERN.

CHANNELIZING DEVICES SHALL BE PLACED FOR OPTIMUM VISIBILITY, NORMALLY AT RIGHT ANGLES TO THE TRAFFIC FLOW.

CHANNELIZING DEVICES PLACED ALONG SHOULDER EDGES OR IN DROPOFFS SHALL HAVE A MINIMUM OF 600 mm FROM THE TOP OF THE CHANNELIZING DEVICE TO THE TOP OF THE PAVEMENT.

WHERE PERMANENT OBJECT MARKERS ARE NOT ALREADY IN PLACE, ORANGE AND WHITE VERTICAL PANELS SHALL BE ERECTED TO DELINEATE EXPOSED BRIDGE ENDS.

THE FIRST MODULE OF EACH INERTIAL BARRIER SYSTEM SHALL HAVE A MINIMUM OF 0.175 SQ m OF TYPE II HIGH PERFORMANCE ORANGE RETROREFLECTIVE SHEETING FACING TRAFFIC. EITHER A VERTICALLY RECTANGULAR OR DIAMOND SHAPE MAY BE USED.

BARRICADES

	TYPE I	TYPE II	TYPE III
WIDTH OF RAIL	200 mm MIN-300 mm MAX	200 mm MIN-300 mm MAX	200 mm MIN-300 mm MAX
LENGTH OF RAIL	600 mm MIN	600 mm MIN	1200 mm MIN
WIDTH OF STRIPES	150 mm	150 mm	150 mm
HEIGHT	900 mm MIN	900 mm MIN	1500 mm MIN
NO. OF REFL. RAIL FACES	2 (ONE EACH DIRECTION)	4 (TWO EACH DIRECTION)	*

* 3 IF FACING TRAFFIC IN ONE DIRECTION, 6 IF BOTH DIRECTIONS.

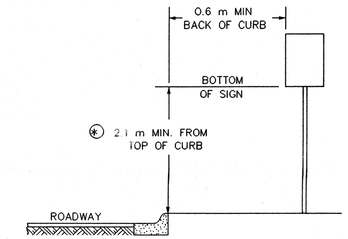
FOR WOODEN BARRICADES, NOMINAL LUMBER DIMENSIONS WILL BE SATISFACTORY.

FOR RAILS LESS THAN 900 mm LONG, 100 mm WIDE STRIPES MAY BE USED.

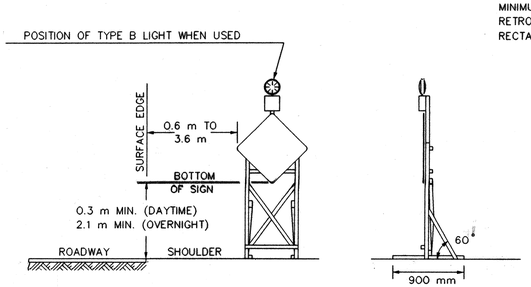
THE ENTIRE AREA OF BARRICADE RAILS SHALL BE FULLY REFLECTORIZED WITH TYPE II HIGH PERFORMANCE RETROREFLECTIVE SHEETING.

BARRICADES USED ON ROADWAYS WITH SPEED LIMITS OF 45 MPH OR MORE SHALL HAVE A MINIMUM OF 0.175 SQ m OF RETROREFLECTIVE AREA FACING TRAFFIC.

THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION, AND TOWARD THE CENTER OF THE ROADWAY AT ROAD CLOSURES.



HEIGHT AND LATERAL DIMENSIONS FOR POST MOUNTED SIGNS (ALL SIGNS LEFT IN PLACE OVERNIGHT OR LONGER)



HEIGHT AND LATERAL DIMENSIONS FOR SIGNS MOUNTED ON SKIDS OR OTHER MOVABLE MOUNTS

NO.	DATE	REVISION	BY	APP'D
3				
2				
1	7-8-96	ADDED TO SKID MOUNTED SIGN DETAIL	WPR	NAB

KANSAS DEPARTMENT OF TRANSPORTATION

DESIGN DETAILS FOR BARRICADES, CHANNELIZING DEVICES AND SIGN MOUNTING LOCATIONS

TE752SI 7/1/94

FHWA APPROVAL	8-1-96		APP'D	Nelda A. Buckley	
DESIGNED	PR	DETAILED PR	QUANTITIES		TRACED
DESIGN CK.	AFT	DETAIL CK. AFT	QUAN. CK.		TRACE CK.

Design Filename: h:\30304\standards\te752sl.dgn

Plotted on: Tuesday, February 29, 2000 10:35 AM PYounle

Plotted on Tuesday February 29, 2000 10:35 AM
PYounle Plot Queue: \\WPPLOT\BROKer-ox
Plot Scale: 0.025400L000000 Plot Table: \\wpw00\bridges\30282\plot\plotpen\kdot+fullpen
Design Filename: h:\30304\standards\te752sl.dgn

FHWA REGION NO.	STATE	PROJECT NO.	YEAR	SHEET NO.	TOTAL SHEETS
7	KANSAS	46N-0156-01	2000	9	9

(A)



W20-1
1200 mm x 1200 mm

(B)



W20-4
1200 mm x 1200 mm

(C)



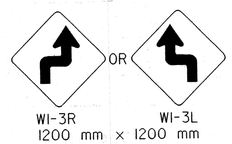
W20-5
1200 mm x 1200 mm

(D)



W20-7a
1200 mm x 1200 mm

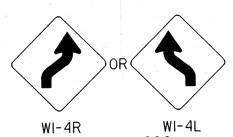
(E)



WI-3R
1200 mm x 1200 mm

WI-3L
1200 mm x 1200 mm

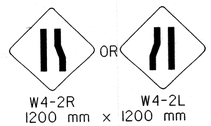
(E)



WI-4R
1200 mm x 1200 mm

WI-4L
1200 mm x 1200 mm

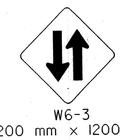
(F)



W4-2R
1200 mm x 1200 mm

W4-2L
1200 mm x 1200 mm

(G)



W6-3
1200 mm x 1200 mm

(H)



WI-6
1200 mm x 600 mm

(J)



WI3-1
600 mm x 600 mm

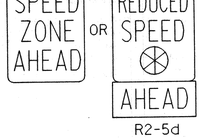
(K)



R2-1
600 mm x 750 mm

(L)

R2-5c
600 mm x 750 mm



R2-5d
600 mm x 300 mm

R2-5b
600 mm x 750 mm

The R2-5d sign shall be used where the speed limit drops 15 MPH or greater. It shall be subsidiary to the R2-5b.

(M)



R4-1
600 mm x 750 mm

(N)



W7-3A
600 mm x 450 mm

(O)



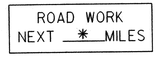
R4-7
600 mm x 750 mm

(P)



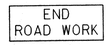
RII-2
1200 mm x 750 mm

(R)



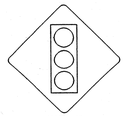
G20-1
1500 mm x 600 mm

(S)



G20-2A
1200 mm x 600 mm

(T)



W3-3
1200 mm x 1200 mm

(U)



W20-7b
1200 mm x 1200 mm

(V)



W21-5
1200 mm x 1200 mm

(W)



W21-5a
1200 mm x 1200 mm

(X)



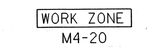
WI4-3
900 mm x 1200 mm
x 1200 mm

(Y)

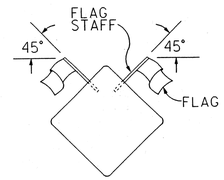


W21-5b
1200 mm x 1200 mm

(Z)



M4-20
600 mm x 150 mm
(BLACK/ORANGE)



TWO (2) 450 mm x 450 mm FLUORESCENT RED-ORANGE FLAGS ARE TO BE ATTACHED IN THE POSITION SHOWN ON SIGNS (B) (C) (D) (E) (F) AND (U). THE FLAGS AND STAFFS ARE TO BE ATTACHED IN SUCH A MANNER THAT THE SIGN FACE WILL NOT BE OBSCURED. THE FLAGS MAY BE EITHER A CLOTH OR VINYL MATERIAL. THE FLAGS SHALL BE SUBSIDIARY TO THE WARNING SIGN BID ITEMS.



AHEAD, ~~1500 FT OR 1MPH~~



AHEAD, 1000 FT, 1500 FT OR 1/2 MILE



RIGHT or LEFT



SPEED TO BE DETERMINED BY THE ENGINEER.

NOTE:

ALL DEVICES SHALL COMPLY WITH THE CURRENT EDITION OF THE MANUAL ON UNIFORM TRAFIC CONTROL DEVICES FOR STREETS AND HIGHWAYS. WHENEVER THIS STANDARD CONFLICTS WITH THE MANUAL, THIS STANDARD SHALL GOVERN.

EXCEPT FOR M-SIGNS USED IN ROUTE MARKER ASSEMBLIES AND AS OTHERWISE NOTED, ALL SIGNS SHALL BE FULLY RETROREFLECTORIZED WITH TYPE II HIGH PERFORMANCE SHEETING.

FOR ALL PROJECTS LET AFTER JULY 1, 1996, ALL SIGNS NOW REQUIRING TYPE II HIGH PERFORMANCE SHEETING SHALL BE REPLACED WITH SIGNS RETROFLECTORIZED WITH FLUORESCENT ORANGE PRISMATIC GRADE RETROFLECTIVE SHEETING.

NOTE:

SIGN (R) IS TO BE USED IF THE PROJECT EXCEEDS 3.2 km. THE LENGTH SHOWN "*" SHALL BE TO THE NEAREST WHOLE MILE.

FOR SPEEDS OF 30 MPH OR LESS, SIGN (E) WILL BE A WI-3. FOR SPEEDS ABOVE 30 MPH, THE WI-4 SIGN WILL BE USED.

WHEN USED ON INTERSTATE, FREEWAY OR EXPRESSWAY ROUTES, SIGNS (K) (L) (M) AND (O) ARE TO BE 1200 mm x 1500 mm, THE R2-5d IS TO BE 1200 mm x 600 mm, AND SIGN (Z) IS TO BE 1200 mm x 600 mm.

SIGNS (K) AND (L) SHOULD BE USED ONLY IF THE ENGINEER DETERMINES THAT A REDUCED SPEED IS REQUIRED THROUGHOUT THE PROJECT.

SIGN (K) SHALL BE INSTALLED THROUGHOUT THE LENGTH OF THE PROJECT AS DIRECTED BY THE ENGINEER, AND BE PAID FOR AT THE "EACH PER DAY" BID FOR THE SIZE OF SIGN USED.

WHERE THE ROAD IS OPEN BEYOND THE POINT OF CLOSURE, EITHER THE RII-3, ROAD CLOSED ( ) MILES AHEAD LOCAL TRAFFIC ONLY SIGN, OR THE RII-4, ROAD CLOSED TO THRU TRAFFIC SIGN, SHOULD BE SUBSTITUTED FOR SIGN (P).

SIGN (S) SHOULD BE PLACED 150 m FROM THE END OF THE ACTUAL WORK AREA, AND NOT NECESSARILY AT THE EXTREME LIMITS OF THE PROJECT.

SIGN (J) IS TO BE ELIMINATED IF THE ADVISORY SPEED IS WITHIN 5 MPH OF THE SPEED LIMIT.

NO.	DATE	REVISION	BY	APP'D
	10-10-96	ADDED SIGN NO. (Z)	WPR	NAB
3	6-9-96	ADDED SIGN NO. TO (W) AND (Y)	TCP	NAB
2	5-8-96	CHANGED SIGN (L)	TCP	NAB
1	5-8-96	CHANGED DATE ON PRISMATIC NOTE	TCP	NAB

KANSAS DEPARTMENT OF TRANSPORTATION

TYPICAL TRAFFIC CONTROL SIGNS

TE755SI 7/1/94

FHWA APPROVAL	12-12-96	APP'D	Nelda A. Buckley
DESIGNED	DETAILED KDS	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

Design Filename: h:\30304\standards\te755sl.dgn
Plotted on Tuesday, February 29, 2000 10:35 AM PYounle

Plotted on Tuesday February 29, 2000 10:35 AM
PYounle Plot Queue: \\WPPLOT\BRDXerox
Plot Scale: 0.025400\1.000000 Pen Table: \\cvpw00\br-ldges\30282\pilot\pltpen\kdotfull.pen
Design Filename: h:\30304\standards\te755sl.dgn

CITY OF MERRIAM, KANSAS

MERRIAM DRIVE BRIDGE
OVER TURKEY CREEK
SCOUR REPAIRS AND BRIDGE DECK REPAIRS

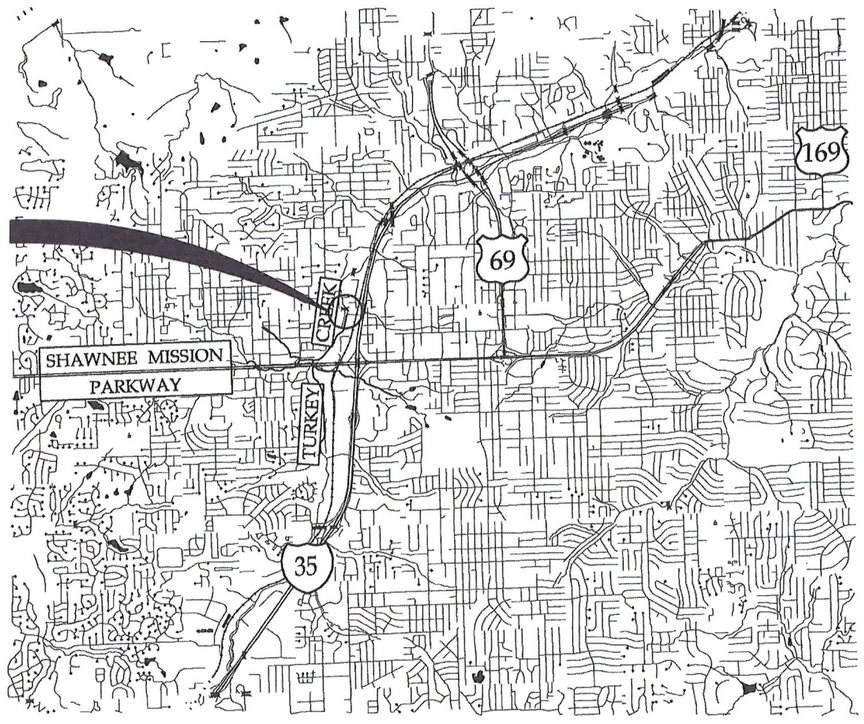
INDEX OF SHEETS

SCOUR REPAIRS:
1. TITLE SHEET
2. PLAN AND NOTES
3. ELEVATION AND SECTION

BRIDGE DECK REPAIRS:
4. PHOTOGRAPH LOCATIONS
5. DECK REPAIR PLAN
6. GENERAL NOTES AND DETAILS
7. SITE PHOTOGRAPHS

8.-10. TRAFFIC CONTROL

PROJECT LOCATION



LOCATION MAP
NO SCALE

TRAFFIC CONTROL:

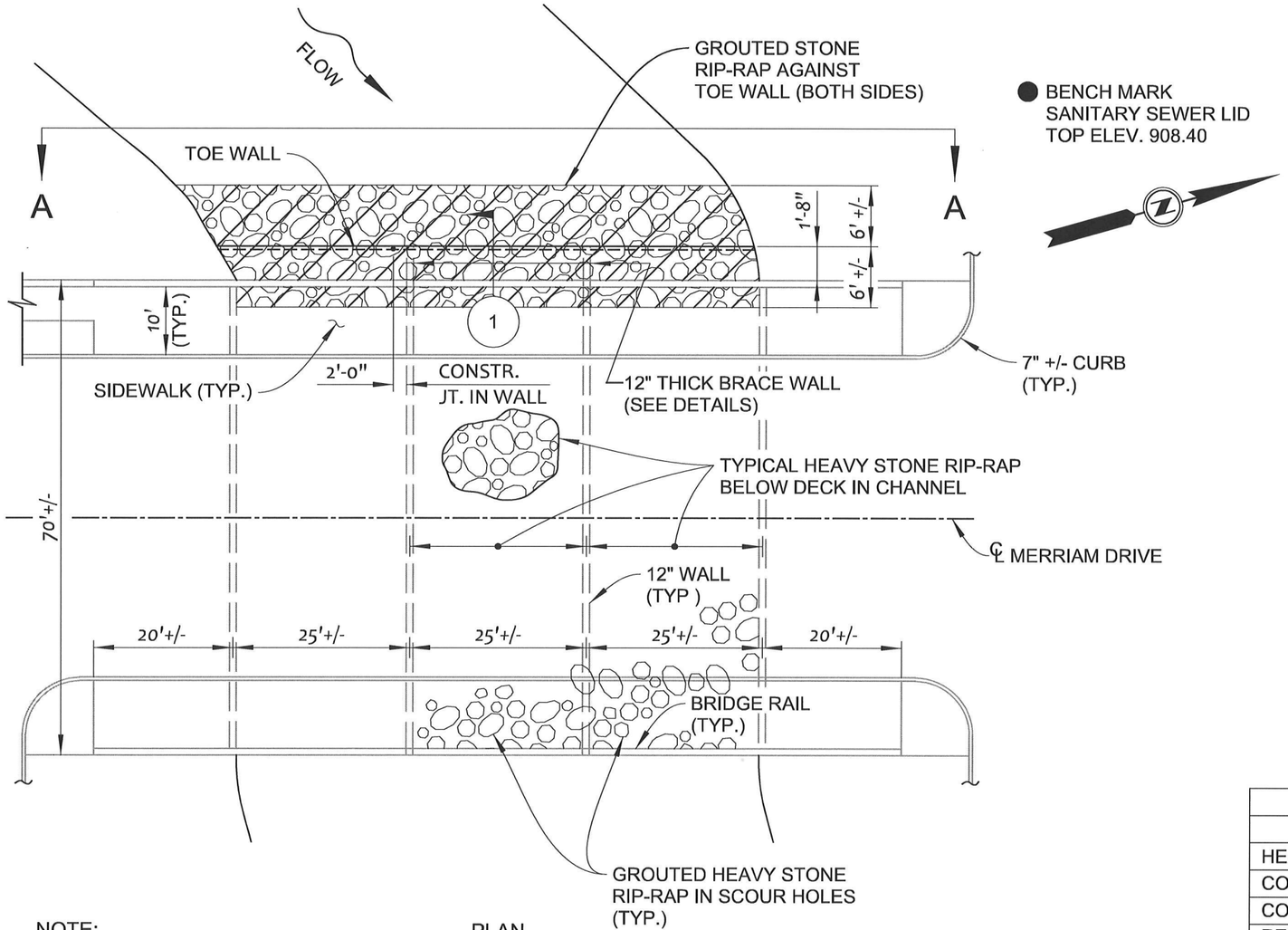
CONTRACTOR SHALL SUBMIT TRAFFIC CONTROL PLAN TO ENGINEER FOR APPROVAL PRIOR TO CONSTRUCTION. TWO (2) LANES OF TRAFFIC SHALL BE MAINTAINED ACROSS THE MERRIAM DRIVE BRIDGE DURING CONSTRUCTION. SIDEWALKS SHALL BE KEPT OPEN FOR PEDESTIAN/BICYCLE TRAFFIC. ACCESS TO BUSINESSES SHALL BE PROVIDED DURING CONSTRUCTION.

MERRIAM
just right.

6200 EBY STREET
MERRIAM, KANSAS 66202
913-322-5500

DELICH, ROTH & GOODWILLIE, P.A.
ENGINEERS
913 SHEIDLEY
SUITE 110
BONNER SPRINGS, KANSAS 66012
CARL REED, 913-213-5142

RECORD DRAWINGS
12/10/2013



PLAN

BENCH MARK
SANITARY SEWER LID
TOP ELEV. 908.40

NOTE:

SEE SHEET #3 FOR SECTION 1.
SEE SHEET #3 FOR ELEVATION A-A.

CONSTRUCTION NOTES:

1. RIP-RAP TO BE HEAVY STONE RIP-RAP (1/4 TON).
2. GROUT FOR HEAVY STONE RIP-RAP SHALL BE KCMMB, 4K WITH APPROVED ADMIXTURE TO CREATE AN 8" MAXIMUM SLUMP (5% - 8% AIR).
3. CONCRETE TOE WALL AND BRACE WALL TO BE 12" THICK KCMMB, 4K WITH APPROVED ADMIXTURE TO CREATE A 5" MAXIMUM SLUMP (5% - 8% AIR).
4. REINFORCING STEEL TO BE GRADE 60 EPOXY COATED BARS WITH 18" MINIMUM LAP.
5. ALL GRADING OF EXISTING GRAVEL BAR AND PLACEMENT OF HEAVY STONE RIP-RAP SHALL BE INCLUDED IN THE BID ITEM "HEAVY STONE RIP-RAP".
6. THE CONTRACTOR SHALL HIRE AN INDEPENDENT LABORATORY ACCEPTABLE TO THE OWNER AND TAKE FOUR (4) CONCRETE CYLINDERS FOR TESTING TOE WALL AND BRACE WALL CONCRETE. ONE CYLINDER SHALL BE TESTED AT 7 DAYS, ONE AT 14 DAYS, AND TWO AT 28 DAYS. TESTING RESULTS SHALL BE SENT TO THE CONTRACTOR AND OWNER AFTER EACH TEST. THE COST FOR THE TESTING SHALL BE SUBSIDIARY.
7. PRIOR TO PLACING MATERIAL IN ANY GIVEN CELL, STREAM FLOW WATER SHALL BE DIVERTED FROM THE AREA BEING MODIFIED TO SOUTH CELL AND WATER SHALL BE REMOVED FROM THE CELL BEING MODIFIED. THE CONTRACTOR SHALL DEVELOP A PLAN FOR DIVERTING STREAM WATER DURING CONSTRUCTION.
8. ALL DIMENSIONS SHOWN IN PLANS ARE APPROXIMATE AND SHALL BE VERIFIED BY THE CONTRACTOR. THE CONTRACTOR SHALL VISIT THE SITE AND MAKE ALL NECESSARY MEASUREMENTS BEFORE SUBMITTING A BID FOR THIS PROJECT.

ESTIMATED QUANTITIES		
ITEM	MEASURE	ESTIMATED QUANTITY
HEAVY STONE RIP-RAP ***	TONS	~~500~~ 658.6
CONCRETE KCMMB, 4K (5" SLUMP)*	C.Y.	~~28.0~~ 22.5
CONCRETE KCMMB, 4K (8" SLUMP)**	C.Y.	~~35.0~~ 82
REINFORCING STEEL (EPOXY)	LBS	~~800~~ 594

* 5" (MAXIMUM SLUMP FOR TOE WALL AND BRACE WALL)
** 8" (MAXIMUM SLUMP FOR GROUT)
*** (GROUTED IN AREAS AS SHOWN)

PLOT DRIVER: P:\12.122.01\Drawings\Batchplots\PDF DRG-Half.plt | PENTABLE: P:\12.122.01\DRAWINGS\BATCHPLOTS\PLAN_COLOR.TBL
DATE: 12/10/2013 | TIME: 7:34:26 AM | FILENAME: P:\12.042.01\Drawings\Sheets\Record Drawings\02-plan-RECORD DRAWING.dgn

REV.	DATE	BY	DESCRIPTION
1			
2			
3			
4			

REVISIONS

SCALE: 1" = 20'
DATE CHECKED: 12/10/2013
DESIGNER C.B.R.
DRAFTER: D.B.R.
CHECKED BY: C.B.R.

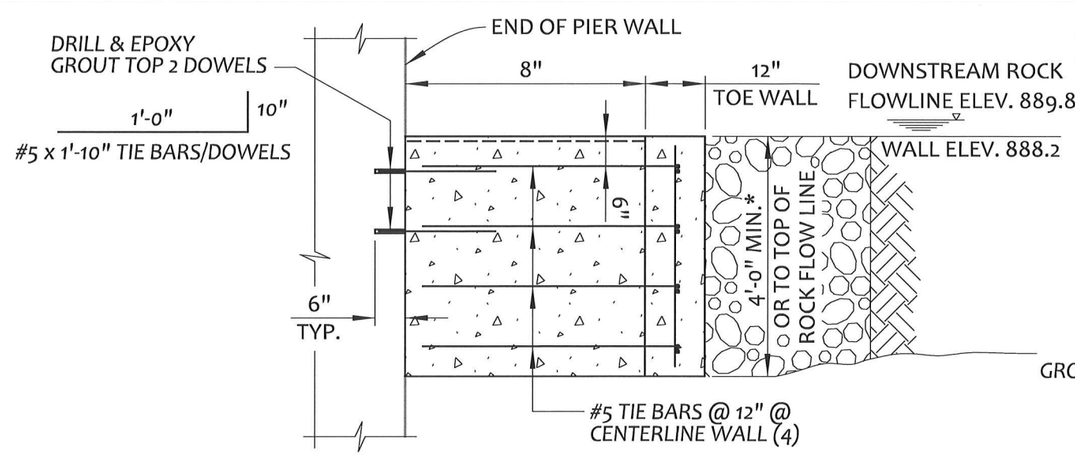
DELICH ROTH & GOODWILLIE, P.A.
ENGINEERS
913 SHEIDLEY, SUITE 110
BONNER SPRINGS, KS 66012
(913)-441-1100

MERRIAM DRIVE BRIDGE OVER TURKEY CREEK SCOUR REPAIRS

CITY OF MERRIAM, KANSAS

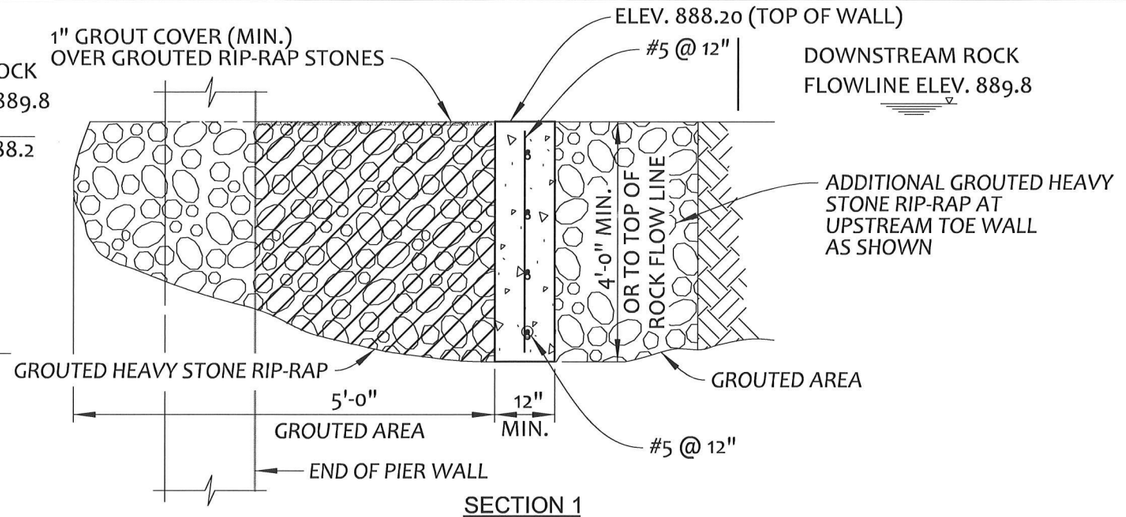
PLAN AND NOTES

PROJECT NO.: 12.042.01
SHEET NO.: 2
OF SHEETS: 10

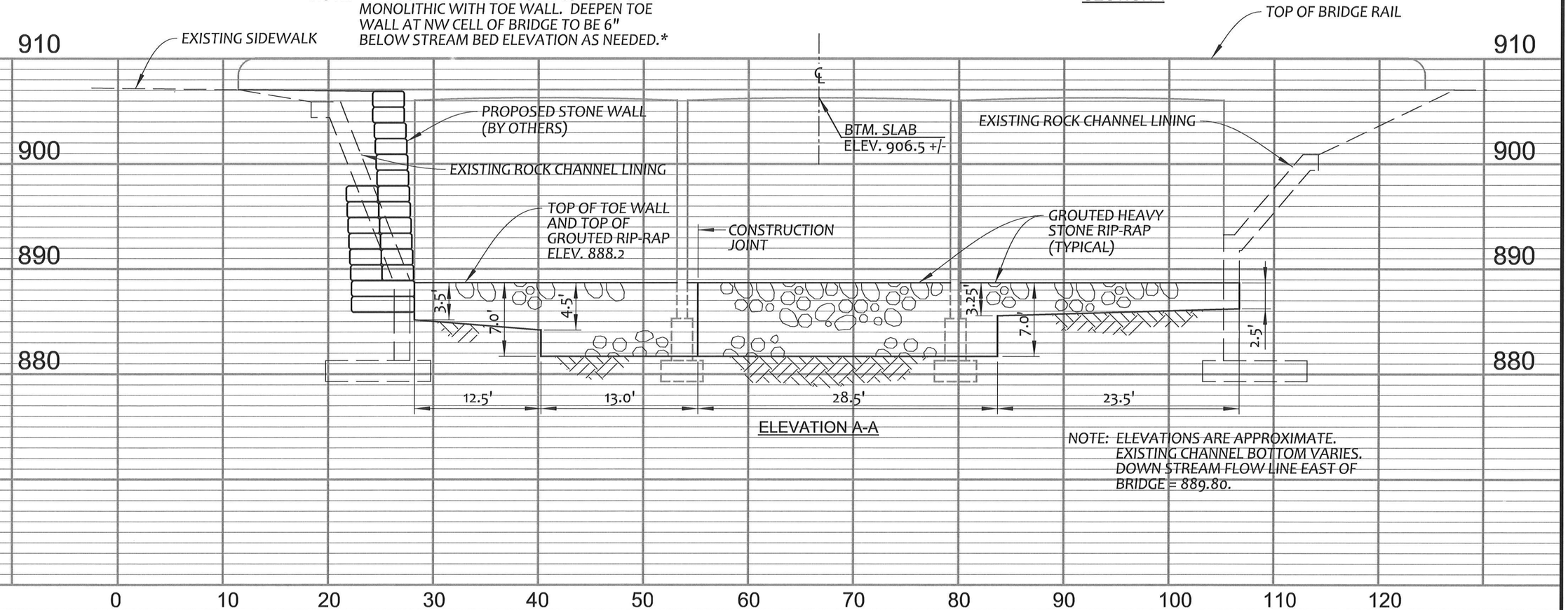


BRACE WALL DETAIL (AT PIER WALLS)

NOTE: 12" THICK BRACE WALL TO BE CONSTRUCTED MONOLITHIC WITH TOE WALL. DEEPEN TOE WALL AT NW CELL OF BRIDGE TO BE 6" BELOW STREAM BED ELEVATION AS NEEDED.*



SECTION 1



ELEVATION A-A

NOTE: ELEVATIONS ARE APPROXIMATE. EXISTING CHANNEL BOTTOM VARIES. DOWN STREAM FLOW LINE EAST OF BRIDGE = 889.80.

REV.	DATE	BY	DESCRIPTION
1			
2			
3			
4			

REVISIONS

SCALE: 1" = 10' H&V
DATE CHECKED: 12/10/2013
DESIGNER: C.B.R.
DRAFTER: D.B.R.
CHECKED BY: C.B.R.

DELICH ROTH & GOODWILLIE, P.A. ENGINEERS
913 SHEIDLEY, SUITE 110
BONNER SPRINGS, KS 66012
(913)-441-1100

MERRIAM DRIVE BRIDGE
OVER TURKEY CREEK
SCOUR REPAIRS

CITY OF MERRIAM, KANSAS

ELEVATION AND SECTION

PROJECT NO.: 12.042.01
SHEET NO.: 3
OF SHEETS: 10

PLOT DRIVER: P:\12.122.01\Drawings\Batchplots\PDF DRG-Half.plt PENTABLE: P:\12.122.01\DRAWINGS\BATCHPLOTS\PLAN_COLOR.TBL
DATE: 12/10/2013 TIME: 11:03:39 AM FILENAME: P:\12.042.01\Drawings\Sheets\Record Drawings\03-elevation-RECORD DRAWING.dgn



LEGEND:

PHOTO #

SEE SHEET #7 FOR PHOTOS.

Plot Driver: P:\12.122.01\Drawings\Batchplots\PDF DRG-Half.plt
Pentable: P:\12.122.01\DRAWINGS\BATCHPLOTS\PLAN_COLOR.TBL
Date: 5/28/2013 | Time: 12:49:45 PM | File Name: P:\12.042.01\Drawings\Sheets\04-plan.dgn

NO.	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			

REVISIONS

DESIGNER: C.B.R.
DRAFTER: D.B.R.
CHECKED BY: C.B.R.
DATE CHECKED: 5/28/2013

DELICH ROTH & GOODWILLIE, P.A.
ENGINEERS
913 SHEIDLEY, SUITE 110
BONNER SPRINGS, KS 66012
(913) 441-1100

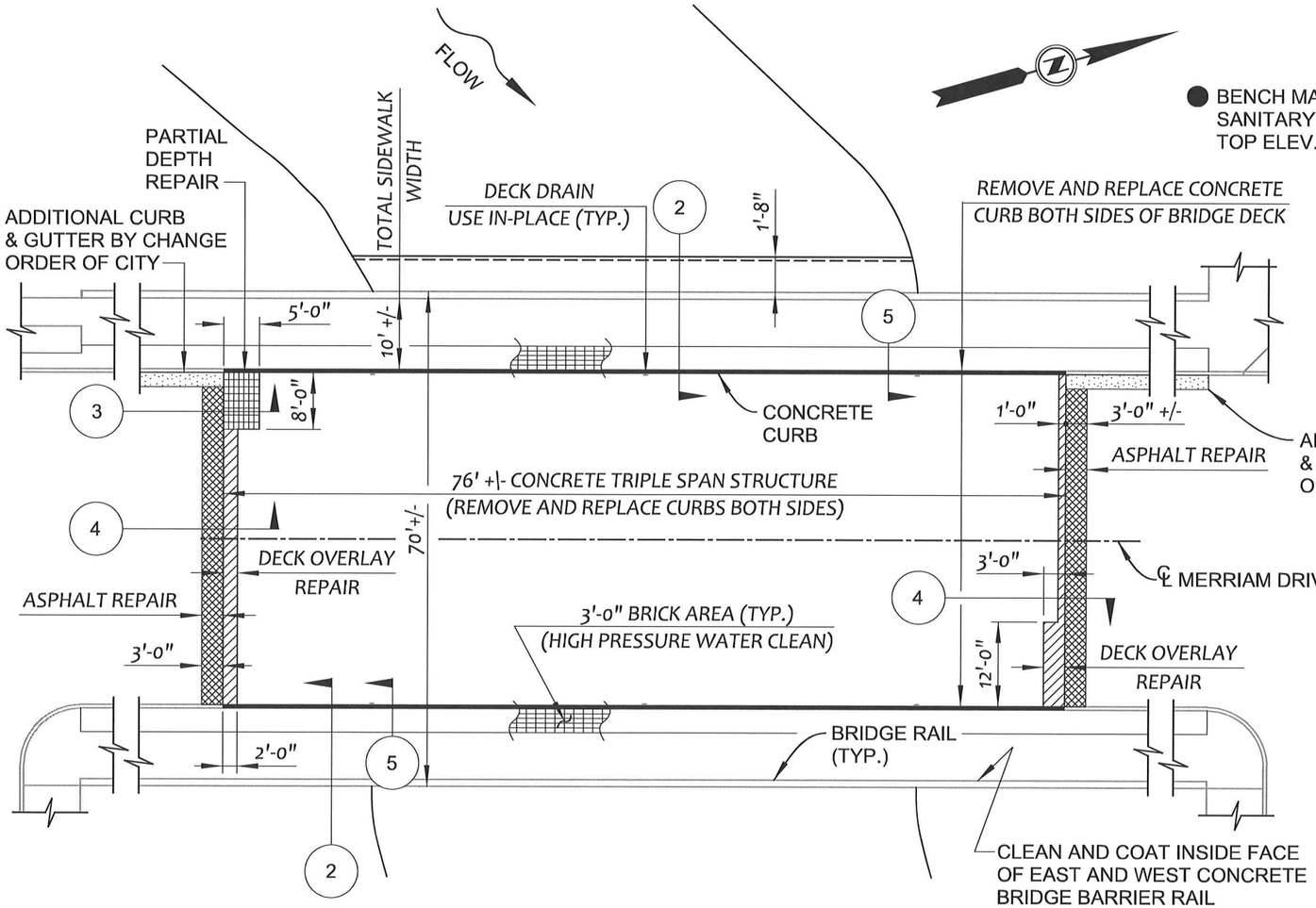
MERRIAM DRIVE BRIDGE
OVER TURKEY CREEK
BRIDGE DECK REPAIRS

CITY OF MERRIAM, KANSAS

PHOTOGRAPH LOCATIONS

DRG PROJECT NO. 12.078.01

SHEET 4 OF 10



BENCH MARK
SANITARY SEWER LID
TOP ELEV. 908.40

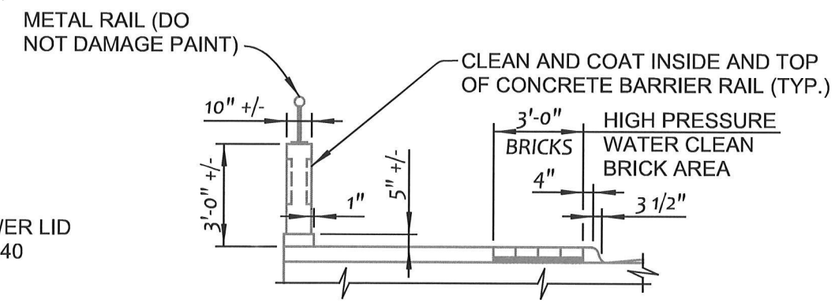
PLAN

NOTE:

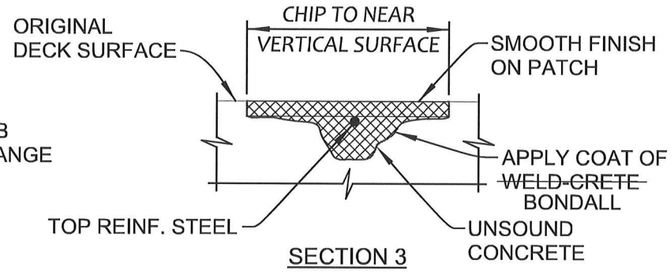
SEE SHEET #3 FOR SECTION 1.
SEE SHEET #3 FOR ELEVATION A-A.
RECORD DRAWING OVERLAY AREAS NOT REDRAWN (AREAS VARY).

LEGEND

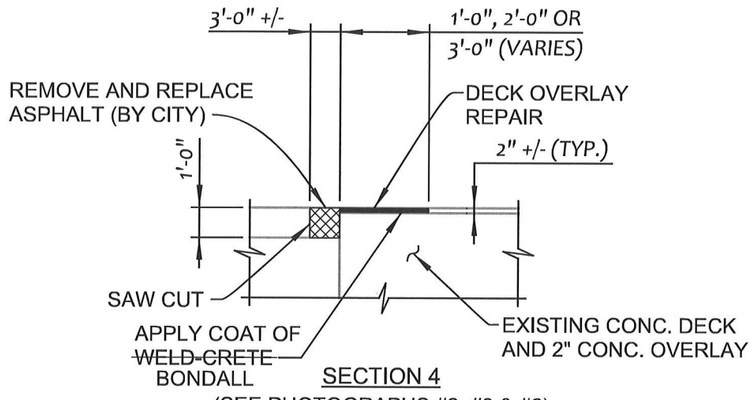
- DECK OVERLAY REPAIR
- ASPHALT REPAIR
- PARTIAL DEPTH REPAIR
- REMOVE AND REPLACE CURB



SECTION 2
(SEE PHOTOGRAPH #8)



SECTION 3
(SEE PHOTOGRAPH #4)



SECTION 4
(SEE PHOTOGRAPHS #2, #3 & #6)

REV.	DATE	BY	DESCRIPTION
1			
2			
3			
4			

REVISIONS

SCALE: 1" = 20'
DATE CHECKED: 12/10/2013
DESIGNER: C.B.R.
DRAFTER: D.B.R.
CHECKED BY: C.B.R.

DELICH ROTH & GOODWILLIE, P.A.
ENGINEERS
913 SHEIDLEY, SUITE 110
BONNER SPRINGS, KS 66012
(913)-441-1100

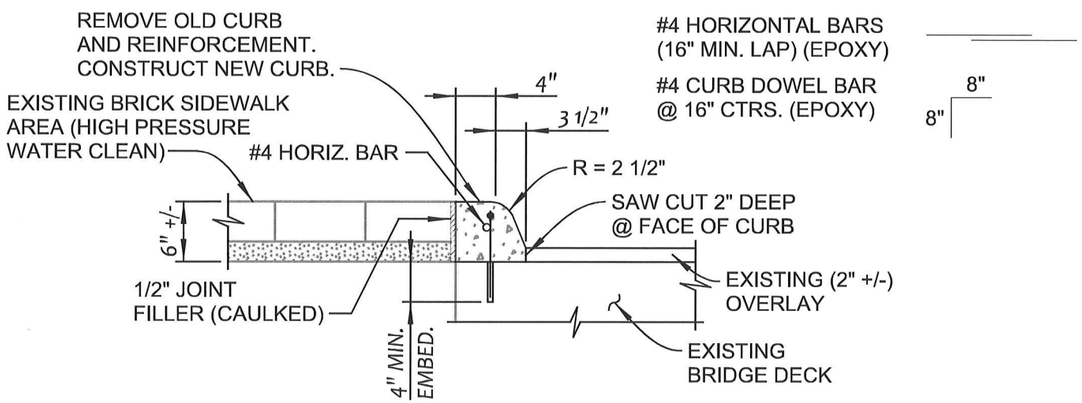
MERRIAM DRIVE BRIDGE OVER TURKEY CREEK BRIDGE DECK REPAIR

CITY OF MERRIAM, KANSAS

DECK REPAIR PLAN

PROJECT NO.: 12.078.01
SHEET NO.: 5
OF SHEETS: 10

PLOT DRIVER: P:\12.122.01\Drawings\Batchplots\PDF DRG-Half.plt | PENTABLE: P:\12.122.01\DRAWINGS\BATCHPLOTS\PLOTSIPLAN_COLOR.TBL
DATE: 12/10/2013 | TIME: 9:18:37 AM | FILENAME: P:\12.042.01\Drawings\Sheets\Record Drawings\05-plan-RECORD DRAWING.dgn



SECTION 5
(SEE PHOTOGRAPH #7)

ESTIMATED QUANTITIES		
ITEM	MEASURE	ESTIMATED QUANTITY
TRAFFIC CONTROL	L.S.	1
AREA PREPARED FOR PATCHING (PARTIAL DEPTH)	SQ. YDS.	~~4.5~~ 3.7
DECK OVERLAY REPAIR	SQ. YDS.	~~17.5~~ 18.51
CURB REPLACEMENT	L.F.	152
CLEAN BRICK PAVERS	SQ. FT.	720
CLEAN AND COAT BRIDGE BARRIER RAIL	SQ. FT.	910
ASPHALT REPAIR (BY OTHERS)	TONS	10

CONSTRUCTION AND GENERAL NOTES:

1. CONCRETE: Concrete for bridge Deck Overlay Repair, Curb Replacement and Area Prepared for Patching (Partial Depth), shall be KCMMB, 4K with 3" maximum slump (5%-8% Air).

2. REINFORCING STEEL: Reinforcing steel shall be ASTM A615, Grade 60 (epoxy coated). Drilling and epoxy grouting (4" minimum embedment) of vertical curb dowel bars shall be SUBSIDIARY to bid item "Curb Replacement".

3. AREA PREPARED FOR PATCHING: This item shall consist of removing unsound concrete and bituminous patches from the bridge deck, cleaning reinforcing bars and filling the removed patched areas with concrete. Quantity shown is an estimate of the areas involved. The exact areas shall be determined by tapping, before, during and after chipping operation to assure that all unsound concrete has been removed. See KDOT Specifications. Areas to be patched will be determined by the Engineer. After unsound material is removed, the area shall be high pressure air cleaned and the surface shall be coated with ~~WELD-CRETE~~ BONDALL bonding agent prior to placing new concrete.

4. DECK OVERLAY REPAIR: The unsound deck overlay (2" +/-) shall be removed as indicated on the plans. The exact area will be determined by the Engineer. After removal of the unsound concrete overlay, the area shall be high pressure air cleaned and the surface shall be coated with ~~WELD-CRETE~~ BONDALL bonding agent prior to placing new concrete.

5. CURB REPLACEMENT: The existing concrete curb for the length of the bridge deck shall be removed down to the top of the existing bridge deck. Saw cut a straight line 2" deep through the existing (2" +/-) overlay at the edge of the existing curb. Remove existing horizontal and vertical reinforcement. Install new curb dowel bars at 16" centers (max.) by drilling and epoxying into existing bridge deck with a minimum embedment of 4" Clean the curb repair area with high pressure air and apply ~~WELD-CRETE~~ BONDALL bonding agent prior to placing new curb. All material, labor and equipment shall be included in bid item "Curb Replacement".

6. CLEAN AND COAT BRIDGE BARRIER RAIL: The inside face and top of the existing bridge barrier rails on both sides of the bridge shall be sand blast cleaned and shall be coated with white SUPER THOROSEAL per manufacturers recommendations. The existing metal handrail shall be protected to prevent damage to the painted surfaces. Contractor shall repaint, at no cost to the owner, any damaged handrail areas with paint approved by the owner.

7. ASPHALT REPAIR: The deteriorated aphalt at the ends of concrete bridge shall be removed and replaced as indicated on the plans. The replacement asphalt shall be Superpave (By City).

8. TRAFFIC CONTROL: Contractor shall submit traffic control plan to Engineer for approval prior to construction. Two (2) lanes of two (2) way traffic shall be maintained across the Merriam Drive Bridge during construction. Sidewalks shall be kept open for pedestrian/bicycle traffic. Access to businesses shall be provided during construction.

PLOT DRIVER: P:\12.122.01\Drawings\Batchplots\PDF DRG-Half.plt | PENTABLE: P:\12.122.01\DRAWINGS\BATCHPLOTS\PLAN_COLOR.TBL
DATE: 12/10/2013 | TIME: 9:07:21 AM | FILENAME: P:\12.042.01\Drawings\Sheets\Record Drawings\06-notesanddetails-RECORD DRAWING.dgn

REV.	DATE	BY	DESCRIPTION
1			
2			
3			
4			

REVISIONS

SCALE: 1" = 20'
DATE CHECKED: 12/10/2013
DESIGNER C.B.R.
DRAFTER: D.B.R.
CHECKED BY: C.B.R.

DELICH ROTH & GOODWILLIE, P.A. ENGINEERS
913 SHEIDLEY, SUITE 110
BONNER SPRINGS, KS 66012
(913)-441-1100

MERRIAM DRIVE BRIDGE
OVER TURKEY CREEK
BRIDGE DECK REPAIR

CITY OF MERRIAM, KANSAS

GENERAL NOTES AND DETAILS

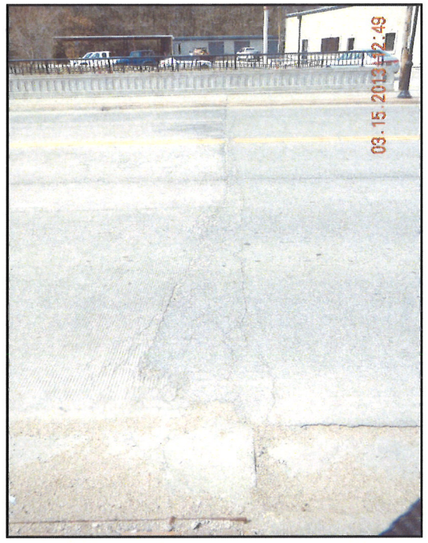
PROJECT NO.: 12.078.01
SHEET NO.: 6
OF SHEETS: 10



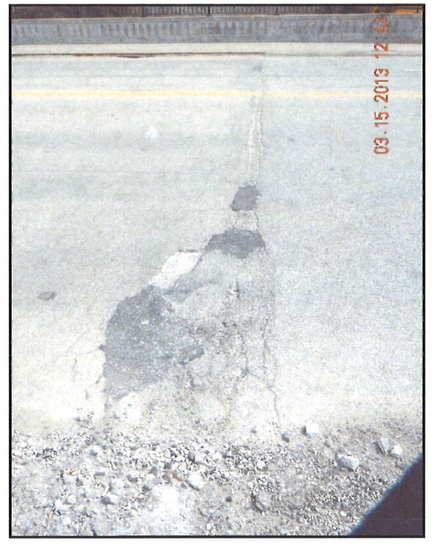
PHOTOGRAPH #1
WEST SIDE OF BRIDGE/LOOKING NORTHEAST



PHOTOGRAPH #2
JOINT AT NORTH END OF BRIDGE/LOOKING EAST



PHOTOGRAPH #3
JOINT AT NORTH END OF BRIDGE/LOOKING WEST



PHOTOGRAPH #4
JOINT AT SOUTH END OF BRIDGE/LOOKING EAST



PHOTOGRAPH #5
CLOSE UP OF TYPICAL DECK DRAIN



PHOTOGRAPH #6
JOINT AT SOUTH END OF BRIDGE/LOOKING WEST



PHOTOGRAPH #7
CONCRETE CURB AND BRICK SIDEWALK AREA



PHOTOGRAPH #8
TYPICAL CONCRETE BARRIER WALL



PHOTOGRAPH #9
ACCESS RAMP TO TURKEY CREEK

NO.	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			

REVISIONS

DESIGNER: C.B.R.
DRAFTER: D.B.R.
CHECKED BY: C.B.R.
DATE CHECKED: 5/29/2013

DELICH, ROTH & GOODWILLIE, P.A.
ENGINEERS
913 SHEIDLEY AVE., STE. 110
BONNER SPRINGS, KS 66012
(913)-441-1100
14 WEST 3rd STREET, STE. 220
KANSAS CITY, MO 64105
(816)-221-4222

MERRIAM DRIVE BRIDGE
OVER TURKEY CREEK
BRIDGE DECK REPAIR

CITY OF MERRIAM, KANSAS

SITE PHOTOGRAPHS

DRG PROJECT NO. 12.078.01
SHEET 7
OF 10

Plot Driver: P:\12.122.01\Drawings\Batchplots\PDF DRG-Half.plt
Pentable: P:\12.122.01\DRAWINGS\BATCHPLOTS\PLAN_COLOR.TBL
Date: 5/29/2013 Time: 7:51:20 AM File Name: P:\12.042.01\Drawings\Sheets\07-photos.dgn

1. MUTCD COMPLIANCE:

ALL TEMPORARY TRAFFIC CONTROL DEVICES AND THEIR INSTALLATION AND MAINTENANCE SHALL COMPLY WITH THE LATEST EDITION OF THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (MUTCD) FOR STREETS AND HIGHWAYS WHICH HAS BEEN ADOPTED BY THE SECRETARY OF TRANSPORTATION. WHENEVER THE TEMPORARY TRAFFIC CONTROL STANDARDS CONFLICT WITH THE MUTCD, THE STANDARDS SHALL GOVERN.

2. DESIGN SPEED:

THOSE ITEMS DELEGATED TO TEMPORARY TRAFFIC CONTROL SHOULD BE DESIGNED AND INSTALLED USING THE POSTED/LEGAL SPEED OF THE ROADWAY PRIOR TO WORK STARTING.

3. CLEAR ZONE:

ALL CONSTRUCTION EQUIPMENT (INCLUDING VEHICLES), MATERIALS, AND DEBRIS SHALL BE STORED OUT OF THE CLEAR ZONE. WHERE THIS CANNOT BE ACHIEVED, THE CONTRACTOR SHALL PLACE APPROPRIATE SIGNS, OBJECT IDENTIFIERS, AND/OR BARRICADES AS DESIGNATED BY THE ENGINEER. TEMPORARY TRAFFIC CONTROL DEVICES NEEDED FOR THIS CONDITION SHALL BE CONSIDERED SUBSIDIARY TO OTHER BID ITEMS.

4. MINIMUM LANE WIDTHS:

LANE WIDTHS SHALL BE A MINIMUM OF 11' (MEASURED BETWEEN CENTERLINES OF PAVEMENT MARKINGS) OR AS SHOWN ON THE PLANS, OR AS DIRECTED BY THE ENGINEER. A LANE WIDTH LESS THAN 11' MAY REQUIRE RESTRICTED ROADWAY WIDTH SIGNING.

5. FLAGGER:

A MINIMUM OF ONE FLAGGER SHALL BE STATIONED WITHIN EACH MULTI-LANE ROADWAY ACTIVITY AREA WHERE WORK IS IN A CLOSED LANE ADJACENT TO TRAFFIC AND NOT SEPARATED BY A CONCRETE SAFETY BARRIER SYSTEM.

6. PAVEMENT MARKING:

WHEN THE WORK WILL OCCUPY A LOCATION MORE THAN THREE DAYS, ALL CONFLICTING PAVEMENT MARKINGS SHALL BE REMOVED OR MASKED AND ALL TRANSITION TAPERS, CROSSOVERS, AND EDGE LINES ALONG CHANNELIZING DEVICES SHALL BE MARKED WITH SOLID 4" WIDE PAVEMENT MARKING.

7. FIRST MODULE OF IBS:

THE FIRST MODULE OF EACH INERTIAL BARRIER SYSTEM (IBS) SHALL HAVE A MINIMUM OF 2 SQ. FT. OF FLUORESCENT ORANGE ASTM TYPE IV SHEETING FACING TRAFFIC. EITHER A VERTICAL RECTANGLE OR DIAMOND SHAPE MAY BE USED.

8. PEDESTRIAN / BICYCLE SAFETY:

WORK ZONE SIGNS SHALL NOT INHIBIT PEDESTRIAN AND BICYCLE TRAFFIC ON SIDEWALKS OR OTHER AREAS DESIGNATED FOR PEDESTRIAN OR BICYCLE USE.

CONSIDERATION SHOULD BE MADE TO SEPARATE PEDESTRIAN AND BICYCLE MOVEMENTS FROM BOTH WORK SITE ACTIVITY AND VEHICULAR TRAFFIC. UNLESS A REASONABLE SAFE ROUTE THAT DOES NOT INVOLVE CROSSING THE ROADWAY CAN BE PROVIDED, PEDESTRIANS AND BICYCLISTS SHOULD BE APPROPRIATELY DIRECTED WITH ADVANCE SIGNING THAT ENCOURAGES THEM TO CROSS TO THE OPPOSITE SIDE OF THE ROADWAY. IN URBAN AND SUBURBAN AREAS WITH HIGH VEHICULAR TRAFFIC VOLUMES, THESE SIGNS SHOULD BE PLACED AT INTERSECTIONS (RATHER THAN MIDBLOCK LOCATIONS) SO THAT PEDESTRIANS AND BICYCLISTS ARE NOT CONFRONTED WITH MIDBLOCK WORK SITES THAT WILL INDUCE THEM TO ATTEMPT SKIRTING THE WORK SITE OR MAKING A MIDBLOCK CROSSING.

WHEN EXISTING PEDESTRIAN FACILITIES ARE DISRUPTED, CLOSED, OR RELOCATED, THE TEMPORARY FACILITIES SHALL BE DETECTABLE AND INCLUDE ACCESSIBILITY FEATURES CONSISTENT WITH THE FEATURES PRESENT IN THE EXISTING PEDESTRIAN FACILITY.

9. CHANGED STOP CONDITIONS:

ATTACH TWO FLAGS AND A RED TYPE "B" HIGH INTENSITY WARNING LIGHT TO ANY STOP SIGN THAT CREATES A NEW STOP CONDITION OR MOVES THE STOP CONDITION TO A NEW LOCATION. LEAVE FLAGS AND LIGHTS IN PLACE FOR AT LEAST THE FIRST 30 DAYS. INSTALL W3-1 (SYMBOLIC STOP AHEAD) SIGN IN ADVANCE OF STOP SIGN IF STOP SIGN IS NOT VISIBLE FOR A MINIMUM OF DISTANCE 'A' (SEE CHART ON TE710) OR IF STOP CONDITION IS MOVED TO LESS THAN DISTANCE 'A' FROM AN EXISTING STOP AHEAD SIGN.

10. LUMP SUM BIDDING:

WHEN TRAFFIC CONTROL IS BID LUMP SUM, ADDITIONAL DEVICES WILL BE PAID FOR AS EXTRA WORK.

11. NIGHTTIME LIGHTING:

WHEN NIGHTTIME WORK IS REQUIRED, FLOODLIGHTS SHOULD BE USED TO ILLUMINATE FLAGGER STATIONS, EQUIPMENT CROSSINGS, AND OTHER AREAS WHERE EXISTING LIGHTING IS NOT ADEQUATE FOR THE WORK TO BE PERFORMED SAFELY.

IN NO CASE SHALL FLOODLIGHTS BE PERMITTED TO CREATE A DISABLING GLARE FOR THE DRIVER. THE ADEQUACY OF THE FLOODLIGHT PLACEMENT AND ELIMINATION OF POTENTIAL GLARE SHOULD BE CHECKED BY DRIVING THROUGH THE PROJECT.

12. NCHRP REPORT 350 CRASHWORTHY REQUIREMENTS:

TRAFFIC CONTROL DEVICES SHALL MEET THE EVALUATION CRITERIA IN NCHRP REPORT 350 AS SUPPLEMENTED BY FHWA MEMORANDUM "IDENTIFYING ACCEPTABLE HIGHWAY SAFETY FEATURES," DATED JULY 25, 1997. AVAILABLE ON THE INTERNET AT http://safety.fhwa.dot.gov/roadway_dept/road_hardware/nchrp_350.htm

ANY DEVICE NOT ADDRESSED BY THE TE STANDARDS MAY BE APPROVED ON A CASE BY CASE BASIS BY THE ENGINEER. THE DEVICE SHALL BE ACCOMPANIED BY AND INSTALLED ACCORDING TO NCHRP REPORT 350.

THE CONTRACTOR SHALL:

1) PROVIDE TO THE ENGINEER A COPY OF THE MANUFACTURER'S SELF-CERTIFICATION THAT ANY CATEGORY 1 (i.e. - PLASTIC CONICAL DELINEATORS, TUBULAR MARKERS, DRUMS WITHOUT ATTACHMENTS) AND CATEGORY 2 (i.e. - PORTABLE SIGN STANDS (WITH SIGNS), TYPE II AND III BARRICADES, AND VERTICAL PANELS) DEVICES USED ON THE PROJECT ARE NCHRP REPORT 350 COMPLIANT.

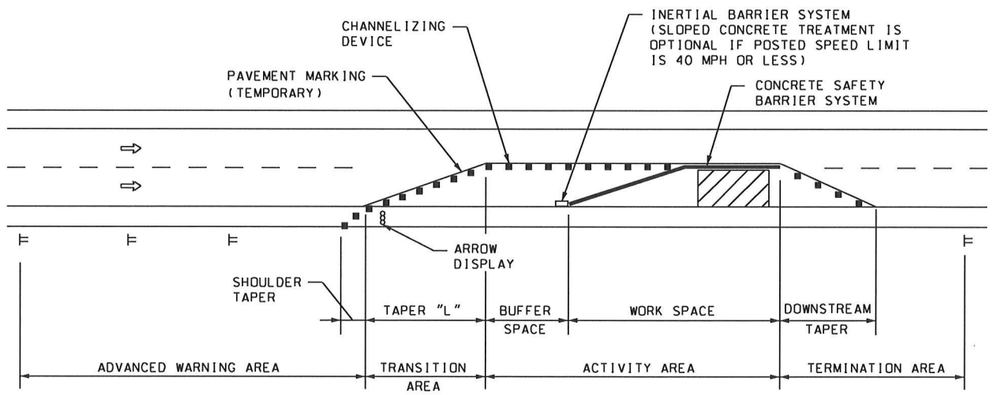
2) PROVIDE TO THE ENGINEER A COPY OF THE ENTIRE FHWA NCHRP REPORT 350 ACCEPTANCE LETTER (WZ-xxx) FOR ANY CATEGORY 2 DEVICE (i.e. - PORTABLE SIGN STANDS (WITH SIGNS), TYPE II AND III BARRICADES, AND VERTICAL PANELS) USED ON THE PROJECT. WORK ZONE FHWA NCHRP REPORT 350 ACCEPTANCE LETTERS (WZ-xxx) ARE AVAILABLE ON THE INTERNET AT:
http://safety.fhwa.dot.gov/roadway_dept/road_hardware/listing.cfm?code=workzone

3) CERTIFY THAT THE TRUCK MOUNTED ATTENUATORS (TMA'S) (WHICH ARE DEFINED AS CATEGORY 3 DEVICES BY THE FHWA MEMORANDUM) WERE PURCHASED PRIOR TO OCTOBER 1, 1998, AND INCLUDE A COPY OF THE ENTIRE FHWA ACCEPTANCE LETTER STATING THAT THE TMA'S ARE NCHRP REPORT 230 COMPLIANT; OR IF THE DEVICES WERE PURCHASED AFTER OCTOBER 1, 1998, INCLUDE A COPY OF THE ENTIRE FHWA'S ACCEPTANCE LETTER STATING THAT THE TMA'S ARE NCHRP REPORT 350 COMPLIANT.

ALL CATEGORY 1 & 2 DEVICES SHALL BE NCHRP REPORT 350 COMPLIANT. TMA'S, PURCHASED PRIOR TO OCTOBER 1, 1998, MAY BE USED UNTIL THE END OF THEIR SERVICEABLE LIVES.

13. TYPE "A" LOW INTENSITY WARNING LIGHTS:

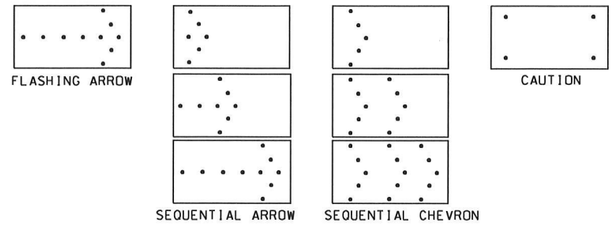
A TYPE "A" LOW INTENSITY WARNING LIGHT IS AN L.E.D. BI-DIRECTIONAL FLASHING WORK ZONE WARNING LIGHT.



TYPICAL WORK ZONE COMPONENTS

NOTE:
REFER TO STD. TE702 FOR TAPER "L" FORMULA.

ARROW DISPLAYS



ARROW DISPLAY ELEMENTS SHALL BE CAPABLE OF A MINIMUM 50 PERCENT DIMMING FROM THEIR FULL-RATED LAMP VOLTAGE. FULL LAMP VOLTAGE SHOULD BE USED DURING THE DAY AND DIMMED MODE SHALL BE USED AT NIGHT. FOR SHOULDER WORK, ROADSIDE WORK NEAR THE SHOULDER, BLOCKING THE SHOULDER, OR FOR TEMPORARY CLOSING ONE LANE ON A TWO-LANE, TWO-WAY ROADWAY, AN ARROW PANEL SHALL BE USED ONLY IN THE CAUTION MODE.

BUFFER SPACE

SPEED (MPH) ✱	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*	*65*	*70*
LENGTH (ft)	*115*	*155*	*200*	*250*	*305*	*360*	*425*	*495*	*570*	*645*	*730*

NEITHER WORK ACTIVITY NOR STORAGE OF EQUIPMENT, VEHICLES, OR MATERIAL SHOULD OCCUR IN THE BUFFER SPACE. WHEN A PROTECTION VEHICLE IS PLACED IN ADVANCE OF THE WORK SPACE, ONLY THE SPACE UPSTREAM OF THE VEHICLE CONSTITUTES THE BUFFER SPACE.

✱ POSTED SPEED PRIOR TO WORK STARTING

IF TEMPORARY CONCRETE SAFETY BARRIER SYSTEM IS USED TO SEPARATE APPROACHING TRAFFIC FROM THE WORK SPACE, THE BARRIER SYSTEM SHALL BE CONSIDERED PART OF THE ACTIVITY AREA. A FULL LANE WIDTH SHOULD BE AVAILABLE THROUGHOUT THE LENGTH OF THE BUFFER SPACE. SEE TYPICAL WORK ZONE COMPONENTS.

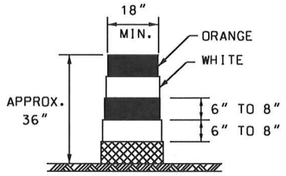
NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	REVISED NOTE 7	J.M.	A.A.A.
2	8-8-07	ADD NOTE 13, REVISE NOTE 8 & WZ COMPONENTS	M.B.	A.A.A.
1	12-29-05	MODIFIED BUFFER SPACE TABLE	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

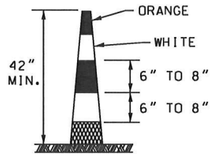
GENERAL TRAFFIC CONTROL

TE700 9/1/00

FHWA APPROVAL 4-20-09		APP'D Anthony A. Akrobaire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.



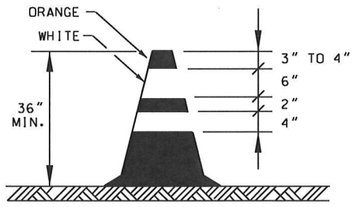
DRUM



CONICAL
DELINEATOR

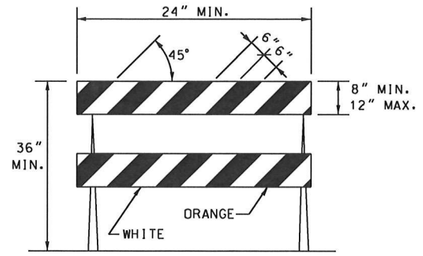
DRUMS AND CONICAL DELINEATORS SHALL HAVE AT LEAST TWO ORANGE AND TWO WHITE 6" TO 8" WIDE RETROREFLECTIVE STRIPES. ADDITIONAL STRIPES MAY BE NON-RETROREFLECTIVE. IF THERE ARE NON-RETROREFLECTIVE SPACES BETWEEN ADJACENT STRIPES, THEY SHALL BE NO MORE THAN 3" WIDE.

ALL RETROREFLECTIVE STRIPES ON DRUMS SHALL BE ASTM TYPE III SHEETING. THE WHITE STRIPES ON CONICAL DELINEATORS SHALL BE ASTM TYPE III SHEETING. ORANGE STRIPES ON ALL CONICAL DELINEATORS SHALL BE FLUORESCENT ORANGE ASTM TYPE IV SHEETING.



TRAFFIC CONE

TRAFFIC CONES MAY BE USED AS CHANNELIZING DEVICES FOR DAYTIME OPERATIONS ONLY. THEY WILL NOT BE PAID FOR SEPARATELY, BUT WILL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS. THE ENGINEER MAY REQUIRE THAT TRAFFIC CONES BE SUPPLEMENTED BY OTHER TRAFFIC CONTROL DEVICES IN CERTAIN SITUATIONS.

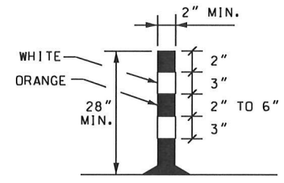


TYPE II BARRICADE

FOR RAILS LESS THAN 36" LONG, 4" WIDE STRIPES MAY BE USED.

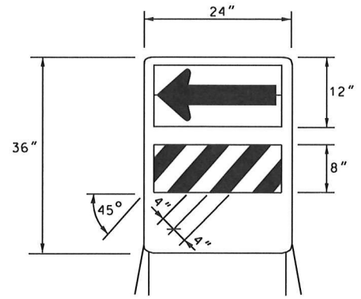
THE ENTIRE AREA OF BARRICADE RAILS, BOTH FRONT AND BACK, SHALL BE ASTM TYPE III SHEETING.

THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.



TUBULAR MARKER

THE TWO WHITE RETROREFLECTIVE STRIPES SHALL BE ASTM TYPE III SHEETING.

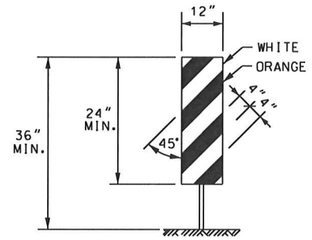


DIRECTION INDICATOR BARRICADE

THE ARROW PANEL SHALL BE BLACK ON FLUORESCENT ORANGE ASTM TYPE IV SHEETING. THE STRIPES SHALL BE ORANGE AND WHITE ASTM TYPE III SHEETING SLOPING DOWNWARD IN THE DIRECTION TRAFFIC IS TO PASS.

THE DIRECTION INDICATOR BARRICADE SHALL BE USED IN SERIES TO DIRECT THE MOTORIST INTO THE INTENDED LANE OF TRAVEL.

THE ARROW PANEL SHOULD NOT BE VISIBLE TO OPPOSING TRAFFIC.



VERTICAL PANEL

THE ENTIRE AREA OF VERTICAL PANELS, BOTH FRONT AND BACK, SHALL HAVE ASTM TYPE III SHEETING. THE STRIPES SHALL SLOPE DOWNWARD TO THE TRAFFIC SIDE FOR CHANNELIZATION.

ITEM / LOCATION		CROSS-OVERS	SHOOFLY DIVERSIONS	TANGENTS	TAPERS	RAMPS	HEAD TO HEAD	OBJECT IDENTIFIER	
PORTABLE									
	DRUMS	YES	YES	YES	YES	YES	(1)	YES	
	CONICAL DELINEATORS	YES	YES	YES	YES	YES	(1)	YES	
	VERTICAL PANELS	(2)	(2)	(2)	(2)	(2)	(1,2)	YES	
	DIRECTION INDICATOR BARRICADE	NO	NO	NO	YES	NO	NO	NO	
	TYPE II BARRICADE	(2)	(2)	(2)	(2)	NO	NO	YES	
FIXED									
	TUBULAR MARKERS	(3)	(3)	(3)	NO	(3)	YES	NO	
	VERTICAL PANELS	(3)	(3)	(3)	(3)	(3)	(3)	YES	

(1) Not allowed on centerline delineation along freeways or expressways.

(2) The stripes shall slope downward to the traffic side for channelization.

(3) May be used upon the approval of the Engineer.

TAPER FORMULAS:

L = WS FOR SPEEDS OF 45 MPH OR MORE

$L = WS^2/60$ FOR SPEEDS OF 40 MPH OR LESS

WHERE: L = MINIMUM LENGTH OF TAPER IN FEET
S = NUMERICAL VALUE OF POSTED SPEED PRIOR TO WORK STARTING IN MPH
W = WIDTH OF OFFSET IN FEET

CHANNELIZER PLACEMENT:

(A) THE SPACING BETWEEN DEVICES IN TRANSITION AREA (TAPER) SHOULD NOT EXCEED A DISTANCE IN FEET EQUAL TO 1/2 THE POSTED SPEED LIMIT IN MPH PRIOR TO WORK STARTING.

(B) THE SPACING BETWEEN DEVICES IN THE ADVANCED WARNING AREA AND THE ACTIVITY AREA SHOULD NOT EXCEED A DISTANCE IN FEET EQUAL TO TWO TIMES THE POSTED SPEED LIMIT IN MPH PRIOR TO WORK STARTING.

(C) CHANNELIZING DEVICES SHALL BE PLACED FOR OPTIMUM VISABILITY, NORMALLY AT RIGHT ANGLES TO THE TRAFFIC FLOW.

(D) CHANNELIZING DEVICES PLACED ALONG SHOULDER EDGES OR IN DROPOFFS SHALL HAVE A MINIMUM OF 24" FROM THE TOP OF THE CHANNELIZING DEVICE TO THE TOP OF THE PAVEMENT.

NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	CHANNELIZER PLACEMENT & TRAFFIC CONE DETAIL	J.M.	A.A.A.
2	8-8-07	TRIMLINE CHANGED TO CONICAL DELINEATOR	M.B.	A.A.A.
1	11-19-03	CHANGED BORDER	B.H.	S.A.B.

KANSAS DEPARTMENT OF TRANSPORTATION

CHANNELIZING DEVICES

TE702 9/1/00

FHWA APPROVAL 4-20-09		APP'D Anthony A. Alrobaire	
DESIGNED L.E.R.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

GENERAL NOTES

1. MAINTENANCE:

THE CONTRACTOR SHALL MAINTAIN ALL SIGNS AND DEVICES IN AN UPRIGHT POSITION. THE CONTRACTOR SHALL CLEAN OR REPLACE ANY DAMAGED OR ILLEGIBLE SIGN OR DEVICE AS DIRECTED BY THE ENGINEER.

2. EXISTING SIGNS:

IF EXISTING SIGNS THAT ARE TO REMAIN (WHETHER DENOTED ON THE PLANS OR NOT) INTERFERE WITH CONSTRUCTION WORK, THE CONTRACTOR SHALL REMOVE, STORE, AND RESET THE SIGNS. THIS SHALL BE SUBSIDIARY TO OTHER TRAFFIC CONTROL BID ITEMS. SIGNING DAMAGED BY THE CONTRACTOR SHALL BE REPLACED AT THE CONTRACTOR'S EXPENSE.

3. CONFLICTING SIGNS, SIGNS NOT IN USE, AND TRAFFIC SIGNALS:

SIGNS AND TRAFFIC SIGNALS THAT ARE IN CONFLICT WITH THE TRAFFIC CONTROL PLAN OR DO NOT APPLY TO THE TRAFFIC OPERATIONS SHALL BE IMMEDIATELY REMOVED, TURNED SO NOT VISIBLE TO TRAFFIC FROM ANY DIRECTION, OR COMPLETELY COVERED WITH ADEQUATE OPAQUE WATERPROOF MATERIAL. TAPE SHALL NOT BE APPLIED TO THE FACE OF THE SIGN.

4. PORTABLE AND POST MOUNTED SIGNS:

TEMPORARY TRAFFIC CONTROL SIGNS THAT ARE ANTICIPATED TO REMAIN IN PLACE FOR 3 DAYS OR LESS ARE CONSIDERED "PORTABLE." PORTABLE SIGNS SHALL BE MOUNTED ON AN APPROVED SUPPORT AT A MINIMUM HEIGHT OF 12" ABOVE THE TRAVELED WAY. TRAFFIC CONTROL SIGNS IN PLACE FOR OVER 3 DAYS ARE REQUIRED TO BE MOUNTED ON APPROVED POSTS. A MINIMUM OF 42" OF THE APPROVED POST MUST BE BELOW THE GROUND SURFACE WITH ADEQUATE BACKFILL AND COMPACTION. ALL POSTS AT MINIMUM SHALL EXTEND TO THE TOP EDGE OF THE SIGN AND NO GREATER THAN 6" ABOVE THE SIGN.

WHEN THE SIGN WIDTH IS EQUAL TO OR GREATER THAN 9', THREE OR MORE WOOD POSTS MAY BE USED WITH A MINIMUM OF 4' BETWEEN THE CENTERLINE OF EACH POST. ALL SIGNS LESS THAN 9' IN WIDTH SHALL USE A MAXIMUM OF TWO WOOD POSTS.

"ROLL-UP" SIGNS MAY BE USED FOR PORTABLE WARNING SIGNS. THEY MUST BE FLUORESCENT ORANGE ASTM TYPE IV SIGNS OF OPAQUE MATERIAL. MESH SIGNS ARE NOT ALLOWED.

5. SHEETING:

ALL ORANGE SIGNS SHALL HAVE FLUORESCENT ORANGE ASTM TYPE IV SHEETING. ALL OTHER SIGNS SHALL HAVE ASTM TYPE III SHEETING OF STANDARD COLORS.

6. SIGNS INVOLVING SPEEDS:

THE W3-5 (SPEED REDUCTION) SHOULD BE USED ONLY IF THE ENGINEER DETERMINES THAT A REDUCED SPEED IS REQUIRED ON THE PROJECT.

THE KM4-20 (WORK ZONE) PLAQUE SHALL BE PLACED ABOVE ALL SPEED LIMIT SIGNS, (R2-1), EXISTING AND TEMPORARY. MOUNT THE WORK ZONE PLAQUES TO THE POST. DO NOT OVERLAP THE R2-1 AND KM4-20 SIGNS.

FOR SPEEDS OF 30 MPH OR LESS, THE W1-1(TURN) OR W1-3(REVERSE TURN) SHOULD BE USED. FOR SPEEDS OF 35 MPH OR MORE, THE W1-2(CURVE) OR W1-4(REVERSE CURVE) SHOULD BE USED. THE W13-1(MPH) IS TO BE ELIMINATED IF THE ADVISORY SPEED IS WITHIN 5 MPH OF THE SPEED LIMIT.

7. SIGNS CONTROLLING WORK ZONE:

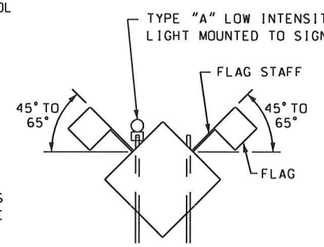
THE KG20-2(END ROAD WORK) SHOULD BE PLACED 500' FROM THE END OF THE ACTUAL WORK SPACE, NOT NECESSARILY AT THE EXTREME LIMITS OF THE PROJECT. THE KG20-2 SHOULD BE MOUNTED ON TWO POSTS. THE KG20-2 MAY BE MOUNTED ON ONE POST IF IN URBAN AREAS WHERE UTILITIES ARE A PROBLEM AND WIND LOADS ARE NOT AN ISSUE.

WHERE TWO WORK ZONES ARE LESS THAN 1 MILE APART IN RURAL AREAS OR $^{1}/_{4}$ MILE APART IN URBAN AREAS, THE KG20-2(END ROAD WORK) FOR THE FIRST WORK ZONE AND THE W20-1(ROAD WORK) FOR THE SECOND WORK ZONE SHOULD BE ELIMINATED.

8. WARNING LIGHTS ON SIGNS:

TYPE "A" LOW INTENSITY WARNING LIGHTS SHOULD BE USED WITH ALL CONSTRUCTION ACTION WARNING SIGNS AND SHALL NOT BE USED ON SIGNS MOUNTED LESS THAN 5' HIGH ON TEMPORARY SUPPORTS. ON ALL OTHER CONSTRUCTION WARNING SIGNS, TYPE "A" LOW INTENSITY WARNING LIGHTS ARE TO BE USED AS DIRECTED BY THE ENGINEER.

TYPE "A" LOW INTENSITY WARNING LIGHTS SHALL BE MAINTAINED SO AS TO BE CAPABLE OF BEING VISIBLE ON A CLEAR NIGHT FROM A DISTANCE OF 3000 FT. IF A TYPE "A" LOW INTENSITY WARNING LIGHT HAS A SEPARATE BATTERY CASE, THE BATTERY CASE SHALL BE MOUNTED NO HIGHER THAN 12" ABOVE THE GROUND AND MOUNTED BEHIND THE SIGN POST. A TYPE "A" LOW INTENSITY WARNING LIGHT WHERE THE LENS AND BATTERY ARE ONE UNIT SHALL BE MOUNTED ON THE TEMPORARY SIGN POST NEAREST TO THE TRAVELED WAY. FLAGS SHALL NOT INTERFERE WITH THE VISABILITY OF THE TYPE "A" LOW INTENSITY WARNING LIGHT.



TWO (2) 18" x 18" FLUORESCENT RED-ORANGE FLAGS SHALL BE ATTACHED (IN THE POSITION SHOWN) ON THE W20-2(DETOUR), W1-1(TURN), W1-2(CURVE), W1-3(REVERSE TURN), W1-4(REVERSE CURVE), W3-3(SIGNAL AHEAD), W4-2(LANE REDUCTION), W20-4(ONE LANE ROAD), W20-5(LANE CLOSED), W20-7A(FLAGGER), AND W3-4 (BE PREPARED TO STOP) SIGNS AND ANY OTHER ACTION SIGNS AS SHOWN ON THE PLANS OR DIRECTED BY THE ENGINEER. THE FLAGS AND STAFFS ARE TO BE ATTACHED IN SUCH A MANNER THAT THE SIGN WILL NOT BE OBSCURED. THE FLAGS MAY BE EITHER A CLOTH OR VINYL MATERIAL. THE FLAGS SHALL BE SUBSIDIARY TO THE CONSTRUCTION SIGN BID ITEMS.

9. AFAD'S

FLAGGER SHALL BE A PERSON WHO PROVIDES TEMPORARY TRAFFIC CONTROL.

FLAGGER STATIONS SHALL BE LOCATED SUCH THAT APPROACHING ROAD USERS WILL HAVE SUFFICIENT DISTANCE TO STOP AT AN INTENDED STOPPING POINT.

THE FIVE CATEGORIES OF WORK DURATION AND THEIR TIME AT A LOCATION SHALL BE:

A. LONG-TERM STATIONARY IS WORK THAT OCCUPIES A LOCATION FOR MORE THAN 3 DAYS.
B. INTERMEDIATE-TERM STATIONARY IS WORK THAT OCCUPIES A LOCATION MORE THAN ONE DAYLIGHT PERIOD UP TO 3 DAYS, OR NIGHTTIME WORK LASTING LONGER THAN 1 HOUR.
C. SHORT-TERM STATIONARY IS DAYTIME WORK THAT OCCUPIES A LOCATION FOR MORE THAN 1 HOUR WITHIN A SINGLE DAYLIGHT PERIOD.
D. SHORT DURATION IS WORK THAT OCCUPIES A LOCATION UP TO 1 HOUR.
E. MOBILE IS WORK THAT MOVES INTERMITTENTLY OR CONTINUOUSLY.

ONE LANE, TWO-WAY VEHICULAR TRAFFIC FLOW REQUIRES AN ALL-RED INTERVAL OF SUFFICIENT DURATION FOR ROAD USERS TO CLEAR THE PORTION OF THE TEMPORARY TRAFFIC CONTROL ZONE CONTROLLED BY THE TRAFFIC CONTROL SIGNALS. SAFEGUARDS SHALL BE INCORPORATED TO AVOID THE POSSIBILITY OF CONFLICTING SIGNAL INDICATIONS AT EACH END OF THE TEMPORARY TRAFFIC CONTROL ZONE.

SIGN LAYOUT INFORMATION



KG20-2
(BLACK ON ORANGE)

STD. SIZE EXPWY/FREEWAY
6" C
48"x 24"



KM4-20
(BLACK ON ORANGE)

STD. SIZE	EXPWY/FREEWAY
3" C	6" C
24"x 6"	48"x 12"

NOTE:
TEXT DIMENSIONS ARE IN INCHES.
BORDER IS BLACK NON-REFLECTIVE.
SEE STD. TE730 OR TE731 FOR KG20-5 SIGN LAYOUT.

LETTER SIZES FOR BLACK ON ORANGE "DESTINATION" SIGNS

STD. SIZE	EXPWY/FREEWAY
6" C	10" D



COLORS: W7-3a

LEGAND - BLACK (NON-REFLECTIVE)
BACKGROUND - ORANGE (REFLECTIVE)

MILEAGE TO BE DETERMINED BY THE ENGINEER.



W8-11

COLORS:
LEGAND - BLACK (NON-REFLECTIVE)
BACKGROUND - ORANGE (REFLECTIVE)



W8-9a

COLORS:
LEGAND - BLACK (NON-REFLECTIVE)
BACKGROUND - ORANGE (REFLECTIVE)

THIS SIGN IS TO BE INSTALLED FOR CONDITIONS OF HEIGHT DIFFERENTIAL BETWEEN ADJACENT LANES OVER ONE (1) INCH. IT IS TO BE PLACED AT EACH INTERSECTING CROSSROAD, MAJOR TRAFFIC GENERATOR, OR AT APPROXIMATELY ONE-HALF (1/2) MILE INTERVALS. THE SIGNS ARE TO BE REMOVED OR COVERED WHEN NOT APPLICABLE.

MINIMUM ADVANCE WARNING SIGN SPACING (IN FEET):

	A	B	C
URBAN (40 MPH OR LOWER)	100	100	100
URBAN (45 MPH OR HIGHER)	350	350	350
RURAL (55 MPH OR LOWER)	500	500	500
RURAL (60 MPH OR HIGHER)	750	750	750
EXPRESSWAY/FREEWAY	1000	1500	2640

THE MINIMUM SPACING BETWEEN SIGNS SHALL BE NO LESS THAN 100', UNLESS DIRECTED BY THE ENGINEER.

THE SPACING BETWEEN ANY SIGNS MAY BE INCREASED BEYOND THE MINIMUM VALUES IN THE TABLE ABOVE AS APPROVED BY THE ENGINEER IN ORDER TO MAXIMIZE VISIBILITY.

NO.	DATE	REVISIONS	BY	APP'D
3	4-20-09	ADDED AFAD NOTE & NEW SIGN LAYOUT	J.M.	A.A.A.
2	8-8-07	MODIFIED NOTES 4, 5, 6, 7 & 8, KG20-2 LAYOUT	M.B.	A.A.A.
1	12-29-05	MODIFIED FLAGS, M4-20 & SIGN LAYOUT INFO	M.B.	A.A.A.

KANSAS DEPARTMENT OF TRANSPORTATION

TRAFFIC CONTROL SIGNS

TE710 9/1/00

FHWA APPROVAL 4-20-09		APP'D Anthony A. Akroboire	
DESIGNED B.A.H.	DETAILED B.A.H.	QUANTITIES	TRACED
DESIGN CK.	DETAIL CK.	QUAN. CK.	TRACE CK.

This page was intentionally left blank for duplex printing.

APPENDIX F – HAZARDOUS AND TOXIC WASTE

This page was intentionally left blank for duplex printing.

ENVIRONMENTAL LAWS AND REGULATIONS

Various Federal and State legislation regulates the proper use, disposal, and clean up of hazardous materials and waste.

Resource Conservation and Recovery Act (RCRA)

Passed by Congress in 1976, this law gives the Environmental Protection Agency the authority to regulate hazardous waste disposal. This is accomplished by tracking the hazardous waste from its generation to its disposal. In 1984, there were several amendments made to this act, called the Hazardous and Solid Waste Act (HSWA). The amendments were enacted to include underground storage tank regulation, all releases of hazardous waste to the environment, and mandates for corrective action at hazardous waste facilities.

Toxic Substance Control Act (TSCA)

Also enacted by Congress in 1976, this law deals with all chemicals produced or imported into the United States. It requires that the chemical be adequately tested for toxicity prior to its commercial release to the public. In 1979, this act was amended to include the prohibition of the manufacture and distribution polychlorinated biphenyls (PCBs). PCBs are required to be labeled and disposed of properly. In the Statutes of Kansas, K.A.R. 28-55-5 states that PCB treatment, storage and disposal facilities shall comply with all applicable requirements of 40 CFR 761 subpart D, as in effect on November 1, 1986 and 40 CFR 264 subparts B, C, D, G and H, as in effect on November 1, 1986.

Comprehensive Environmental Response, Compensation and Liability Act (CERCLA)

Nicknamed Superfund, this law was enacted to start a trust fund for the payment of remediation at abandoned hazardous waste sites. This law was passed in 1980. The issue of abandoned or uncontrolled sites has grown through the years. The EPA manages the National Priority List, which lists certain CERCLA sites of the highest priority. By law, the State of Kansas also maintains a registry list of all abandoned and uncontrolled hazardous waste sites within the state. Superfund Amendments and Reauthorization Act (SARA) amended CERCLA in 1984. The amendments included more state and community involvement in the remediation process, increased focus on human health problems posed by contaminated sites, selection of remedies that permanently cleaned the site, and consideration of standards and other requirements in State and other Federal laws and regulations.

Federal Facility Compliance Act (FFCA)

In 1992, Congress passed the Federal Facility Act, which requires the Department of Energy (DOE) to develop and submit Site Treatment Plans for the development of capacity and technologies for treating mixed waste. All facilities that the DOE stores or generates these wastes must have such a plan. It also requires schedules for bringing new treatment facilities into operation.

State Regulations

The State of Kansas provides regulations for the use, management, and disposal of hazardous waste. These regulations are found in Article 31 on Hazardous Waste Management with additional statutes on Public Health in Chapter 65 and Hazardous Waste in Regulation Article 34. Environmental Use Controls are mentioned in K.S.A 65-1, 221 to 65-1, 235 and gives the state authority to assist existing programs to help address environmental contamination in a cost-effective manner and restrict the use of land where controls exist. It also empowers the State to enforce the regulations, defines hazardous waste, and controls the management, treatment, and disposal of hazardous waste. Statute Article 34 Solid Waste and Regulation Article 55 establishes criteria for facilities that handle and dispose of PCB waste. Article 28 contains regulations pertaining to the protection of surface waters and the water supply from environmental contamination and establishes water quality standards.

RECORDS SEARCH

A records search for potential contaminated sites along the reach of Turkey Creek addressed by this DDR, was performed in 2009, and updated in 2015 and again in 2023. The records search consisted reviewing various Federal and State of Kansas databased for past and present environmental activities and enforcement actions at properties adjacent to the creek. The information sources reviewed as part of this records search included:

- USEPA Enviromapper - GIS database identifying Superfund sites, sites were hazardous releases have occurred, and facilities regulated by the USEPA that handle materials designated as hazardous waste.

- USEPA CERCLIS Database - The Comprehensive Environmental Response, Compensation, and Liability Information System (CERCLIS) database contains general information on sites across the nation and US territories including location, status, contaminants, and actions taken. It provides information on sites being assessed under the Superfund Program, hazardous waste sites and potential hazardous waste sites.

- KDHE Identified Sites – Contaminated sites managed by the Bureau of Remediation (except for leaking underground or above ground petroleum storage tanks) are included in the Identified Sites List (ISL).

- KDHE AST and UST Assessment Database – this database provides information on site were leaks have occurred or a tank has been closed.

- KDHE Database of Registered USTs

- KDHE Environmental Interest Finder – Kansas consolidated from the Identified Sites List, the AST and UST Assessment Database, and the Database of Registered USTs into an interactive online GIS system. Information in this system was reviewed in February 2023. A link to the KDHE interactive online GIS system is:

  https://maps.kdhe.state.ks.us/keif/

- USEPA Cleanups In My Community (CIMC) Map/Database – EPA and it's state, territory and tribal partners have developed a variety of cleanup programs to assess and, where necessary, clean up these contaminated sites. CIMC is intended to provide information about sites that might need to be cleaned up, are being cleaned up, or have been cleaned, in a particular community or anywhere in the United States. The scope of sites included in the CIMC GIS database are: Brownfields Properties, Brownsfields Grants Jurisdictions, RCRA Corrective Action, Superfund, Federal Facilities and Federal Agency Hazardous Waste Compliance Docket, Removals/Responses, and Recovery Act Funded Cleanups. The link to the CIMC Map/Database is:

  https://ofmpub.epa.gov/apex/cimc/f?p=cimc:MAP:0:::71:P71_WELSEARCH:AL|State|AL|||true|true|true|true|true|true||-1|sites|N|basic

FINDINGS

Below are the findings from the records search:

Superfund Sites

There were no Superfund sites identified in areas of potential soil disturbance.

Sites identified on CERCLIS Database

The CERCLIS Database identified no sites being investigated or remediated for hazardous waste within the areas of potential soil disturbance.

KDHE Identified Sites

There were no identified sites in the proximity of potential soil disturbance.

Solid Waste Facilities

There are no known solid waste facilities in the project area.

Registered and Closed Underground Storage Tanks (USTs)

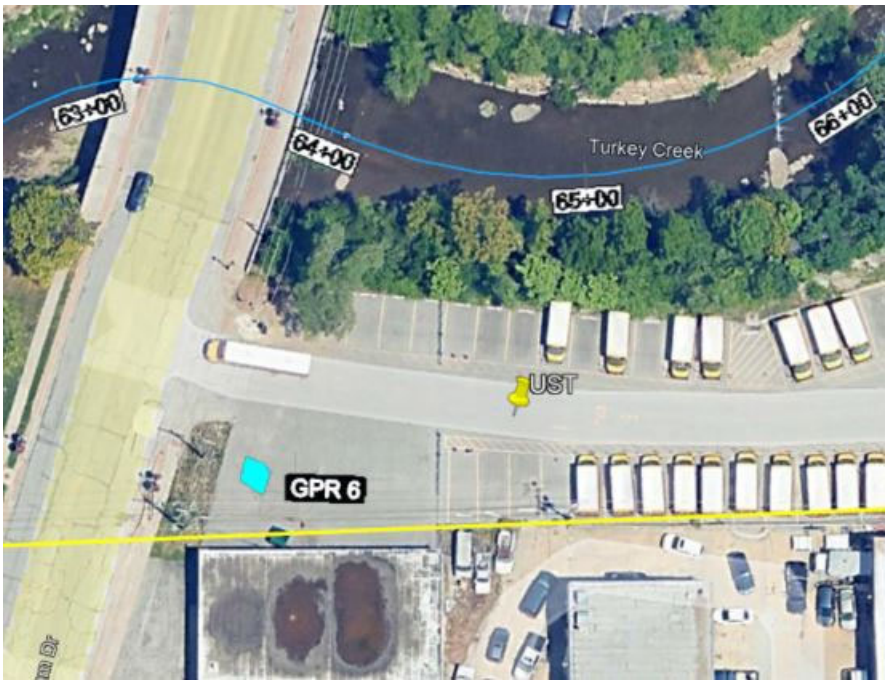
There are six properties along the project reach that formerly or currently have USTs. These are described below, working from downstream to upstream.

- 5568 Merriam Dr, Merriam, KS
  State of Kansas records identify this location as either previously or currently having a tank. No other information is provided on the tank size or contents other than it is out of use. The proposed levee is approximately 75-ft from the building on the property and the location of the tank is unknown. There was a Ground Penetrating Radar (GPR) survey performed on the property and two subsurface anomalies were discovered but it is uncertain if these are USTs (see figure below) If the tank on this property did have a past leak, there is the potential petroleum contamination may be encountered when excavating the inspection trench. It is recommend a boring be advanced in the location of the proposed levee to determine contaminated soil will need to be addressed during construction. See Appendix F, Figure F-1 for location.



GPR Anomalies on 5568 Merriam Dr

- 5639 Merriam Drive, Merriam, KS
  This property contained one-300 gallon waste oil UST, a 10,000 gallon gasoline UST, and a 2,000 gallon diesel UST. This USTs and the associated distribution piping were removed. During removal, 20 CY of contaminated soil was removed near the 300 gallon waste oil tank and 150 CY of contaminated soil was removed near the gasoline and diesel tanks. The GPR survey of the property discovered a subsurface anomaly on a property near the entry road to this property (5643 Merriam Dr). However, KDHE records do not identify a UST on 5643 Merriam Dr. This property also has two existing 10,000 gallon USTs that are in use. The presumed location of the existing tank is shown in Appendix F, Figure F-1. They were installed in 2019 and have all the latest protective measures to detect and contain any leakage. The potential of encountering petroleum contamination during construction of the proposed floodwall is believed to be low since contaminated soil was removed during closure of the older tanks. Due to the location of the existing tanks, it does not appear construction of the floodwall will be impacted.



GPR Anomaly on 5643 Merriam Dr

- 5726 Merriam Drive
  During subsurface investigation via borings, debris was discovered on this site. There were no tanks identified in the KDHE records for this property, however a GPR survey identified two subsurface anomalies on the property (see Figure below).



GPR Anomalies on 5726 Merriam Dr

- 5730 Merriam Drive, Merriam KS
  Three 560 gallon gasoline USTs were removed from this property. The KDHE records indicate no contaminated was encountered. (see Appendix F, Figure F-2 for location). The building associated with this property was demolished to develop the Farmers Market.

- 5745 Merriam Dr, Merriam, KS
  Kansas records indicate there was a UST on this property but status except to indicate it is out of use. The location, size, and former contents of the tank are also unknown. The property is more than 300 feet from the proposed levee, so it is unlikely there will be any impact to construction of the levee.

- 5903 Merriam Dr, Merriam, KS
  There are two 10,000 gallon gasoline USTs and one 8,000 gallon diesel UST being actively used on this property (see Appendix F, Figure F-3). This is no indication of past leaks from these tanks and they have spill basins, so it is unlikely contaminated soil will be encountered during installation of the proposed floodwall and gas main.

- 6032 Merriam Dr, Merriam, KS
  There was a 500 gallon waste oil UST removed from this property in 1990. Kansas records indicate the tank did not leak and there was not contaminated soil. Therefore, there is not expected to be any contamination encountered during installation flood wall near this property.

- Additional Subsurface Anomolies
  There were three additional subsurface anomalies discovered at 9400 W 61st St, 9420 W 61st, and 80 feet west of the intersection of Merriam Ln and W 61st st. KDHE records do not identify these properties as having USTs.



GPR Anomalies on 9400, 9420 W 61st St, and 80 feet West of the intersection of W61st and Merriam Ln

Additional information on the GPR survey findings is in the table below.

Ground Penetrating Radar (GPR) Investigation Summary					
GPR #	Approx. Depth (ft)	Approx. Surface Area (sqft)	Approx. Location along UTC Alignment	Bank (Left or Right)	Parcel Location
1	Undetermined	103.6	26+75	R	Northwest corner of Parcel 126
2	4.5	235.8	28+00	R	Southwest edge of Parcel 109
3	3.3	89.1	29+75	R	North edge of Parcel 109 / South edge of Parcel 106
4	Undetermined	319.3	53+30	R	Center West of Parcel 27
5	5.2	77.7	53+50	R	Center East of Parcel 27
6	1.9	13.8	64+00	R	Southeast corner of Parcel 10
7	Undetermined	133.7	62+50	L	Northeast corner of Parcel 17 / Northwest corner of Parcel 15
8	3.3	55.7	63+00	L	Northeast corner of Parcel 15

Spills

There were a number of spills report along the reach of Turkey addressed by this DDR. Most were associated with minor leaks of mineral oil from transformers and contamination associated with the spill was cleaned up. The only significant spill reported is described below:

- 61st and Merriam Ln, Merriam, KS
  On April 11th, 2022, and vehicle accident knocked down three utility poles and impacted four transformers. It was estimated that 91 pints of mineral oil from the leaked from the transformers and 95% of the oil went down a storm drain and into Turkey Creek. The oil did not contain Polychlorinated Biphenyls (PCBs), so it is not anticipated this was negatively impact construction of the proposed levee in this area. See Appendix F, Figure F-5 for location.

Other Areas of Concern

There were two areas of concern identified during investigations performed in February of 2023. A proposed levee crosses the west side of this property and it is also the location of a proposed retention basin. These are described delow:

- 5726 Merriam Dr, Merriam, KS
  In February 2023 railroad ties and debris were encountered while advancing soil boring ADU-212 (see Appendix F, Figure F-2 for location). During advancing of a soil boring in February 2023, railroad ties and debris was encountered. The debris was described as having very poor strength properties as well has having a foul odor. The low strength properties may impact the foundation of the proposed levee and due to the foul odor would not be suitable for the base of the retention basin. All debris removed as part of the levee inspection trench and excavation of the retention basin will need to be disposed of in a permitted landfill. It is recommended additional investigation be performed to determine the nature and extent of the debris to assess the impact to the project.

- 5920 Merriam Dr, Merriam, KS
  In February 2023, petroleum contamination was encountered at a depth of 10 to 11 feet while advancing Boring ADU-201. The source of this contamination is unknown, however a historical aerial photo from 1941 shows a turn in and exit to the site with a small structure. It is speculated this may have been an former gas station but no records could be found. The current use of the property is auto repair. A soil sample was collected and tested for Total Petroleum Hydrocarbon (TPH) and Volatile Organics (VOCs). The results are in Table 1. The only range exceeding the KDHE Tier 2 RSK cleanup level of 250 mg/kg was the Mid Range Hydrocarbons at 316 mg/kg. However, this result is below the Non-Residential Scenario of 350 mg/kg. The VOC analysis was non-detect, which could suggest this is an old spill and the underlying constituents that drive risk associated with TPH have degraded. Since this is a non-residential property, the contaminated soil would likely not need to be cleaned up. However, it would be good practice to segregate contaminated soil during excavation and dispose of it in a permitted landfill. Most petroleum contaminated soil is classified as a Special Waste and typically can be disposed of in a RCRA Subtitle D landfill, which is the same type of landfill that accepts household trash. Additional testing would be necessary to meet disposal facility requirements.

Facilities Handling Hazardous Waste

There are a number of facilities located along the Turkey Creek channel that handle or produce hazardous wastes. These facilities are regulated by USEPA. Although they handle hazardous wastes, there have been no reported spills or areas requiring cleanup that would have an impact on the project.

CONCLUSION/RECOMMENDATIONS

Based upon the findings from the records search performed, there are no known hazardous waste sites that will impact the proposed work. In addition, no solid waste facilities, such as former landfills, were identified at locations that would impact the project. There were several spills reported near Turkey Creek along the reach addressed by this DDR, but they were quickly remediated and should not have any significant impact to the project. There are six properties with current or former USTs, with the current status of the tanks unknown on two of those properties, 5560 Merriam Dr and 5745 Merriam Dr. There is no known existing contamination associated with any of the current or former tank locations, also some contamination was encountered and remediated when the tanks that were closed and removed.

Two areas of concern were discovered during a February 2023 soils investigations. Buried railroad tires and debris were discovered while advancing a boring on 5726 Merriam Dr. This may present foundation issues for the proposed levee along this reach as well as issues with construction of a retention basin proposed on this property. The other issue discovered during February 2023 soil investigations was petroleum contamination on 5920 Merriam Dr at a depth of 10 to 11 feet below the ground surface. Sample results indicated TPH levels above residential values for Mid Range Hydrocarbons but below non-residential levels. VOC analysis results were all non-detect, which suggests the source of contamination is very old and the underlying constituents driving the health-based TPH risk levels have degraded over time.

Overall, the risk of encountering significant environmental/HTRW impacts during construction appear relatively low based on Federal and State environmental records. However, there remains uncertainty about activities that occurred prior to Federal/State regulations being enacted and records kept. An example of this is the debris encountered at 5726 Merriam Dr and the petroleum contamination at 5920 Merriam Dr, both discovered during subsurface investigations performed in February 2023. Neither of these sites were expected to have contamination issues based on environmental records. It is not possible nor cost effective to identify all potential environmental subsurface issues along a greater than one mile stretch of the channel addressed in this DDR. The density of borings and other investigative techniques would be both schedule and cost prohibitive. Therefore, it is recommended that the Contractor be required to develop contingency plans to address unexpected environmental conditions and contingency funding be set aside so unforeseen conditions can be addressed without significant project delays.

As for the two recently discovered sites with environmental concerns, 5726 Merriam Dr. and 5920 Merriam Dr, it is recommended additional investigations be performed to more fully characterize the nature and extent of the

environmental issues on these properties. This will help to determine the engineering measures that should be included in the design and help refine cost impacts to the project. For the debris at 5726 Merriam Dr, it is likely all debris below the proposed features will need to be removed and disposed at a permitted facility. For the petroleum contamination encountered at 5920 Merriam Dr, based on the TPH levels, it does not appear any remedial activities would need to occur. However, it is good practice to segregate any contaminated soil from clean soil excavated and dispose of it at a permitted landfill rather than reusing the soil on site or at other project locations. In this case it would likely be classified as a Special Waste that could be disposed of in a RCRA Subtitle D landfill, which is the type of landfill that accepts typical household waste.

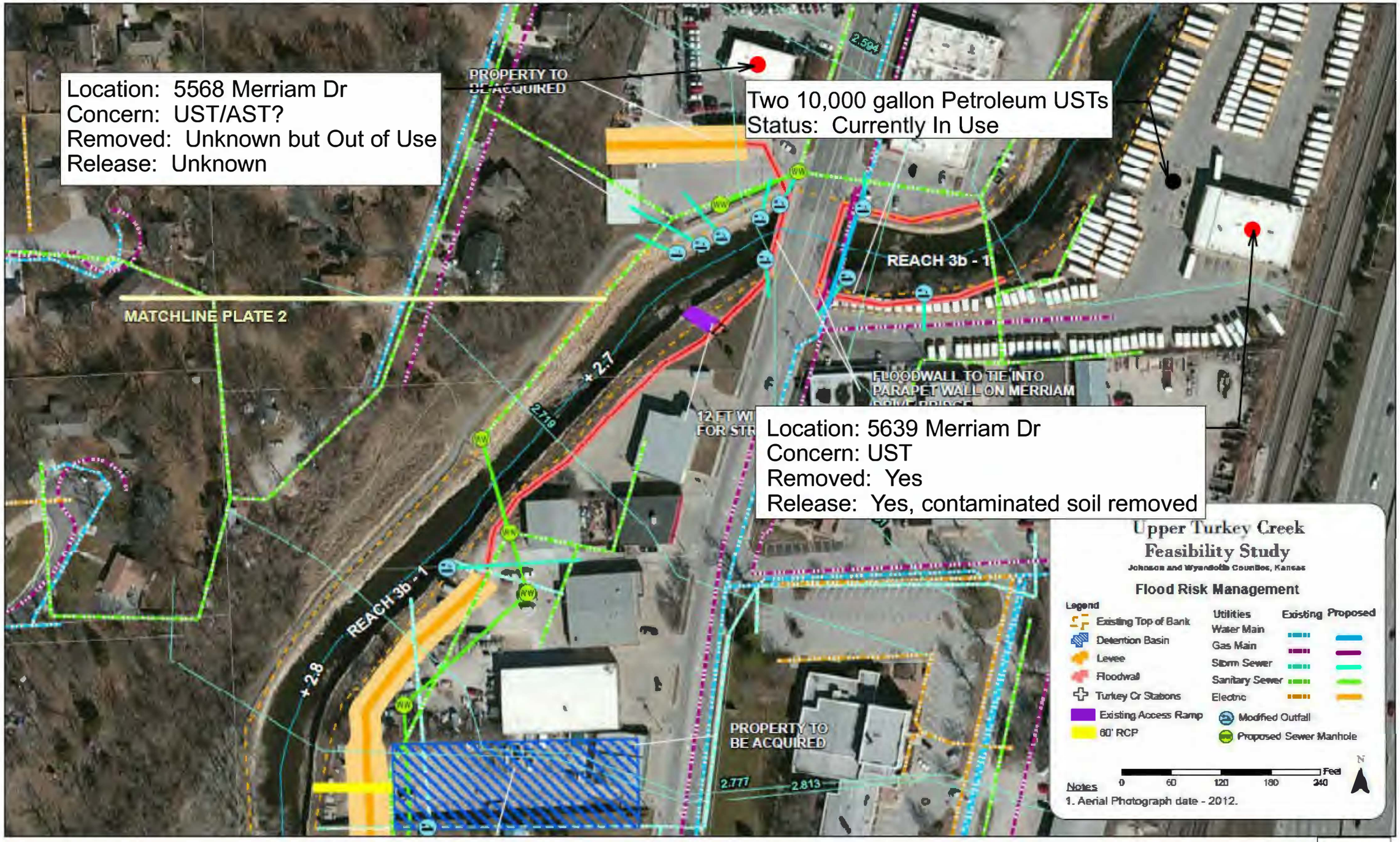

Location: 5568 Merriam Dr
Concern: UST/AST?
Removed: Unknown but Out of Use
Release: Unknown
PROPERTY TO BE ACQUIRED
Two 10,000 gallon Petroleum USTs
Status: Currently In Use
REACH 3b - 1
MATCHLINE PLATE 2
FLOODWALL TO TIE INTO PARAPET WALL ON MERRIAM
Location: 5639 Merriam Dr
Concern: UST
Removed: Yes
Release: Yes, contaminated soil removed
REACH 3b - 1
PROPERTY TO BE ACQUIRED
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure F-1

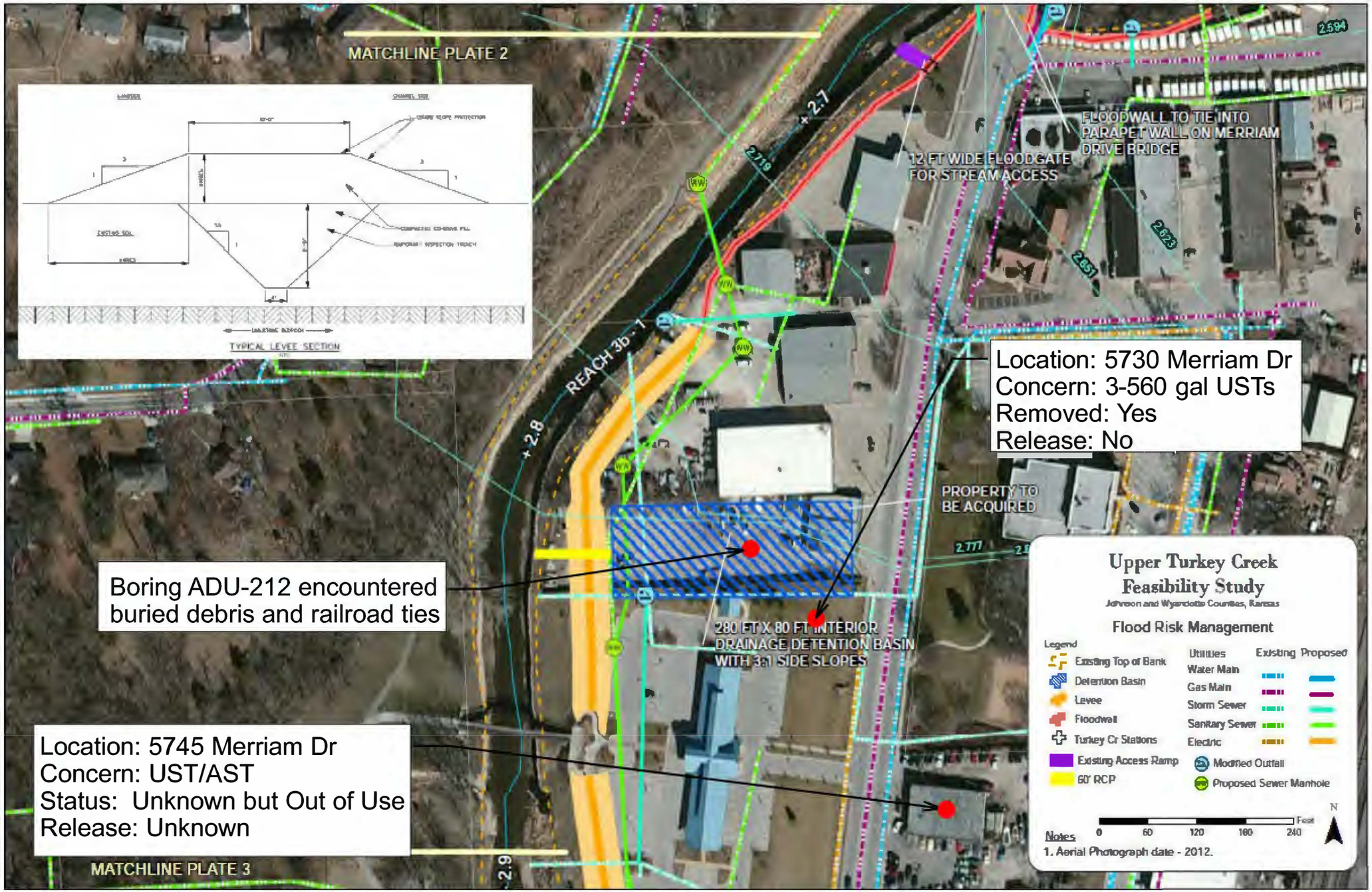

MATCHLINE PLATE 2
TYPICAL LEVEE SECTION
FLOODWALL TO TIE INTO PARAPET WALL ON MERRIAM DRIVE BRIDGE
12 FT WIDE FLOODGATE FOR STREAM ACCESS
REACH 3b-1
2.7
2.8
2.9
2.719
2.673
2.651
2.594
Location: 5730 Merriam Dr
Concern: 3-560 gal USTs
Removed: Yes
Release: No
PROPERTY TO BE ACQUIRED
Boring ADU-212 encountered buried debris and railroad ties
280 FT X 80 FT INTERIOR DRAINAGE DETENTION BASIN WITH 3:1 SIDE SLOPES
Location: 5745 Merriam Dr
Concern: UST/AST
Status: Unknown but Out of Use
Release: Unknown
MATCHLINE PLATE 3
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
N
Feet
0
60
120
180
240
Notes
1. Aerial Photograph date - 2012.

Figure F-2

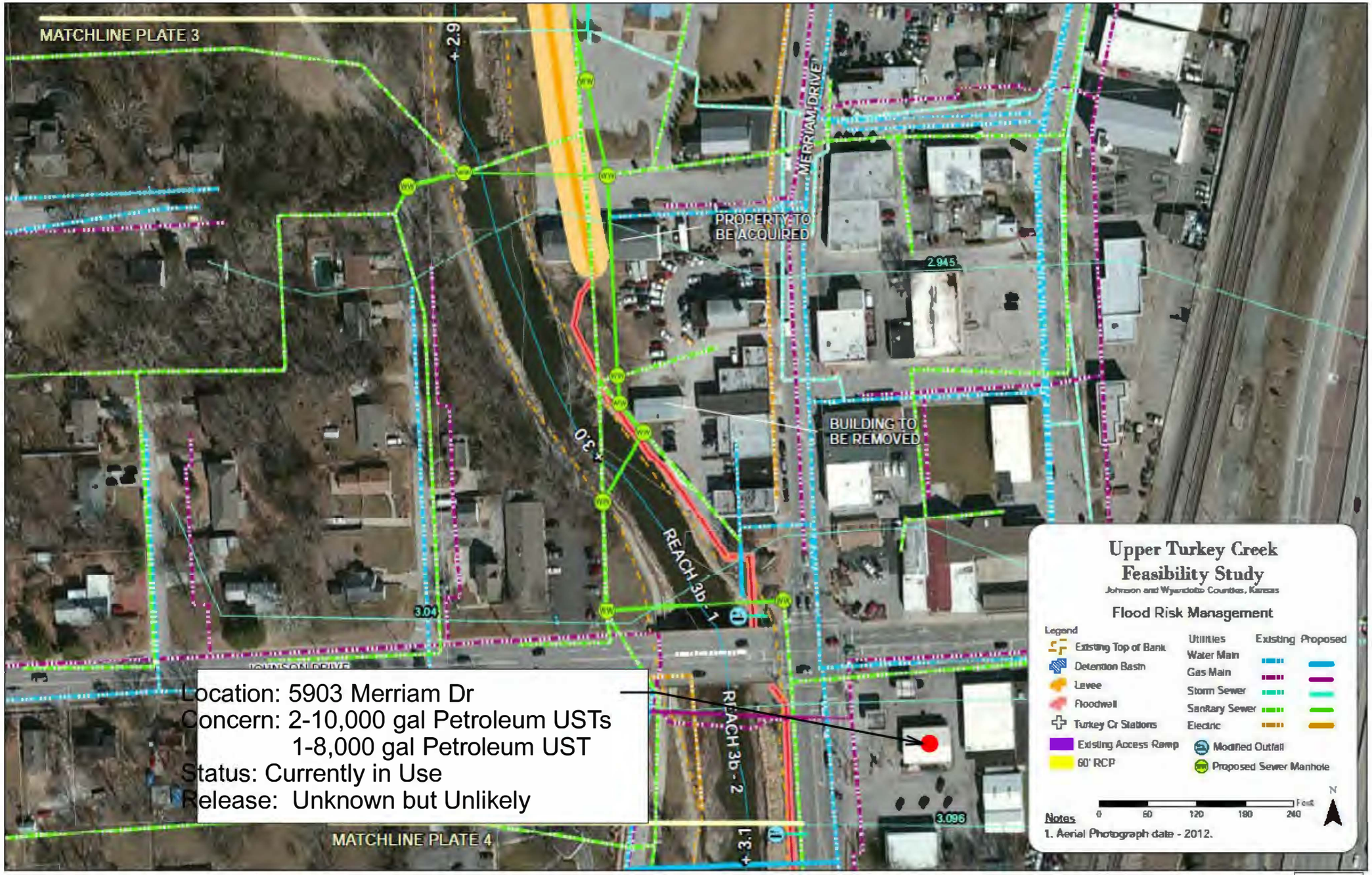

MATCHLINE PLATE 3
+ 2.9
MERRIAM DRIVE
PROPERTY TO BE ACQUIRED
2.945
BUILDING TO BE REMOVED
+ 3.0
REACH 3b - 1
3.04
JOHNSON DRIVE
REACH 3b - 2
+ 3.1
3.096
MATCHLINE PLATE 4
Location: 5903 Merriam Dr
Concern: 2-10,000 gal Petroleum USTs
1-8,000 gal Petroleum UST
Status: Currently in Use
Release: Unknown but Unlikely
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
0
60
120
180
240
Feet
N
Notes
1. Aerial Photograph date - 2012.

Figure F-3

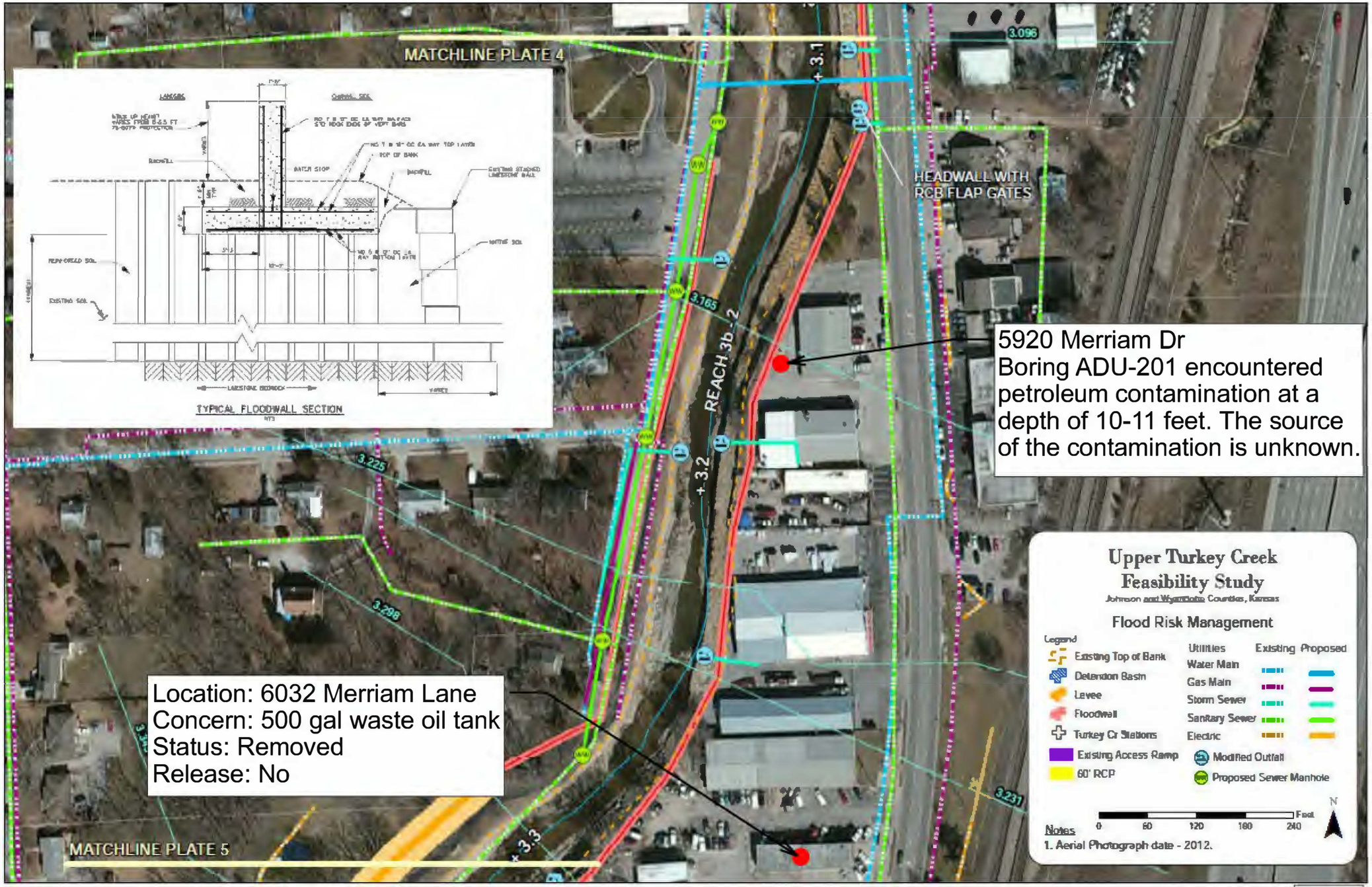

MATCHLINE PLATE 4
TYPICAL FLOODWALL SECTION
HEADWALL WITH RCB FLAP GATES
REACH 3b_2
3.165
3.225
3.298
3.231
+ 3.1
+ 3.2
+ 3.3
3.096
5920 Merriam Dr
Boring ADU-201 encountered petroleum contamination at a depth of 10-11 feet. The source of the contamination is unknown.
Location: 6032 Merriam Lane
Concern: 500 gal waste oil tank
Status: Removed
Release: No
MATCHLINE PLATE 5
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure F-4

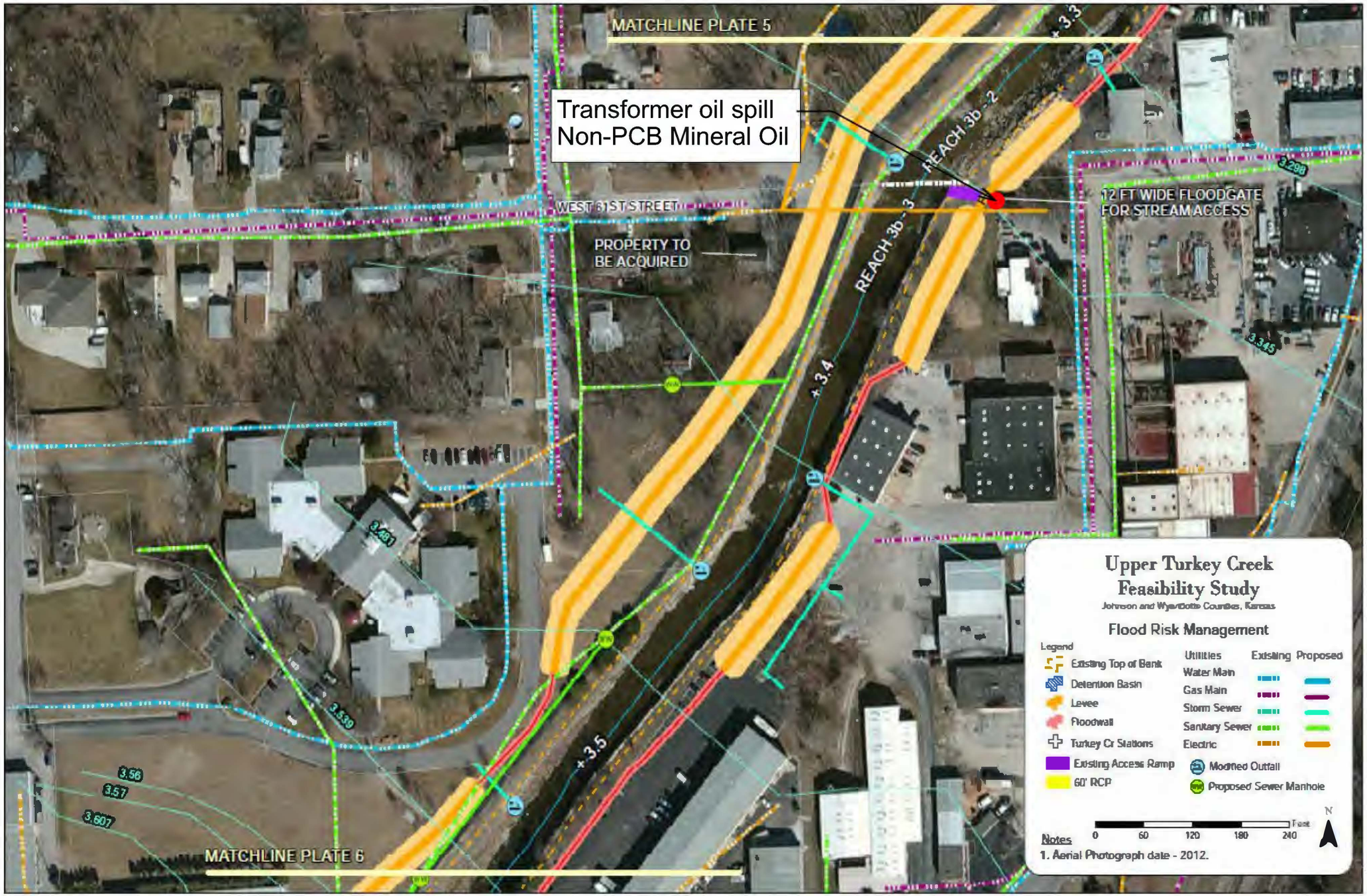

MATCHLINE PLATE 5
Transformer oil spill
Non-PCB Mineral Oil
WEST 61ST STREET
PROPERTY TO
BE ACQUIRED
REACH 3b - 2
REACH 3b - 3
12 FT WIDE FLOODGATE
FOR STREAM ACCESS
+3.3
+3.4
+3.5
3.298
3.345
3.481
3.539
3.56
3.57
3.607
MATCHLINE PLATE 6
Upper Turkey Creek
Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure F-5

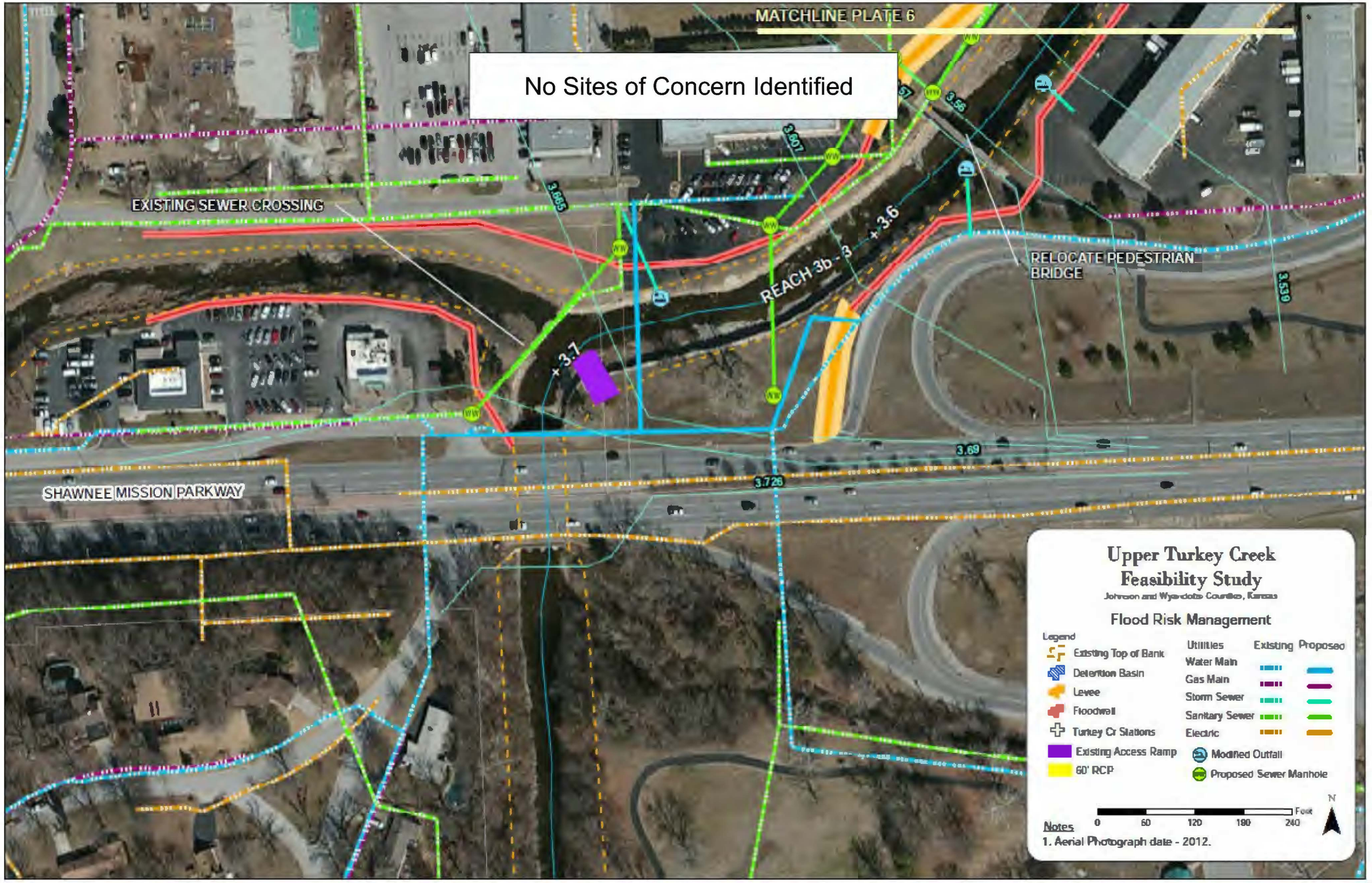

MATCHLINE PLATE 6
No Sites of Concern Identified
EXISTING SEWER CROSSING
SHAWNEE MISSION PARKWAY
REACH 3b - 3
+3.6
+3.7
RELOCATE PEDESTRIAN BRIDGE
3.665
3.607
3.56
3.539
3.69
3.726
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
N
Feet
0
60
120
180
240
Notes
1. Aerial Photograph date - 2012.

Figure F-6

This page was intentionally left blank for duplex printing.

APPENDIX G – PRELIMINARY DDR (FEASIBILITY VALIDATION REPORT)

This page was intentionally left blank for duplex printing.

US Army Corps of Engineers ®
Kansas City District

Upper Turkey Creek Flood Risk Management Project

P2 Number: 505229

Merriam, Kansas

Preliminary Design Document Report

June 2023

THIS PAGE IS INTENTIONALLY BLANK.

TABLE OF CONTENTS

Section 1—Introduction 1
1.1 Purpose 1
1.2 Location 1
1.3 Protected Area Description 3
1.4 Feasibility Study 3
Section 2—Feasibility Study Recommended Plan Summary 7
2.1 Hydrology and Hydraulics 7
2.2 Levee and Floodwall 8
2.3 Bridge Modifications 11
2.4 Utilities 11
2.4.1 Storm 12
2.4.2 Sanitary 13
2.4.3 Domestic Water 14
2.4.4 Electrical 14
2.4.5 Natural Gas 15
2.5 Detention 15
Section 3—Feasibility Study Recommended Plan Modifications 18
3.1 Bridges 18
3.1.1 Merriam Drive Bridge (Reach 3b-1) 18
3.1.2 Farmer's Market Pedestrian Bridge RM 2.875 18
3.1.3 Pedestrian Bridge at RM 3.57 18
3.2 Detention 18
3.3 Hydrology and Hydraulics 18
3.3.1 Hydrology 19
3.3.2 Hydraulics 19
3.4 Storm Drainage Structures 19
Section 4—Feasibility Study Recommended Plan Preliminary Analysis 24
4.1 Hydrology and Hydraulics 24
4.2 Climate Change Analysis 26
4.3 Underseepage 26
4.4 Slope Stability 26
4.4.1 Architect-Engineer Feasibility Stability Analyses 26
4.4.2 U.S. Army Corps of Engineers Feasibility Stability Analyses 27
4.4.3 Current Analyses 28
4.4.4 Future Analyses 29
4.5 Settlement 29

4.5.1 Current Analyses 29
4.5.2 Future Analyses 29
4.6 Structures 30
4.6.1 Alternative—Secant Pile Floodwall 30
4.6.2 Stability 30
4.6.3 Strength Design Parameters 31
4.7 Bridges 31
4.7.1 Merriam Drive Bridge RM 2.636 (Reach 3b-1) 31
4.8 Utilities 31
4.8.1 Crossings 32
4.8.2 Uplift 32
4.9 Roadways 33
4.10 Site Access Considerations 34
4.10.1 Access Roads 34
4.10.2 Access Ramps 34
4.10.3 Construction Staging Areas 35
4.10.4 Access to Public Parks and Trails 35
4.10.5 Access to Private Property 36
4.10.6 Traffic Control 36
4.11 Borrow 37
4.12 Real Estate 37
4.13 Hazardous, Toxic, and Radioactive Waste 37
4.14 Environmental Considerations 38
4.15 Preliminary Cost Estimate Update 39
4.16 Additional Data Requirements 39
4.16.1 Topographic and Real Estate Survey 39
4.16.2 Subsurface Investigation 39
4.16.3 Laboratory Testing of Soil Samples 39
Section 5—Criteria 42
5.1 U.S. Army Corps of Engineers Criteria 42
5.2 Federal and State Criteria 43
List of Acronyms and Abbreviations 46
Appendix A—Plates from Feasibility A-1
Appendix B—Geotechnical B-1
Appendix C—Civil C-1
Appendix D—Hazardous, Toxic, and Radioactive Waste D-1
Appendix E—Hydraulics and Hydrology E-1
Appendix F—Environmental F-1
Appendix G—Typical Floodwall G-1

Appendix H—Cost Estimate H-1

TABLES

Table 2-1. 24-hour Rainfall Totals per Annual Exceedance Probability 8
Table 2-2. Linear Feet of Levee and Floodwall by Reach 9
Table 2-3. Breakdown of Approximate Levee and Floodwall Heights by Reach 10
Table 2-5. Utility Relocation Summary 12
Table 2-6. Stormwater Structure Modifications per Feasibility Study Recommended Plan 13
Table 2-7. Proposed Sanitary Relocation per Feasibility Study 13
Table 2-8. Proposed Domestic Water Relocations per Feasibility Study 14
Table 2-9. Proposed Electrical Relocation per Feasibility Study 14
Table 2-10. Proposed Natural Gas Relocations per Feasibility Study 15
Table 3-1. Closure Type Requirements[1] 19
Table 3-2. Storm Drainage Modifications to Feasibility Study Recommended Plan 20
Table 3-3. Approximate Storm Drainage Quantities 21
Table 4-1. 90 Percent Reliability Level of Protection per Table 3-7 of the Feasibility Study, and the Included Levee Profile Elevations in the Final Feasibility Study HEC-RAS Modeling 25
Table 4-2. Sudden Drawdown Analysis Results 28
Table 4-3. Top of Levee Loading Stability Criteria[1] 30
Table 4-4. Top of Levee Loading Strength Criteria 31
Table 4-5. Criteria for Pipelines Crossing Levees* 32
Table 4-6. Anticipated Roadway Modifications 33
Table 4-7. Access Roads 34
Table 4-8. Access Ramp Considerations 35
Table 4-9. Trail Segments Proposed for Replacement 36
Table 5-1. Engineering Manuals 42
Table 5-2. Engineering Regulations 42
Table 5-3. Engineering and Construction Bulletin 43
Table 5-4. State of Kansas Criteria 43
Table 5-5. City of Merriam, Kansas, Criteria 43
Table 5-6. American Concrete Institute 43
Table 5-7. American Institute of Steel Construction 43
Table 5-8. American Society of Civil Engineers 43
Table 5-9. American Public Works Association 44
Table 5-10. National Oceanic and Atmospheric Administration 44

FIGURES

Figure 1-1. Upper Turkey Creek—Merriam, Kansas Location 2
Figure 1-2. Feasibility Recommended Plan 5

SECTION 1
Introduction

1.1 PURPOSE

The purpose of this report is to evaluate the recommended plan from Upper Turkey Creek Johnson County and Wyandotte County, Kansas Flood Risk Management Project Feasibility Report with Integrated Environmental Assessment (2015), also referred to as the Feasibility Study (FS). Changes in criteria and in existing conditions impacting the FS recommended plan will also be evaluated. The findings contained within this report are preliminary and rely heavily on engineering judgment and lessons learned from studies and designs completed on similar projects after the 2015 FS was approved.

1.2 LOCATION

The Upper Turkey Creek Flood Risk Management (FRM) project is a new FRM project located in the City of Merriam, in Johnson County, Kansas. Turkey Creek is a right bank (RB) tributary of the Lower Kansas River located in Johnson and Wyandotte counties, Kansas between approximate river miles (RMs) 2.7 to 3.7. Figure 1-1 shows the location of the Upper Turkey Creek FRM project.

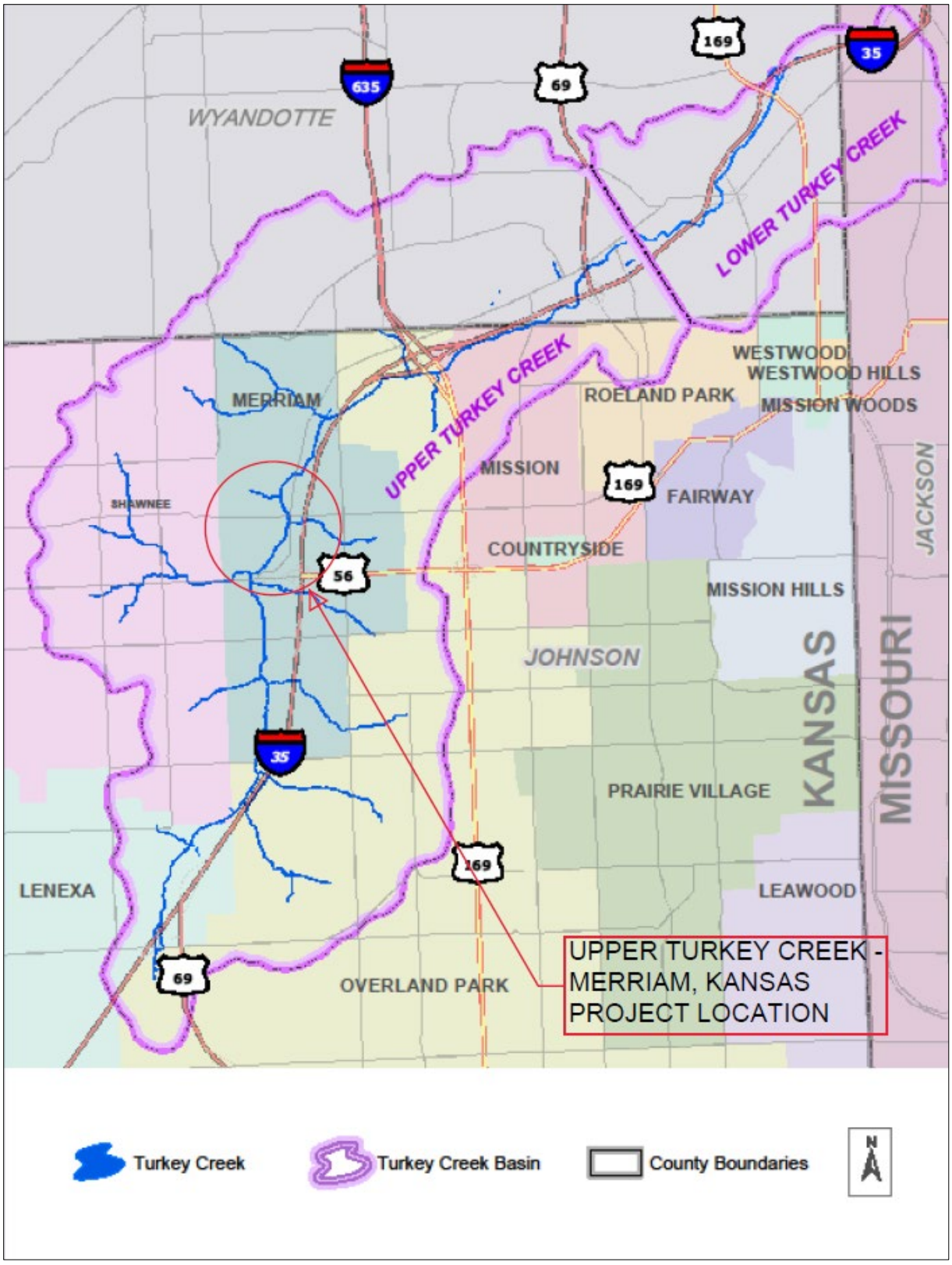


Figure 1-1. Upper Turkey Creek—Merriam, Kansas Location

1.3 PROTECTED AREA DESCRIPTION

The watershed contains some of the most intensely developed urban locations in Johnson and Wyandotte counties, Kansas. Approximately 75 percent of the Turkey Creek watershed consists of residential, commercial, and industrial land use. Most of Turkey Creek and its tributaries have been channelized.

The need for the project lies in the potential for flooding that could result in life loss (LL) and/or property damage in the Upper Turkey Creek Basin. The majority of the Turkey Creek channel that passes through the City of Merriam, Kansas, can contain a 20 percent annual exceedance probability (AEP) (1/5-year event), although two areas flood at the 50 percent AEP (1/2-year event). Flooding in and around Johnson County, Kansas, led to presidential disaster declarations in 1993 and 1998. The estimated cost of these disasters exceeded $50 million. Heavy rains on October 4, 1998, produced flooding that caused several million dollars in damage to businesses and public property along Turkey Creek in Johnson and Wyandotte counties, Kansas. Reports indicate that parts of the Kansas City metropolitan area in both Kansas and Missouri received almost eight inches of rain that day with some locations receiving three to five inches of rain within a three-hour period. More than five inches fell in Lenexa, Kansas, in a little more than an hour. Nearly 100 calls for water rescue were received, and numerous roads throughout the area, including individual sections of Interstate 35 (I-35) and Interstate 435 (I-435), were impassible. Following the 1993 and 1998 flood losses and as Lower Turkey Creek investigations were ongoing, the U.S. House of Representatives Committee on Transportation and Infrastructure passed the 1999 resolution and adopted the Upper Turkey Creek Basin Study authority in 2000.

1.4 FEASIBILITY STUDY

The basis of this project is the recommended plan from the 2015 FS. Most of the protected area is on the RB of Turkey Creek upstream of an authorized FRM project in the downstream 8,700 feet of the Turkey Creek channel, also locally known as Lower Turkey Creek. The Upper Turkey Creek reach includes the main downtown reach of Merriam, Kansas, as well as a commercial and industrial area just south of Johnson Drive. The features, which include earthen levees, floodwalls, and a detention basin, extend upstream to downstream from Shawnee Mission Parkway to Merriam Drive. Figure 1-2 shows the recommended feasibility plan. Unless otherwise specified, consistent with EM 1110-2-6056, “Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums”, dated 31 December 2010, all elevations presented in this report are in the North American Vertical Datum of 1988 (NAVD 88).

THIS PAGE IS INTENTIONALLY BLANK.

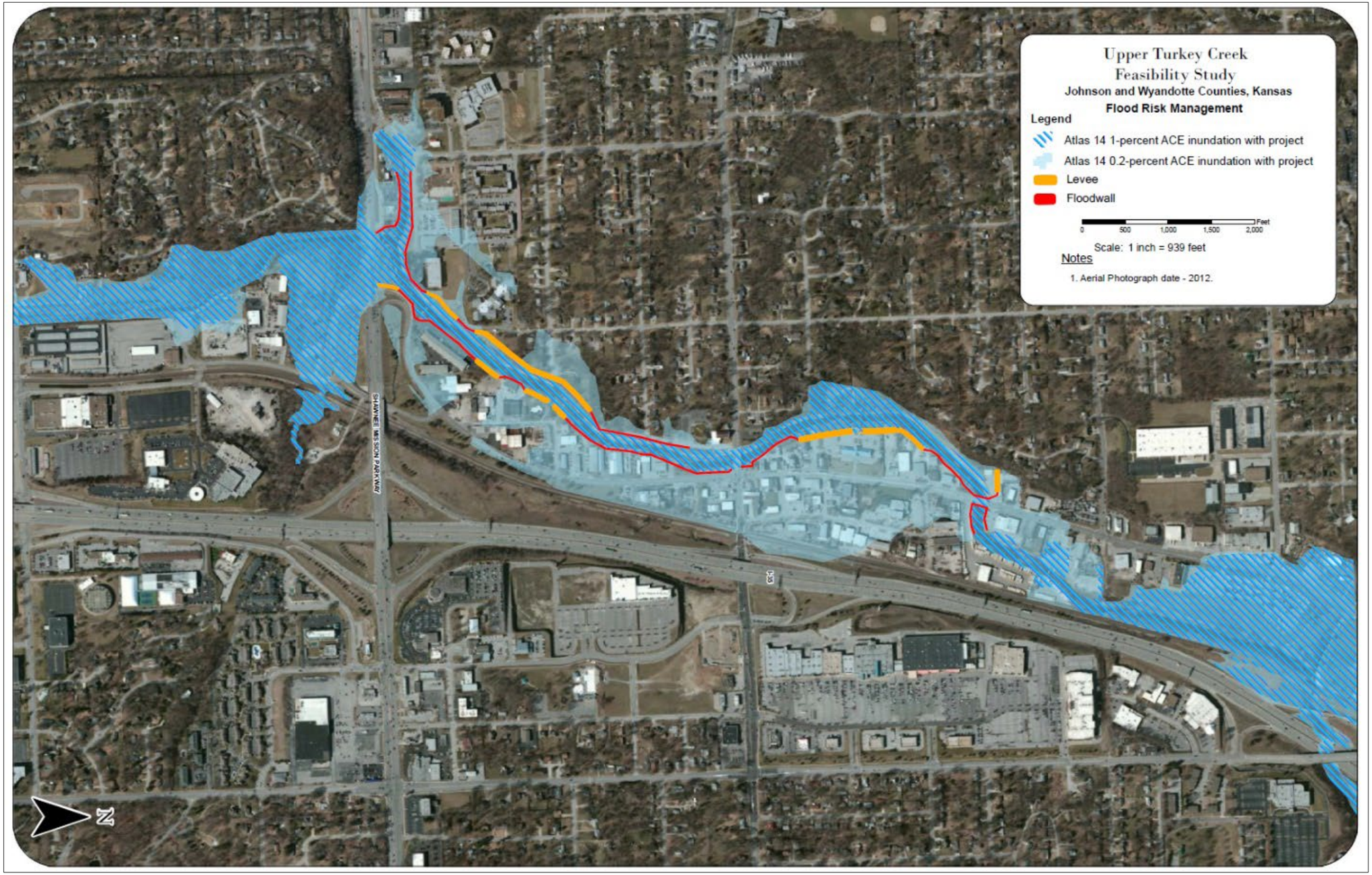


Figure 1-2. Feasibility Recommended Plan

THIS PAGE IS INTENTIONALLY BLANK.

SECTION 2
Feasibility Study Recommended Plan Summary

The 2015 FS recommended plan was formulated and evaluated using updated National Oceanic and Atmospheric Administration (NOAA) Atlas 14 rainfall data. The primary plan features are 3,380 feet of levee up to approximately 6 feet in height and 6,820 feet of floodwall up to approximately 6.5 feet in height. A 2.1-acre-foot, gravity-drained stormwater detention basin is included for interior drainage and is located on the RB adjacent to the Merriam Marketplace near the downstream end of the project. The project also includes stormwater, water and sanitary sewer relocations, and environmental compensatory mitigation of seven acres of mast-producing trees.

2.1 HYDROLOGY AND HYDRAULICS

The hydrologic basis for the FS was based on a HEC-1 model developed by the Larkin Group Inc. (Larkin) for the Johnson County, Kansas, Storm Management Program. The HEC-1 model included the complete watershed contributing to the upper portion of Turkey Creek. Due to the very limited period of record of the Storm Watch System gages in the Turkey Creek watershed at the time of the modeling, and because there is no discharge gaging on Turkey Creek, a gage on Brush Creek was used as an analog for validation of the HEC-1 model of Turkey Creek for the 1998 hydrologic event. It was estimated that the 1998 event exceeded a 1 percent AEP (1/100-year) discharge for Brush Creek, and the hydrologic modeling parameters were adjusted using the observed precipitation within the Brush Creek watershed to calibrate hydraulic modeling with the observed high water marks. Due to the lack of observed precipitation data within the Turkey Creek watershed, a similar correlation of the 1 percent AEP (1/100-year) discharge with the observed high water marks was assumed for Turkey Creek as a validation of adopted hydraulic modeling parameters.

Due to the active development within the Upper Turkey Creek watershed, and the lack of gaging, calibration with previous hydrologic events in 1961, 1977, 1993, and 1998 were not performed. The hydrologic model was completed in 2005 and was generated to support a floodplain mapping initiative in Johnson County, Kansas.

The original Upper Turkey Creek hydrologic analysis was based on NOAA Technical Paper (TP)-40, Rainfall Frequency Atlas of the Eastern United States for Duration from 30 minutes to 24 hours and Return Periods from 1 to 100 Years. An update to the in-progress (at the time) Upper Turkey Creek Feasibility Report was performed in 2014 based on the publishing of the NOAA Atlas 14 point precipitation frequency estimates which were published in 2013. When these updates were made to the modeling, the HEC-1 model was imported into Hydrologic Modeling System (HEC-HMS) version 4.0. Slight edits to the modeling inputs were required to import the model into HEC-HMS, but the extent of these edits was not documented in the feasibility report. Table 4-1 presents the respective TP-40 and Atlas 14 24-hour adopted point precipitation frequencies and annual maximum rainfall totals evaluated during the FS.

The hydraulic modeling for the FS was based upon a River Analysis System (HEC-RAS), version 3.1.3, one-dimensional (1D) steady flow model.

Table 2-1. 24-hour Rainfall Totals per Annual Exceedance Probability

AEP (percentage)	Average Return Period (years)	24-hour Rainfall Amount TP-40/Atlas 14 (inches)
50	2	3.6/3.7
20	5	4.5/4.7
10	10	5.3/5.5
4	25	6.2/6.8
2	50	6.9/7.8
1	100	7.8/8.8
0.5	200	8.4/9.9
0.2	500	9.3/11.4

2.2 LEVEE AND FLOODWALL

The recommended plan consists of construction of levee and floodwall sections along Turkey Creek in the City of Merriam, Kansas, from Shawnee Mission Parkway on the south (upstream end) to Merriam Drive just north of West 57th Street on the north (downstream end). Most of the protected area is on the RB of Turkey Creek. The levee is up to 6 feet in height and includes a homogeneous clay embankment with a 10-foot crest width and 1V:3H side slopes. A floodwall up to 6.5 feet in height will be constructed where there is insufficient space to construct a levee to the design elevation or insufficient space for a minimum slope of 1V:2H from the creek bed to the riverside levee toe.

The floodwall consists of an inverted T-wall design with a base of 10 feet. The FS plan indicates that a key will not be necessary for sliding as the base will be entrenched within the reinforced soil on the landside of the floodwall. Reinforced concrete with a minimum compressive strength of 4,000 pounds per square inch (psi) compressive and stem and footing thicknesses of 18 inches will provide the necessary strength capacity needed to withstand the 1 percent AEP (100-year event) flows with an estimated assurance (conditional non-exceedance probability) of 82.9 percent (floodwall varying 0–6.5 foot tall) level of protection referred to as the Final NED Plan.

The existing Turkey Creek channel in the project area has approximately 1V:1H to 1V:2H side slopes and is lined with stacked limestone blocks. The channel is generally less than 8 feet in depth. The limestone block retaining wall was built in sections from the 1960s to the 1980s and sections of it have been repaired since. The wall is maintained by the Merriam Drainage District (MDD). MDD inspects this area of the creek twice a year. They estimate that ten percent of the original wall has been replaced.

The limestone block retaining wall is a gravity block wall with no reinforcements tying the blocks together. The individual blocks are approximately 6 feet long, 2 feet deep, and 14 to 18 inches tall. The face of each block in the stack is offset approximately 2 inches back from the face of the block below it. There is no drain behind the wall. The wall was backfilled with local soil which appears to be mainly clay. In some recent replacement sections of wall, filter fabric and aggregate backfill were placed behind the wall.

The wall experiences regular breaches with short sections (roughly 100 feet long or less) completely collapsing into the creek (see Section 4.0 of Appendix B-5 of the FS). The cause of the breaches is unknown, but the wall was described in the FS as, “not an engineered structure”. When breaches occur, MDD rebuilds the section with the original limestone blocks. During site visits for the FS, soil was observed migrating through the limestone blocks. The walls were judged to not be capable of providing support to the proposed levee or floodwall.

The recommended plan includes ground improvement in floodwall areas where the projected slope between the proposed floodwall heel and creek bed will be steeper than 1V:2H, totaling approximately two-thirds of the total length of floodwall. No ground improvement is proposed in the leveed areas.

The recommended ground improvement consists of 16-inch diameter, concrete, soil replacement columns extending to the top of bedrock. The columns will be placed approximately 2 feet on-center in an equilateral triangle pattern. The recommended width of ground improvement is 15 feet, centered on the floodwall footing. This ground improvement, consisting of more than 12,000 columns, is intended to provide a floodwall foundation that does not rely on the stability and support of the existing limestone-block retaining wall. The recommended plan is based on the MDD, continuing to repair breached segments of the limestone-block retaining wall so there will not be permanent ongoing scour of the improved ground supporting the floodwall. Discussions will be held with MDD as the design progresses to clarify the sponsor responsibilities and timeframe for repairs.

Table 2-2 details the levee and floodwall quantity breakdown in linear feet. Reach 3b-1 is the Merriam, Kansas downtown area, Reach 3b-2 is the Industrial and Railway Drive (West 61st Street), and Reach 3b-3 is the Parkway Vicinity. Table 2-3 details the extents and locations of the floodwall and levee reaches through the project reach, as well as approximate average heights based on the 2020 light-detection and ranging (LiDAR) surface as compared to the levee profile indicated in the FS final HEC-RAS model. It is noted that the lengths indicated in Table 2-3 are not precisely consistent with those illustrated in GIS files, and that the average heights are based on the reaches delineated in the GIS files.

Table 2-2. Linear Feet of Levee and Floodwall by Reach

Left Bank (LB)	**Reach 3b-1 (feet)**	**Reach 3b-2 (feet)**	**Reach 3b-3 (feet)**
Levee	168	290	1,064
Floodwall	235*	840*	670* and 1,070**
Right Bank (RB)			
Levee	1,051	150	660
Floodwall	1,047* and 300**	1,390**	1,410** and 330*

*Auger Cast Piles (ACP)
**Shallow Footings (SF)

Table 2-3. Breakdown of Approximate Levee and Floodwall Heights by Reach (sheet 1 of 2)

Levees/Floodwalls	Construction Feature(s)
LB	Reach 3b-1—Merriam Downtown • 160 feet of floodwall average height of 5 to 6 feet, downstream of Merriam Drive • 75 feet of floodwall average height of 5 to 6 feet upstream of Merriam Drive • 168 feet of levee average height of 4 feet upstream of Merriam Drive Reach 3b-2—Industrial and Railway Drive (West 61st Street): • 840 feet of floodwall average height of 4 to 6 feet begins at 300 feet upstream of Johnson Drive to 500 feet upstream of West 60th Street • 290 feet of levee average height of 5 feet from 500 feet upstream of West 60th Street to West 61st Street Reach 3b-3—Parkway Vicinity: • 744 feet of levee average height of 3 to 5 feet from West 61st Street to 70 feet downstream of West 62nd Street • 200 feet of floodwall average height of 3 to 5 feet from 70 feet downstream of West 60th Street to 20 feet upstream of West 62nd Street • 320 feet of levee average height of 3 feet from 20 feet upstream of West 62nd Street to 60 feet upstream of the pedestrian bridge (north side of Skate World parking lot) • 1,070 feet of floodwall average height of 3 feet from 60 feet upstream of pedestrian bridge to south side of Skate World parking lot then along west 62nd Terrace

Table 2-4. Breakdown of Approximate Levee and Floodwall Heights by Reach (sheet 2 of 2)

Levees/Floodwalls	Construction Feature(s)
RB	Reach 3b-1—Merriam Downtown: • 220 feet of floodwall average height of 5 to 6 feet downstream of Merriam Drive to Merriam Drive parapet wall • 532 feet of floodwall average height of 5 to 6 feet from Merriam Drive parapet wall to West 57th Street • 1,051 feet of levee average height of 6 feet from West 57th Street to 180 feet south of the farmer's market parking lot • 595 feet of floodwall average height of 6 feet downstream of Johnson Drive Reach 3b-2—Industrial and Railway Drive (West 61st Street): • 1,390 feet of floodwall average height of 4 to 6 feet south of Johnson Drive to 70 feet north of West 61st Street • 150 feet of levee average height of 5 feet to West 61st Street Reach 3b-3—Parkway Vicinity: • 240 feet of levee average height of 3 to 5 feet from West 61st Street to 190 feet north of West 62nd Street • 290 feet of floodwall average height of 5 to 6 feet from 190 feet north of West 62nd Street to West 62nd Street • 240 feet of levee average height of 5 to 6 feet from 62nd Street to 240 feet south of West 62nd Street • 890 feet of floodwall average height of 4 feet from 240 feet south of West 62nd Street to 130 feet north of Shawnee Mission Parkway • 180 feet of levee average height of 5 feet to Shawnee Mission Parkway • 560 feet of floodwall average height of 4 feet on west side of Turkey Creek extending to north of Shawnee Mission Parkway and running along south side of drainage ditch that runs parallel to West 62nd Terrace

2.3 BRIDGE MODIFICATIONS

The recommended plan constructing a 4- to 6-foot, reinforced concrete headwall on the upstream and downstream sides of the Merriam Drive Bridge (Reach 3b-1), respectively, to maintain flows in the creek channel and reduce overtopping of the bridge during higher intensity storm events. The Shawnee Mission Parkway bridge was assumed to overtop to minimize induced damages upstream as stated in the FS.

The pedestrian bridge located at RM Station 3.57 (Reach 3b-3) requires an extension to span 175 feet across the new levee and floodwall system. It is expected the extension will consist of additional span(s) to the bridge and a possible raise of the abutments.

2.4 UTILITIES

The FS investigated known utilities within the recommended plan's project limits. All the storm, sanitary, domestic water, electrical, and natural gas facilities identified in the FS that cross a proposed levee or floodwall have recommended relocations and/or modifications for the project. Table 2-5 summarizes the general recommendations according to the utility. Information specific to each utility type is detailed in the following subsections.

Table 2-5. Utility Relocation Summary

Utility	Existing Facility Type	General Recommended Relocation Crossing Levee Embankment or Floodwall
Sanitary Sewer	12-inch polyvinyl chloride (PVC) (assumed)	• Remove existing sanitary pipe and replace with 12-inch reinforced concrete pipe (RCP). • Install manholes on landside.
Storm	Varies	• Install manhole on landside. • Install gate box on riverside with flap gate. • If installation of riverside gate box is not feasible, install flap gate at floodwall or flap gate at headwall. • Remove existing storm pipe and replace with minimum 24-inch RCP under levee/floodwall.* • Grout-fill pipes to abandon if removal is not feasible.
Water	6-inch and 8-inch ductile iron pipe (DIP) (assumed)**	• Remove and replace with DIP. • Encase any waterline passing through the channel with concrete encasement.
Electric	Overhead electric	Relocate vertically to provide adequate clearance.
Natural Gas	PE (assumed)	Remove polyethylene (PE) pipes and replace with PE pipes.***

*FS indicated the Civil Design and Geotechnical Sections determined a minimum of 24-inch RCP is acceptable. This is a deviation from the minimum 48-inch RCP prescribed in Kansas City District guidance under urban levees. The use of 24-inch RCP requires inspection of these structures with video equipment in place of manual inspection.
**FS indicated limited information is known about the construction material used in existing water lines within the flood corridor. Additional investigations will be performed during the design phase to obtain more information on the existing water lines.
***FS indicated Turkey Creek high water events have durations of a few hours. During feasibility, geotechnical engineering judgement determined underseepage landward of floodwalls/levees is not a concern. If underseepage is not a concern, PE pipe does not risk flotation.

2.4.1 Storm

The recommended plan within the FS identified 44 existing stormwater lines within the project area. A total of 27 locations where existing stormwater lines would cross under the proposed levee or floodwall were identified. See Appendix C for a summary of storm drainage structure existing versus proposed conditions. See Table 2-6 for a general summary of stormwater structure modifications per FS recommended plan.

The FS identified that the addition of proposed levees and floodwalls would require interior drainage management. Interior drainage alternatives including pump stations, underground detention (a concrete box with a series of inlets at ground level placed parallel to the levee), and surface detention were discussed in the FS. A surface detention basin was recommended in the FS recommended plan. Section 2.5 contains additional information on the surfaced detention, herein referred to as the detention basin.

Numerous private storm water lines are assumed to exist within the project area based on site observations of small diameter pipes daylighting into Turkey Creek. Private storm water lines were not

identified within the FS. Private storm water lines will be investigated and evaluated during the next design phase.

Table 2-6. Stormwater Structure Modifications per Feasibility Study Recommended Plan

Location	Stormwater Structure Modification
Landside	Install manhole on landside.
Riverside	Install gate box on riverside with flap gate. If installation of riverside gate box is not feasible, install flap gate at floodwall or flap gate at headwall.
Riverside	Install headwall and flap gates at the existing triple 7-foot by 7-foot reinforced concrete box (RCB) drainage structure at Station 3.109
Under proposed levee/floodwall	Remove existing storm pipe and replace with a minimum 24-inch RCP under levee/floodwall. Abandon grout fill pipes if removal is not feasible.

2.4.2 Sanitary

The recommended plan within the FS identified seven locations where sanitary relocations may be required. Approximately 4,190 feet of 12-inch PVC and 36 total 48-inch diameter manholes were proposed in the FS. Table 2-7 shows a summary of the proposed sanitary sewer relocations.

Table 2-7. Proposed Sanitary Relocation per Feasibility Study

Existing Facility Type	Approximate Station Location (Per FS Stationing)	Proposed Relocation Description	Proposed Approximate Relocation Length (feet)	Number of Proposed 48-inch Diameter Manholes
12-inch PVC	Station 3.664	Under LB floodwall, creek, and RB floodwall	330	3
12-inch PVC	Station 3.480	Under LB levee upstream to Station 3.630	970	8
12-inch PVC	Station 3.400	Under LB levee	130	2
12-inch PVC	Station 3.104	LB relocation upstream to Station 3.282	770	6
12-inch PVC	Station 3.045	Under RB floodwall and creek	220	3
12-inch PVC	Station 2.730	Under 3 creek crossings and RB relocations upstream to Station 3.010	1,670	12
12-inch PVC	Station 2.640	Under LB floodwall	100	2

2.4.3 Domestic Water

The recommended plan within the FS identified four locations where domestic water relocations may be required. The recommended plan assumes domestic water lines will be replaced with DIP based on discussions with the civil engineer and lead planner. Table 2-8 shows a summary of the proposed domestic water.

Table 2-8. Proposed Domestic Water Relocations per Feasibility Study

Existing Facility Type	Approximate Station Location (Per FS Stationing)	Proposed Relocation Description	Proposed Approximate Relocation Length (feet)
6-inch and 8-inch DIP	Station 3.610	Up and over floodwalls and levee	890
8-inch DIP	Station 3.100	Up and over floodwall. Use 8-inch restrained joint DIP. Concrete encased 8-inch DIP for waterline passing through the channel.	280
1-inch steel	Station 3.040	Excavate and replace with one-inch welded steel pipe	50
8-inch DIP	Station 2.623	Up and over the floodwalls with 8-inch restrained joint DIP. Line parallel to and behind the east Merriam Drive curb.	190

2.4.4 Electrical

The recommended plan within the FS identified one location where electrical line relocation may be required. Table 2-9 shows a summary of the proposed electrical relocation.

Table 2-9. Proposed Electrical Relocation per Feasibility Study

Existing Facility Type	Approximate Station Location (Per FS Stationing)	Proposed Relocation Description	Proposed Approximate Relocation Length (feet)
Electrical line	Station 3.350	Up and over LB and RB levees to provide adequate vertical clearance between power line and top of levees. Relocate horizontally on LB to accommodate levee alignment	425

2.4.5 Natural Gas

The recommended plan identified three locations where gas modifications may be required. Table 2-10 shows a summary of the proposed natural gas relocations.

Table 2-10. Proposed Natural Gas Relocations per Feasibility Study

Existing Facility Type	Approximate Station Location (Per FS Stationing)	Proposed Relocation Description	Proposed Approximate Relocation Length (feet)
2 2-inch PE pipes (PE assumed)	Station 3.190	If geotechnical engineering determines underseepage landward of floodwalls/levees is not a concern, then PE pipe does not risk flotation.	TBD
8-inch (material unknown)	Station 2.070*	Up and over the floodwall.	270
4-inch steel	Station 2.623	Up and over floodwall	50

*Station 2.070 gas relocate may be incorrect location in FS as proposed project limits are from approximate Station 3+73 to 2+60.

2.5 DETENTION

A 2.14 acre-foot grassed detention basin, with dimensions of 80 feet wide by 280 feet in length with 3H:1V side slopes and a 60-inch RCP outfall with flap gate, was proposed in the recommended plan to collect stormwater runoff originating from locations landward of the levees and floodwall. The location of this structure is proposed to be north of the current Merriam Marketplace. The detention basin was designed for the 10 percent AEP (1/10-year) event which was determined to be 5.4 inches of excess precipitation by a 2002 study completed by the University of Kansas and American Public Works Association (APWA) of Kansas City. This value is consistent with the current Atlas 14 precipitation depth for the 10 percent AEP (1/10-year) event. No additional detention features were indicated in the feasibility report.

Based on this assumed feasibility-level design, it is anticipated that the design of the constructed feature will incorporate the current U.S. Army Corps of Engineers (USACE) design criteria and guidance and/or local design criteria and guidance.

It is assumed that the facility will be a dry detention facility holding no permanent pool within the detention basin. Baseflows and flows from the contributing watersheds and/or interior drainage may empty through the detention basin to Turkey Creek. It is assumed that no additional sedimentation volume will be incorporated into the detention basin volume due to it not storing a constant pool elevation and it is anticipated that any accumulated sediment and/or debris could be removed in the periods between runoff events. Debris and/or sediment removal schedules and potential costs will need to be documented in the Operations and Maintenance (O&M) manual generated for the project.

USACE design criteria for the detention basin is anticipated to incorporate guidance from Engineering Manual (EM) 1110-2-1413, Hydrologic Analysis of Interior Areas, and include an evaluation of coincident events, including temporary detention of interior drainage during a Turkey Creek mainstem flood event, as well as the design of facilities capable of passing interior drainage during a relatively frequent and low magnitude Turkey Creek mainstem flow event. Due to the relatively small size of the Turkey Creek contributing watershed, it is anticipated the most likely scenario will involve a simultaneous precipitation

event over the interior area while an infrequent and high magnitude flow is occurring on the Turkey Creek mainstem. Based on EM 1110-2-1413 design criteria, generally speaking, "…The minimum [interior drainage] facility should provide interior flood relief such that during low exterior stages (gravity conditions) the local storm drainage system functions essentially as it did without a levee in place for floods up to that of the storm sewer design…"

According to City of Merriam, Kansas, ordinances, Chapter 26—Environment; Article III, Stormwater Management; Division 1, Section 26-170, APWA Standard Specifications and Design Criteria, Section 5600, Storm Drainage Systems (Kansas City Metropolitan Chapter, 2011) and the Urban Drainage Design Manual Hydraulic Engineering Circular No. 22 (HEC-22-FHWA-SA-96-078, U.S. Department of Transportation, Federal Highway Administration, 1996) should be used as the criteria for the design of stormwater management systems. In accordance with APWA 5600 and HEC-22, the design frequency coincidence is presented in Table 7-3 of HEC-22. This table indicates that for a 1 percent AEP (1/100-year) Turkey Creek design event that the coincident tributary design will correspond with a 4 percent AEP (1/25-year) event. Conversely, a 4 percent AEP (1/25-year) event on Turkey Creek should be evaluated with a 1 percent AEP (1/100-year) for the tributary. These values are based upon the contributing area of approximately 350:1 for the ratio of the Upper Turkey Creek watershed to the interior drainage area for the proposed project, and conservatively correlating the 350:1 ratio to the 100:1 ratio presented in Table 7-3 of HEC-22.

APWA 5600, Section 5608, Stormwater Detention and Retention provides guidance on the evaluation and design of detention basins and indicates the design consideration for the primary outlet works and emergency spillway. Pertinent design criteria and/or guidance are:

- Excerpted from APWA 5600, Section 5608.4, "Performance Criteria":

  E. "…**Primary Outlet Works:** The primary outlet shall be designed to meet the following requirements:

  1. The outlet shall be designed to function without requiring attendance or operation of any kind requiring use of equipment or tools, or any mechanical devices.
  2. All discharge from the detention facility when inflow is equal to or less than the maximum design storm under the required Strategy shall be via the Primary outlet system.
  3. The design discharge rate via the outlet shall continuously increase with increasing head and shall have hydraulic characteristics similar to weirs, orifices or pipes.
  4. For dry detention basins not providing extended detention, the design shall allow for discharge of at least 80 percent of the detention storage volume within 24 hours after the peak or center of mass of the inflow has entered the detention basin.
  5. Ponds shall be designed with a non-clogging outlet such as a reverse-slope pipe, or a weir outlet with a trash rack. A reverse-slope pipe draws from below the permanent pool extending in a reverse angle up to the riser and establishes the water elevation of the permanent pool. Because these outlets draw water from below the level of the permanent pool, they are less likely to be clogged by floating debris.
  6. All openings shall be protected by trash racks, grates, stone filters, or other approved devices to ensure that the outlet works will remain functional. Minimum orifice size shall be per the current version MARC/APWA BMP Manual. See MARC/APWA BMP Manual Sections 8.6, 8.10 and 8.12 for design guidance and typical details for low-flow outlets and trash rack designs. Note that multiple design options are available for non-clogging low flow outlets and debris collection and designers are encouraged to select the best design for their site.

  F. **Emergency Spillways:** The emergency spillway may either be combined with the outlet works or be a separate structure or channel meeting the following criteria.

1. **Elevation:** Emergency spillways shall be designed so that their crest elevation is 0.5 feet or more above the maximum water surface elevation in the detention facility attained by the maximum design storm under the required Strategy for the facility.

2. **Capacity:** In cases where the impoundment/emergency spillway is not regulated by either State or Federal agencies, the emergency spillway shall be designed to pass the 1% storm with 1 foot of freeboard from the design stage to the top of dam, assuming zero available storage in the basin and zero flow through the primary outlet. This design provides an added level of protection in the event of a clogged primary outlet or a subsequent 1% storm event that occurs before the flood pool from the initial storm event recedes to the principal outlet elevation.

G. Drawdown Provision: Drain works consisting of valves, gates, pipes, and other devices as necessary to completely drain the facility in 72 hours or less when required for maintenance or inspection shall be provided.

H. Erosion Control: Primary outlet works, emergency spillways, and drain works, as well as conveyance system entrances to detention basins shall be equipped with energy dissipating devices as necessary to limit shear stresses on receiving channels…"

- Excerpted from APWA 5600, Section 5608.5, "Detention Methods":

B. "…**Dry Detention Facility:** For basins designed to be normally dry:

1. **Interior Drainage for non-BMP basins:** Provisions must be incorporated to facilitate interior drainage to outlet structures. Grades for drainage facilities shall not be less than two percent on turf.
2. **Earth Bottoms:** Earth bottoms shall be vegetated.
3. **Side Slopes:** The side slopes of dry ponds should be relatively flat to reduce safety risks and help to lengthen the effective flow path. Slopes shall not be steeper than three horizontal to one vertical and at least 25% of the perimeter shall have a slope of 5 to 1 or flatter.
4. Refer to the MARC/APWA BMP Manual for design requirements on facilities providing control of the WQv (Comprehensive and Frequent Event Strategies)…"

- Excerpted from APWA 5600, Section 5608.7, "Additional Requirements":

A. "…**Access:** Provisions shall be made to permit access and use of auxiliary equipment to facilitate emptying, cleaning, maintenance, or for emergency purposes…"

SECTION 3
Feasibility Study Recommended Plan Modifications

The information within Section 3 is a summary of changes to the FS recommended plan. Section 4 contains details of the analyses performed on the recommended plan.

3.1 BRIDGES

There are three bridges within the project limits that may require relocation or modification based on preliminary findings. A value-engineering (VE) analysis was suggested by the project delivery team (PDT) to occur between the preliminary FS validation and 35 percent design completion. The purpose of the VE study would be to review the existing bridges and determine to what extent the bridges need modification or relocation.

3.1.1 Merriam Drive Bridge (Reach 3b-1)

Preliminary analysis during feasibility did not adequately address uplift and overturning of bridge and hydraulic surcharge forces on the 6.5 feet headwall caused by the 0.5 percent AEP event (1/200-year). The PDT should analyze a parapet wall height that will be adequate for the uplift and overturning on the current bridge and a cost estimate completed. In addition, a cost estimate for a new bridge with more freeboard and channel improvements to allow greater flows to pass should be investigated. The two cost estimates should be compared, and a cost to benefit ratio completed to determine the best path forward.

3.1.2 Farmer's Market Pedestrian Bridge RM 2.875 (Crossing #5 As-Built Drawings Station 85+10.22)

At this point in time, the H&H model is still in progress and a determination has yet to be made on which AEP will control for flows in the channel and how the pedestrian bridges impact that flow. Therefore, modifications to the FS recommended plan will need to be evaluated during design.

3.1.3 Pedestrian Bridge at RM 3.57 (Crossing #2 As-Built Drawings Station 49+72)

The pedestrian bridge at this location may need to be lengthened beyond the existing span by adding new spans in order to tie into a new levee on the west side and new floodwalls on the east side. The length of the new spans cannot be determined until the location of the new levee and new floodwall alignments are established. Survey data will be reviewed and compared to the AEP to determine if the bridge needs to be raised to clear the water surface elevation (WSE).

3.2 DETENTION

No changes to the proposed detention facilities are proposed at this time, however, an updated evaluation of the design presented in the FS will be undertaken as part of the updated design. A validation of the feasibility hydraulic modeling results and an update to the hydraulic analysis is being undertaken using more modern modeling capabilities.

3.3 HYDROLOGY AND HYDRAULICS

Section 3.3.1 contains a summary of hydrologic and hydraulic changes from the FS recommended plan.

3.3.1 Hydrology

Changes to the hydrology could occur during the project development phase due to an ongoing analysis with Federal Emergency Management Agency (FEMA) and its local and regional partners to update existing floodplain mapping. The analyses are ongoing as of March 2023, and preliminary and/or reviewed results were not available at the time of the development of this report. Preliminary hydrology results are anticipated to be available in the June to July 2023 timeframe based on the current schedule; however, the FEMA review and adoption process will not be complete at that time. Updated modeling software, validation efforts, increased levels of detail, and variations in parameterization may result in differing values from the hydrology adopted for the FS and this analysis. Updated modeling information will be incorporated into the design as it becomes available, pending appropriate reviews. The existing modeling adopted for this analysis remains applicable to the project site and watershed and represents the most current data source until the updated modeling has been properly reviewed and made available for consideration within this design effort. It is anticipated that the hydrology for the mainstem of Turkey Creek will be validated or updated according to the ongoing FEMA analysis for the current design effort, and that the interior drainage will be analyzed and that interior drainage features will be incorporated into the final project design.

3.3.2 Hydraulics

Due to the age of the recommended alternative from the FS, and changes observed within the watershed, it is anticipated the existing condition's hydraulic profile may have changed, and an update to the design profile should be performed. Coordination between the USACE and project sponsors will be required to select the adopted design profile based on the validation effort and current hydraulic analysis. Channel armoring and maintenance efforts have been an ongoing process with the MDD since the FS was completed.

Unsteady two-dimensional (2D) hydraulic modeling is being used to update the design profile in consideration of the increased level of detail and hydraulic modeling capabilities, as well as changes to the stream channel that have occurred since the completion of the FS. Unsteady flow hydrographs computed during the FS are being adapted and used for the current modeling effort to compare the steady flow profiles from the FS with the unsteady flow profiles generated during this modeling update and validation effort. Appendix E presents further detail regarding the modeling validation and development.

3.4 STORM DRAINAGE STRUCTURES

EM 1110-2-2902, Conduits Pipes and Culverts Associated with Dams and Levee Systems was updated in December 2020, after the completion of the FS. Closure type requirements based on pipe diameter and flood rise are now prescribed in the updated EM 1110-2-2902. The closure type requirements are for new construction but are also highly recommended for retrofitting existing pipes. Table 3-1 summarizes the active closure requirements.

Table 3-1. Closure Type Requirements[1]

Flood Rise[2]	Pipe Diameter (inches)	Closure Configuration
Fast	<36	(1)Passive
Fast	≥36	(1)Passive AND (1)Active
Slow	<36	(1)Passive OR (1)Active
Slow	≥36	(1)Passive AND (1)Active OR (2)Active

[1]Table 9-1 of EM 1110-2-2902

[2]Fast flood rise refers to floods capable of rising to a flood stage with less than 12 hours prediction time.

The PDT recommends a deviation from Table 9-1 in EM 1110-2-2902 to use two passive closures for pipes greater than or equal to 36 inches. This recommendation is based on concerns that it is not feasible for the sponsor to manually operate over 10 sluices gates throughout the project with less than 12 hours prediction time.

The PDT will further evaluate the feasibility and applicability of two passive closure structures on pipes greater than or equal to 36 inches as we progress through design. If a deviation from the requirements within EM 1110-2-2902 is required, the PDT will provide justifications for deviations and waivers from mandatory design standards in a memorandum. The memorandum will include a risk assessment and rationales for the deviation.

Table 3-2 summarizes modifications to the FS recommended plan. Table 3-3 summarizes approximate storm drainage quantities.

Table 3-2. Storm Drainage Modifications to Feasibility Study Recommended Plan

FS Recommended Plan	Modification to FS Recommended Plan	Current Recommended Plan
Install manhole on landside	Yes	For pipes <36 inches: • Install landside manhole (no landside flap gate). For pipes ≥36 inches: • Install landside flap gate within gate box.
Remove existing storm pipe and replace with a minimum 24-inch RCP under levee/floodwall. Abandon grout fill pipes if removal is not feasible.	No	Remove existing storm pipe and replace with a minimum 24-inch RCP under levee/floodwall. Abandon grout fill pipes if removal is not feasible.
Install gate box on riverside with flap gate. If installation of riverside gate box is not feasible, install flap gate at floodwall or flap gate at headwall.	No	Install gate box on riverside with flap gate. If installation of riverside gate box is not feasible, install flap gate at floodwall or flap gate at headwall.
Install headwall and flap gates at the existing triple 7-foot by 7-foot RCB drainage structure at Station 3.109	Yes	Install headwall and flap gates at the existing riverside triple 7-foot by 7-foot RCB drainage structure at Station 3.109. Install landside flap gates within landside gate box.

Table 3-3. Approximate Storm Drainage Quantities (Sheet 1 of 3)

	FS Recommended Plan	Modifications to FS Recommended Plan	Notes for Modifications to FS Recommended Plan
Pipes <36 inches Number of Riverside Flap Gates Anticipated	14	14	• 14 total riverside 24-inch flap gates
Pipes ≥36 inches Number of Riverside Flap Gates Anticipated	11	11	• 4 total riverside 36-inch flap gates • 1 total riverside 42-inch flap gate • 1 total riverside 48-inch flap gate • 1 total riverside 54-inch flap gate • 1 total riverside 60-inch flap gate • 3 total riverside 7-foot by 7-foot flap gates
Pipes ≥36 inches Number of Landside Flap Gates Anticipated	0	11	• 4 total 36-inch landside flap gates • 1 total 42-inch landside flap gate • 1 total 48-inch landside flap gate • 1 total 54-inch landside flap gate • 1 total 60-inch landside flap gates • 3 total 7-foot by 7-foot landside flap gates

Table 3-3. Approximate Storm Drainage Quantities (Sheet 2 of 3)

	FS Recommended Plan	Modifications to FS Recommended Plan	Notes for Modifications to FS Recommended Plan
Number of Riverside Gate Boxes Anticipated	19	19	• 11 total gate boxes for 24-inch flap gate • 5 total gate boxes for 36-inch flap gate • 1 total gate box for 42-inch flap gate • 1 total gate box for 48-inch flap gate • 1 total gate box for 60-inch flap gate
Number of Landside Gate Boxes Anticipated	0	9	• 4 total landside gate boxes for 36-inch flap gates • 1 total landside gate box for 42-inch flap gate • 1 total landside gate box for 48-inch flap gate • 1 total landside gate box for 54-inch flap gate • 1 total landside gate box for 60-inch flap gate • 1 total landside gate box for 7-foot by 7-foot landside flap gate
Number of Landside Manholes Anticipated	33	27	Manholes anticipated include: • 21 total 48-inch manholes (including 13 associated with re-routing) • 6 total 60-inch manholes
Approximate Linear Feet of Storm Pipe Removed Under Levee Unit	1,460	1,460	Storm pipe removal anticipated under levee unit includes: • 385 linear feet of 12-inch pipe • 50 linear feet of 18-inch pipe • 385 linear feet of 24-inch pipe • 150 linear feet of 36-inch pipe • 50 linear feet of 42-inch pipe • 30 linear feet of 54-inch pipe

Table 3-3. Approximate Storm Drainage Quantities (Sheet 3 of 3)

	FS Recommended Plan	Modifications to FS Recommended Plan	Notes for Modifications to FS Recommended Plan
Approximate Linear Feet of Storm Pipe Replaced Under Levee Unit	890	965	Storm pipe replacement anticipated under levee unit includes: • 610 linear feet of 24-inch RCP • 150 linear feet of 36-inch RCP • 100 linear feet of 48-inch RCP • 30 linear feet of 54-inch RCP
Approximate Linear Feet of Storm Pipe Removed for Pipe Rerouting (Not Under Levee Unit)**	510	510	Storm pipe removal anticipated for pipe rerouting includes: • 75 linear feet of 8-inch pipe • 220 linear feet of 12-inch pipe • 70 linear feet of 18-inch pipe • 50 linear feet of 24-inch pipe • 95 linear feet of 36-inch pipe
Approximate Linear Feet of Storm Pipe Replaced for Pipe Rerouting (Not Under Levee Unit)**	1,025	1,025	Storm pipe replacement anticipated for pipe rerouting includes: • 325 linear feet of 12-inch pipe • 200 linear feet of 18-inch pipe • 400 linear feet of 24-inch pipe • 100 linear feet of 36-inch pipe

**Rerouting of storm pipe includes reroutes for the proposed detention basin. Anticipate 350 linear feet of storm pipe removal, 625 linear feet of storm pipe replacement, and 1 total 60-inch manholes, and 12 total 48-inch manholes. The proposed detention basin outlet structure will consist of a gate box and a 60-inch flap gate.

SECTION 4
Feasibility Study Recommended Plan Preliminary Analysis

The preliminary analysis contained herein is meant as a review of FS recommended plan. The analysis relies heavily on engineering judgment. As the project progresses to the 35 percent design submittal, engineering judgment will be replaced with detailed engineering analysis.

4.1 HYDROLOGY AND HYDRAULICS

The hydrologic modeling from the FS is currently being carried forward unchanged for this analysis pending the completion of the FEMA analysis currently underway. Following the completion of the FEMA analysis and a review by USACE personnel, it is anticipated that a verification of the results will be undertaken for the project site, and that some modifications to the developed modeling for the design of the authorized project will occur to adapt the FEMA-generated hydrology to this analysis and for this project.

Johnson County, Kansas, initiated a hydrologic modeling effort for Turkey and Brush Creeks in calendar year 2023. This effort will incorporate updated hydrology into the current design effort at the conclusion of the analysis. This effort will include 2D rain-on-grid analysis and spatially-varied Atlas 14 precipitation frequency analyses, as well as validation and/or calibration efforts corresponding with observed precipitation events.

The hydraulic modeling for the existing conditions and FS validation effort is based upon a HEC-RAS, version 6.3.1, 2D unsteady model. If software updates occur prior to the completion of the modeling efforts, the most current version of the HEC-RAS software will be used. The top of levee profile indicated in the FS was not consistent between Table 3-7 of the report, and the final HEC-RAS modeling. Table 4-1 presents both datasets. Based upon an examination of project records, the table to the right in Table 4-1, “Final FS HEC-RAS Model” was the value adopted for the FS engineered design.

Due to uncertainties relating to the development of the Turkey Creek watershed over the previous 30 or more years, a lack of records of channel modifications, and the lack of flow data to calibrate with observed stage data, a calibration of the recently developed 2D hydraulic model using historic data was not performed as part of the current hydraulic analysis. Roughness parameters generally consistent with the FS modeling have been adopted, and bridge and culvert structures are simulated according to the structures’ current configuration as of April 2023.

The FS levee and floodwall design profile was originally based upon the 0.5 percent AEP TP-40 flow profile and was designed to pass flows corresponding with the TP-40 1 percent AEP-computed discharges from the hydrologic modeling. During the development of the report, NOAA Atlas 14 was published, and a revision to the hydrologic analysis was performed to evaluate the levee and floodwall profile design in consideration of the NOAA Atlas 14 1 percent AEP precipitation depths. In consultation with the project sponsor, and following an update to the economic analysis, revisions to the design levee and floodwall profile were implemented to allow for the Recommended Alternative 2d levee and floodwall profile to be able to pass the Atlas 14 1 percent AEP flows. This revised levee and floodwall design profile did not provide sufficient height to meet the desired 90 percent assurance (conditional non-exceedance probability) and provided an estimated 82.9 percent assurance for the Atlas 14 1 percent AEP discharges.

Table 4-1. 90 Percent Reliability Level of Protection per Table 3-7 of the Feasibility Study, and the Included Levee Profile Elevations in the Final Feasibility Study HEC-RAS Modeling

	Table 3.7, FS			Final FS HEC-RAS Model*		
	LB	RB		LB	RB	
River Station	Top of Wall/ Levee	Top of Wall/ Levee	1 Percent AEP Water Surface Elevation	Top of Levee/Wall	Top of Levee/Wall	1 Percent AEP Water Surface Elevation
2.623	909.00	909.00	906.47	913.03	913.03	907.81
2.636	Merriam Drive					
2.651	913.00	913.00	910.50	914.00	914.00	912.81
2.719	——	912.74	910.24	——	914.00	912.42
2.777	——	913.89	911.39	——	914.75	913.43
2.813	——	914.75	912.25	——	915.00	914.07
2.866	Merriam Marketplace Pedestrian Bridge					
2.945	——	914.90	912.40	——	915.75	914.21
3.040	——	917.65	915.15	——	917.65	916.38
3.058	Johnson Drive					
3.07	——	918.16	915.66	——	918.16	917.10
3.096	——	917.59	915.09	——	917.59	916.54
3.165	917.55	917.55	915.05	918.00	918.00	916.43
3.225	920.38	920.38	917.88	920.37	920.37	919.22
3.231	920.39	920.39	917.89	920.38	920.38	919.22
3.298	920.99	920.99	918.49	920.98	920.98	919.77
3.345	921.55	921.55	919.05	921.55	921.55	919.97
3.420	924.98	924.98	922.48	924.97	924.97	923.06
3.481	927.83	927.83	925.33	927.83	927.83	925.74
3.539	928.78	928.78	926.28	928.78	928.78	926.92
3.560	930.81	930.81	928.31	931.5	931.5	931.19
3.57	Turkey Creek Trail Pedestrian Bridge					
3.570	N/A	N/A	N/A	932.42	——	930.79
3.607	931.78	931.78	929.28	932.36	——	930.12
3.665	935.31	935.31	932.81	935.31	——	933.9
3.690	936.85	936.85	934.35	——	——	935.54
3.711	Shawnee Mission Parkway Bridge					

*This water surface and levee profile has been adopted for design of the Upper Turkey Creek project based upon presented data in the Feasibility Study.
Note: All Elevations are in feet, NAVD88

4.2 CLIMATE CHANGE ANALYSIS

A climate change analysis consistent with Engineering and Construction Bulletin (ECB) 2014-10 was developed as part of the FS. A qualitative consideration of the impacts to the hydrology due to potential climate change is presented in the FS. Since the FS was completed, ECB 2014-10, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects, has been updated to ECB 2018-14, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects, and two revisions have been published. The current edition, with an expiration date of 19 August 2024, was published on 19 August 2022.

The ECB does not anticipate that the climate change analysis would alter the numerical analysis for the other non-climate aspects of the required hydrologic analyses. The qualitative analysis can inform the decision process related to project planning, engineering, and O&M. Examples of how the qualitative analysis may influence the project design include considering whether the project could be modified in the future or if another strategy should be considered to accommodate projected future increases in discharge. A climate change analysis will be performed as part of the design process and according to ECB 2018-14 or the most current version of USACE guidance if the above document is superseded.

4.3 UNDERSEEPAGE

Underseepage analysis was not documented in the FS. Preliminary underseepage analysis was performed as part of this report (see Appendix B.1). The two existing soil borings near the current project, DC-4, and DC-5, did not show a pervious blanket (foundation sand) in the soil profile. Both borings showed a continuous impervious blanket of clay and weathered shale overlying limestone bedrock approximately 13 feet below existing grade. In other areas, a zone of foundation sand may exist between the impervious clay blanket and bedrock. Even if the impervious blanket is continuous to bedrock throughout the levee unit, vertical underseepage pressures may build up through joints and seams in the bedrock. This may be modeled as a zone of foundation sand.

The preliminary analysis checked sensitivity of the vertical underseepage gradient factor of safety for top of levee (TOL) loading to increasing amounts of foundation sand between the impervious blanket and bedrock, using the levee size and seepage length based on the FS. Based on those parameters, an acceptable factor of safety of 1.6 at the landside levee toe could be obtained with a maximum 5.5-foot thick layer of pervious sand above bedrock.

Underseepage analysis will be performed as we progress through design. A subsurface investigation, including soil borings and laboratory testing, will be used to support this analysis. The design water loading will be determined during the next design phase after review of the hydraulics and hydrology (H&H) model. The design will be bracketed with analysis up to the TOL loading.

4.4 SLOPE STABILITY

Limited global slope stability analyses were performed for the FS, with some results reported in the text of Appendix B and some software output and hand calculations included in Appendix B5-B. These included analyses provided by the original Architect-Engineering (AE) designer and later analyses by USACE.

4.4.1 Architect-Engineer Feasibility Stability Analyses

Stability was analyzed for breach surfaces including the new levee structure and the existing channel at three cross sections considered to be the most critical. The levee or floodwall is not clearly defined in the cross sections in the software output; it is not clear that the new structures were considered in the analyses.

Stability was analyzed using the software program XSTABL, version 5.2, by Interactive Designs, Inc. The program was used to perform general limit equilibrium analysis using the Janbu and Bishop methods. Analysis of normal operating conditions assumed water surfaces approximately 5 to 8 feet below the TOL. For this condition, the minimum factor of safety was calculated as 4.3.

The XSTABL output shows that undrained soil shear strength was used for the analysis. Two soil layers were used, a stronger upper soil layer that extended approximately five feet below ground surface and a thicker lower layer extending from the bottom of the upper layer to the top of bedrock. The upper layer was modeled with a unit weight of 120 pounds per cubic foot (pcf), a cohesion of 2,000 pounds per square foot (psf), and a friction angle of zero. The lower layer was modeled with a unit weight of 115 pcf, a cohesion of 800 psf, and a friction angle of 0.

The three cross sections were also analyzed for sudden drawdown with a reported minimum factor of safety of 1.9. The sudden drawdown analysis output was not included in the report.

Chart-based hand calculations with limited documentation were also provided for the post-construction and sudden drawdown cases at one cross section, with factors of safety shown as 2.3 and 1.2, respectively.

The existing creek bank with the limestone-block retaining wall was analyzed for global stability at one cross section on XSTABL with the wall modeled as a third soil layer with a unit weight of 145 pcf, friction angle of 30 degrees, and cohesion of zero. The calculated factor of safety was 2.2.

Hand calculations with limited documentation were also provided for checking overturning and sliding of the limestone block wall at one cross section, with calculated factors of safety of 2.6 and 2.7, respectively.

4.4.2 U.S. Army Corps of Engineers Feasibility Stability Analyses

An evaluation of Atlas 14 precipitation impacts on the recommended plan was performed after the AE firm that developed the FS was released from contract. During the re-evaluation of the recommended plan, USACE, after additional conversations with the City of Merriam, Kansas and MDD, discovered the existing limestone-block retaining wall that supports the creek bank experiences regular breaches along the proposed levee system reach. When these breaches occur MDD rebuilds the section of breached creek bank. The wall is not considered capable of providing support to the proposed levee or floodwall.

USACE considered several alternatives to either stabilize the creek bank or provide a foundation for the project that does not rely on the bank's continued stability. The first option was to stabilize the creek bank, remove the existing creek bank wall, and replace it with a new modular gravity wall system (gabion basket wall, etc.). This alternative screened out due to excessive cost for construction in the wet and temporary shoring during construction.

Next, two alternatives were evaluated to provide a floodwall foundation that does not rely on the limestone-block retaining wall. While the alternatives do not rely on the existing limestone block wall for short-term stability, this solution relies on the MDD to continue making timely repairs to the limestone-block retaining wall. The exact operational requirements for length of time or conditions before scour affects stability of these alternatives was not defined in the FS. The two alternatives were to construct a secant drilled shaft floodwall or a floodwall supported on unreinforced auger cast piles.

Constructing a secant drilled shaft floodwall consists of installing overlapping drilled shafts to form a secant pile wall landward of the existing limestone block wall. The new floodwall would be an I-wall structurally tied to the drilled shafts. The lateral load capacity of a single drilled shaft proposed to be used in the secant wall was analyzed using Ensoft's LPILE software. A distributed lateral load was applied to the pile based on a 250 psf surcharge at the ground surface and vertical stress from the ground surface to top of bedrock. The analysis assumed the secant wall would not be supported by the existing limestone-block retaining wall or the soil between the retaining wall and the proposed secant wall. Bedrock was modeled as strong rock (vuggy limestone) with an effective unit weight of 82.6 pcf and an unconfined compressive strength of 2,000 psi. This alternative screened out due to higher costs of construction than other alternatives.

Constructing a floodwall supported on unreinforced auger cast piles is the recommended option from the FS and consists of 16-inch diameter, unreinforced concrete, soil replacement columns extending to the top of bedrock. The columns will be placed approximately 2 feet on-center in an equilateral triangle

pattern. The recommended width of ground improvement is 15 feet, centered on the floodwall footing. The recommended plan is based on MDD repairing the breached creek banks so that there will not be permanent ongoing scour of the improved ground supporting the floodwall. The exact O&M requirements for those repairs were not defined. The sketch included in Appendix G shows the floodwall footing extended to a 15-foot width incorporating the entire width of ground improvement. The auger cast piles are unreinforced, and no connection was defined between the piles and the footing. The entire block was analyzed as a wall sitting on bedrock with composite properties for the improved soil. Hand calculations showed factors of safety of 1.3 for sliding and 3.9 for overturning.

4.4.3 Current Analyses

Limited levee slope stability analyses were performed for this report using parameters based on soil properties at other projects or textbook soil properties (see Appendix B.2). The FS states levees were used for the design where there was enough space for greater than 1V:2H slopes up from the creek bed to the levee toe. The FS used 1V:3H slopes for the levee embankment above the top of channel.

Sudden drawdown stability for the riverside slope was analyzed for TOL loading using assumed soil strengths similar to those used for the Argentine Levee raise on the Kansas River near its confluence with the Missouri River (see Appendix B.2). Soil strengths used from the Argentine Levee raise are thought to provide a reasonable estimate of those anticipated to be found during the proposed subsurface investigation. Using these soil strengths and using levee parameters based on available information, sudden drawdown stability was checked for various slopes of a 13-foot tall channel below a 6-foot tall levee.

Analysis results (Table 4-2) suggest that levee cross sections with channel slopes of at least 1V:2H would have an acceptable factor of safety for sudden drawdown. In areas where the channel slope is less steep, the recommended factor of safety could be met with lower soil strengths.

Table 4-2. Sudden Drawdown Analysis Results

Channel Slope	Sudden Drawdown Factor of Safety (1.0–1.2 required)
1V:1H	0.6
1V:2H	1.1
1V:3H	1.3

Preliminary slope stability analysis of the landside slope was also performed for steady state seepage with TOL loading, and assuming soil strength from the Argentine Levee raise.

This analysis resulted in a factor of safety of 0.9 with a 6-foot tall levee with 1V:3H slope loaded to TOL. When a two-foot thick sand drain was added to the base of the landside slope, the factor of safety increased to 1.3, just under the recommended minimum of 1.4. When a small stability berm (3-feet wide by 5-feet tall) was also added, the factor of safety increased to 1.6.

For the recent design of the Argentine, Armourdale, and Central Industrial District Levee raises, sand drains were required to meet the recommended factor of safety of 1.4 for steady state seepage with TOL loading. Depending on impervious blanket thickness, sand drains and small stability berms may be required to meet the factor of safety for that loading for the Upper Turkey Creek levee. In areas with limited space, this could result in changing the cross section from levee to floodwall.

Sustained water loading leading to steady state conditions at top of levee is not expected to occur due to the flashy nature of Turkey Creek. The design water loading will be determined as we progress through design after review of the H&H model. The design will be bracketed with analysis up to the TOL loading

and will include sensitivity checks for at what water loading sand drains or stability berms would be required to maintain the required factor of safety. With a lower design water level, these features may not be required. These analyses will be done as we progress through design when additional information on subsurface conditions is available from soil borings and laboratory testing.

4.4.4 Future Analyses

The proposed levee and floodwall areas will be evaluated per the stability requirements of EM 1110-2-1913 and EM 1110-2-1902 for end-of-construction, steady-state seepage, and sudden drawdown conditions. Soil parameters will be determined after subsurface investigation and laboratory testing. Local stability requirements for floodwalls are discussed in Section 4.5, Structures.

The design water loading will be determined after review of the H&H model. The design will be bracketed with analysis up to TOL loading.

The proposed alternatives for supporting the floodwall without depending on the limestone block creek wall will be re-evaluated as we progress through design. Although soil improvement with auger cast columns was the recommended alternative in feasibility, it is noted that this solution depends on the sponsor performing timely repairs to breaches of the limestone block creek wall. Soil improvement provides short-term stability for the floodwall in the event of breach of the limestone block creek wall, but subsequent water loading in an unrepaired area could scour out the improved soil foundation and lead to breaches of the floodwall. If the soil improvement alternative is carried forward, O&M responsibilities and the consequences of not making those repairs quickly need to be clearly defined. A rough order of magnitude to construct the auger cast columns with floodwall are $5,000 per lineal feet. This cost does not account for the likely possibility of acquiring real estate to construct.

Although the secant pile wall alternative was noted to be too expensive during feasibility, the increased initial cost would be cost-shared between the sponsor and the federal government as part of the project. A floodwall supported by a secant pile wall would be robust and not depend on the limestone-block retaining wall being repaired. The feasibility report does not include the raw numbers for the cost comparison and does not consider the cost of the limestone-block retaining wall repairs in the comparison. A rough order of magnitude to construct a secant pile with floodwall ranges from $2,000-$20,000 per lineal feet due to varying depths to bedrock. These cost comparisons will be considered as we progress through deisng.

4.5 SETTLEMENT

EM 1110-2-1913 states evaluation of the amount of post-construction settlement that can occur from consolidation of the embankment and foundation may be important if the settlement would result in loss of freeboard of the levee or damage to structures in the embankment.

The EM states many districts overbuild a levee or berm by a given percent to account for anticipated settlement, both of the foundation and within the levee or berm fill itself. The EM references overbuild of five to 10 percent for semi-compacted fill, which is the anticipated degree of compaction for berms.

The FS did not include settlement analysis.

4.5.1 Current Analyses

Consolidation settlement of the new compacted embankment soil is assumed to be relatively insignificant. Limited settlement calculations were performed for settlement of the clay blanket due to construction of a new 6-foot tall levee using the consolidation parameters from the Argentine Levee raise (see Appendix B.3). This analysis resulted in approximately 1.6 inches of settlement below the new levee.

4.5.2 Future Analyses

Consolidation settlement of the clay subgrade soils due to construction of the new levee will be analyzed based on soil borings and consolidation testing. Settlement will also be evaluated in floodwall areas using

the floodwall load and footing size. An overbuild may be recommended to maintain the design level of flood risk reduction after settlement.

4.6 STRUCTURES

The 2015 Feasibility National Economic Development (NED) Plan recommended an inverted T-wall floodwall design with a base width of 10 feet and a stem stick-up varying in height up to 6.5 feet. The floodwall would be supported on a 15-foot wide bed of soil reinforced using unreinforced auger cast piles. A key will not be necessary for sliding as the base will be entrenched within the reinforced soil on the landside of floodwall (see Appendix G). The riverside of floodwall will be supported by the soil reinforcing and is weighed down by the soil. A stem and footing thickness of 18 inches and a minimum of 4,000 psi reinforced concrete, will provide the necessary strength capacity needed to withstand the 0.5 percent AEP (1/200-year event) flows.

4.6.1 Alternative—Secant Pile Floodwall

As discussed in the geotechnical section, the recommended plan floodwall dimensions indicate a floodwall base 10-feet wide and system height up to 6.5 feet. The feasibility report mentions that it considered a secant pile floodwall, but it was deemed too expensive, and the selected plan was chosen which calls for ground modified soils columns under the floodwall. The use of a full height secant pile floodwall should be investigated during design process. A decrease in floodwall footprint by up to 8 feet could be expected assuming a 24-inch diameter shaft. This decrease in footprint would be significant in terms of cost and the impact to affected nearby buildings. The project delivery team (PDT) will consider this alternative during design and a cost estimate completed for comparison purposes. The exposed height of the floodwall varies from 0 to 6.5 feet and, therefore, meets the criteria of an I-wall. Current I-wall criteria is provided in EM 1110-2-2502 Floodwalls and Other Hydraulic Retaining walls (August 2022).

4.6.2 Stability

FS criteria was partially based on EM 1110-2-2100 available at the time. The current design requirements for local stability are defined in EM 1110-2-2100. Section 3-2 of EM 2100 (2005) load condition categories. Table 4-3 shows a comparison of the local stability criteria used during feasibility and what is currently required by design. The comparison is for the unusual loading condition only which was the basis of the feasibility recommendations. The return period for both is greater than 10 years but less than 750 years for critical structures as discussed in EM 1110-2-2502 (2022). This is an unusual loading category for critical structures per Figure 6.1 of EM 1110-2-2502 and the load combination of Table 6.1 of EM 2502 must be applied during the floodwall design.

Table 4-3. Top of Levee Loading Stability Criteria[1]

	Feasibility	Design (Critical)
Flotation Min FS	1.2	1.2
Sliding Min FS	1.3	1.3[2]
Rotation (Percent Base in Compression)	75 percent	75 percent
Bearing	Min FS=3.0[2]	Min FS=3.0[2]

[1]Assumes “Well Defined” Site Information Category

[2]For Critical structure designation, typical of levee unit structure unless specifically determined otherwise.

The new structures recommended in feasibility were approximated for cost purposes, and, therefore, the changes to the stability criteria have minimal impact on the cost of the recommended new structures. The criteria have not changed from the FS to current analysis, so no changes are expected to be made to the floodwall.

4.6.3 Strength Design Parameters

For the FS, the structures were evaluated with the new levee and floodwall loads and load factors were required to meet EM 1110-2-2104 (dated 30 November 2016). Table 4-4 summarizes these load factors for design. The comparison is for the unusual loading condition only which was the basis of the feasibility recommendations. The return period for both is greater than 10 years but less than 750 years for critical structures as discussed in EM 1110-2-2104. This is an unusual loading category for critical structures per Table 3-1 of EM 1110-2-2104 and these load factors must be applied during the floodwall design.

Table 4-4. Top of Levee Loading Strength Criteria

	Feasibility	Design (Critical)
Load Factors (LF)	1.6	1.6

4.7 BRIDGES

The following sections present a summary of the preliminary analysis performed on the three bridges requiring relocation and/or modifications.

4.7.1 Merriam Drive Bridge RM 2.636 (Reach 3b-1)

The FS recommended adding 4.5-6.0 feet tall parapet walls on each side of bridge to raise the bridge walls and use them as part of the line of protection. As discussed in Section 3.1, other factors, such as verification of hydraulic surcharge loading, scour effects around the bridge footings and side slopes near abutments, channel alignment, and debris impact should be considered in design.

A 0.5 percent AEP (1/200-year) event is the more likely event the design will meet according to the design PDT. Preliminary analysis shows that a headwall of six feet in height will increase overturning and uplift loads acting on the bridge and the bridge will breach. Preliminary calculations show a 3.5-foot high headwall may be adequate for overturning and uplift to meet the EM 1110-2-2100 requirements. A 3.5-foot headwall would put the top of wall at (911.5 feet [1948 record drawings]+3.5 feet) 915 feet. The 0.5 percent WSE at the bridge would be approximately 917.21 feet with the addition of a calculated 1.94 feet of surcharge on the wall for a total of 919.5 feet at the bridge. At the 919.5-foot height, and with a parapet wall 3.5 feet in height, the 0.5 percent AEP flows would completely overtop and equalize uplift pressures and rotational forces at the bridge. Scour in and around the bridge footings and side slopes, and debris impact forces on the bridge would most certainly need to be considered during design. Therefore, the FS recommended plan will not work due to bridge stability concerns. If the top of protection is lowered below approximately 915 feet, it maybe be possible to include the bridge walls into the line of protection. If the existing bridge is not adequate to meet the mentioned design requirements, the sponsor should consider a new bridge at this location due to channel alignment, age, and height of the bridge in relation to the 0.5 percent AEP event which may be a better alternative from an engineering and economics view. A comparative cost analysis for the existing bridge to remain versus building a new bridge that adequately addresses all concerns should be considered during design. A parametric estimate of $3.5 million was estimated during the June 2022 cost and schedule risk analysis (CSRA).

4.8 UTILITIES

Kansas City District published draft guidance entitled, Piping Systems, with recommendations for construction of a piping system through a levee embankment or its foundation. This guidance, as well as the following engineering manuals, will be used as criteria for utility relocations during design:

- EM 1110-2-1913, Design and Construction of Levees
- EM 1110-2-2502, Floodwalls and Other Hydraulic Retaining Walls
- EM 1110-2-2902a, Conduits, Pipes, and Culverts Associated with Dams and Levee Systems.

Section 5.1 lists these engineering manuals.

NWK local guidance, entitled Piping Systems, is found at:

https://usace.contentdm.oclc.org/utils/getfile/collection/p266001coll1/id/7575

4.8.1 Crossings

The FS investigated known utilities within the recommended plan's project area. The recommended methodology for storm structure relocations has changed since the FS (see Section 3.4). The methodology for the remaining utility relocations has not changed since the FS; however, relocations are expected to be modified during design as data for utilities is incomplete. Working with the real estate section, the PDT will re-evaluate the existing utilities and any additional utilities found with the updated survey or through data gathering. As noted in Section 2.4.1, numerous private storm water lines are assumed to exist within the project area based on site observations of small diameter pipes daylighting into Turkey Creek. Private storm water lines were not identified within the FS. Private storm water lines within the project area will be evaluated as we progress through design.

Table 4-5 details criteria for pipelines crossing levees to be reviewed during design.

*Table 4-5. Criteria for Pipelines Crossing Levees**

Pipelines	Leaving Existing Pipeline in Foundations of Proposed Levees	New Pipeline Installation	
		Pipes Through Levees	Pipes Over Levees
Must be known to be in good condition	X		
Must have adequate strength to withstand levee loading	X	X	
Must have adequate cover as needed to prevent damage by vehicular traffic or heavy equipment			X
Must have adequate cover for frost protection			X
Must have sufficient flexibility in joints to adjust under expected settlement and stretching of pipe	X	X	X
Pressure lines must have provisions for rapid closure in event of leakage or rupture	X	X	X
Gravity discharge pipes must have provisions for emergency closure in event of inoperative flap valves on riverside end	X	X	
Must have pervious backfill under landside third of levee where:			
a. Foundation materials are susceptible to piping	X		
b. Levee materials are susceptible to piping		X	

*Table 8-1 of EM 1110-2-1913

4.8.2 Uplift

The FS did not investigate utilities within the critical area for uplift concerns. Uplift will be analyzed during the design phase to assist in determining recommendations for utilities.

4.9 ROADWAYS

Roadway modifications are anticipated to be required to accommodate the construction of the proposed project. The design will consider impacts to the existing roadways within and adjacent to the proposed project. Table 4-6 shows anticipated roadway modifications.

Table 4-6. Anticipated Roadway Modifications

LB and/ or RB	Roadway	Anticipated Modifications
LB	Frontage Road	No modifications anticipated.
RB	Merriam Drive	Roadway repair of Merriam Drive after the installation of a culvert. Modification of Merriam Drive at the southwestern intersection of Johnson Drive and Merriam Drive may be required to accommodate adjacent floodwall. Modification of Merriam Drive adjacent to proposed detention basin may be required for access.
LB	West 62nd Terrace	Modification of West 62nd Terrace may be required to accommodate adjacent floodwall.
LB and RB	West 62nd Street	Access to Streamway Park Trail may be eliminated of modified along the LB.
LB	Knox Avenue	Modifications of Knox Avenue may be required to accommodate adjacent levee.
RB	Merriam Lane	Modification of Merriam Lane may be required to accommodate adjacent floodwall.
LB and RB	West 61st Street	Campbell Park—Streamway Park access may be eliminated or modified at LB. Access ramp to RB levee may be constructed and accessed via West 61st Street
LB	Campbell Street	Modification of Campbell Street may be required to accommodate adjacent floodwall.
LB	West 60th Street	No modifications anticipated.
LB and RB	Johnson West	Modification of Johnson Drive may be required to accommodate a closure structure (anticipated stoplog gap) east of the Johnson Drive Bridge.
RB	Kessler Lane	No modifications anticipated.
RB	West 58th Terrace	No modifications anticipated.
RB	Hocker Drive	Modification of Hocker Drive (currently access to a parking lot) may be required to accommodate adjacent levee.
RB	West 58th Street	No modifications anticipated.
RB	West 57th Street	No modifications anticipated.

4.10 SITE ACCESS CONSIDERATIONS

Site access will continue to be evaluated during the design. As the proposed project is within a fully developed urban area of Merriam, Kansas, parallel to I-35, site access is restricted and will require coordination with various stakeholders. Stakeholders include the City of Merriam, Kansas, MDD, commercial properties, and private properties.

Several segments of the existing Turkey Creek and Turkey Creek Streamway Trail within the recommended plan's project area are not within property owned by the City of Merriam, Kansas or the MDD. During design we will evaluate access and ownership for areas where routine operation and maintenance of the project will be required (see Section 4.12 for further discussion on real estate).

The CSRA performed within the FS estimated site access restrictions due to tight corridor work could reduce production rates by 20 percent, which is a key cost risk driver of the project. By continuing to evaluate site access during the design phase, this key risk driver can be further mitigated.

During design we will consider ways to mitigate dust, mud, and debris on the construction sites and access roads throughout the design process. Dust control may have an impact on risk drivers such as production rates and overall project cost.

4.10.1 Access Roads

The FS indicated access roads to the project will stem from three urban corridors that connect I-35 to the project include. The corridors include:

- Shawnee Mission Parkway (Exit 228B) is the upstream project limit
- Johnson Drive is the approximate midpoint of the project
- Merriam Drive Bridge is the downstream project limit.

Site access will be further evaluated during design and could potentially utilize several streets that terminate adjacent to Turkey Creek. Table 4-7 lists these streets (from upstream to downstream).

Table 4-7. Access Roads

LB Roadway Access Options	RB Roadway Access Options
W. 61st Street (residential)	W. 62nd Street (commercial)
W. 60th Street (residential)	W. 61st Street (commercial)
Campbell Street (residential)	
Grant Street (residential)	

4.10.2 Access Ramps

Access ramp locations for levee access and creek access will be further evaluated during design. The PDT will also further evaluate limitations on crossing over existing sanitary lines that cross the Turkey Creek channel. The PDT understands that MDD currently will not cross sanitary lines in the channel with vehicles due to concerns for pipe age and low clearance over the pipes.

Per guidance in EM 1913, the following general levee access ramp considerations will be further evaluated during design:

- a minimum of one landside access ramp may be provided at each levee segment.

- If a levee segment does not have an entrance and exit ramp, a turnaround may be provided. (No turnouts are assumed to be needed as each levee segment is less than 2,500 feet.)

There is a total of seven levee segments within the FS recommended plan. Locations for access ramps to each levee segment were not identified in the FS. Table 4-8 summarizes the levee segments as well as access ramp considerations.

Table 4-8. Access Ramp Considerations

Levee Segment Details			Access Ramp Considerations
Approximate Station (Per FS)	**Approximate Length (feet)**	**LB or RB**	
3.65	180	RB	Limited layout options as access from Shawnee Mission Parkway and Merriam Drive is restrictive
3.595	320	LB	Potential access from West 62nd Street with turnaround at upstream end
3.495	1,034	LB	Potential access from West 62nd Street with turnaround at upstream end
3.48	240	LB	Potential access at West 61st Street with turnarounds at upstream and downstream ends
3.39	390	RB	Potential access at West 61st Street with turnarounds at upstream and downstream ends
2.97	1,051	RB	Potential access from West 57th Street with turnaround at upstream end
2.68	168	LB	Limited layout options as access from Merriam Drive is restrictive. Potential access from Merriam Drive with turnaround at upstream.

4.10.3 Construction Staging Areas

Construction staging areas will be further evaluated during design. The FS noted the Construction and Engineering Division staff determined two acres is adequate for construction staging. Four sites have been identified in the FS and will be further evaluated during design:

- Site 1: 0.7 acres northwest of the intersection of Merriam Drive and Turkey Creek. Access to this site is from Merriam Drive.
- Site 2: 2 acres at Merriam Marketplace and adjacent commercial property north to be acquired for the project. Access to this site is from Merriam Drive.
- Site 3: 2 acres including a park and adjacent residential property to be acquired for this project. Access to this site is from West 61st Street via Mastin Street and Shawnee Mission Parkway.
- Site 4: 3.6 acres bounded by Shawnee Mission Parkway and Merriam Drive. Access to this site is from Merriam Drive.

4.10.4 Access to Public Parks and Trails

The FS indicated the sponsor and the public have voiced concerns of potential impacts to the adjacent parks and trails. Parks and trails within and adjacent to the project to be considered during design include the following:

- Merriam Marketplace (facilities used for the Farmer's Market)
- Werner Park (LB located near West 57th Street and Knox Street)
- Campbell Park (LB located at West 61st Street and Knox Street)
- Turkey Creek Streamway Trail (LB from West 75th Street to Werner Park)
    - Farmer's Market Pedestrian Bridge at approximate RM 2.875 (connecting trail to Merriam Drive north of West 63rd Street)
    - Pedestrian Bridge at approximate RM 3.575 (connecting trail to Merriam Marketplace).

As we progress through design we will further evaluate accommodating public access to parks and trails. Approximately 2,285 feet of Turkey Creek Streamway Trail is riverside of the recommended LB levee/floodwall alignment. This long stretch of Turkey Creek Streamway Trail could be inundated quickly during flash flood events. During design we will assess if design modifications are necessary to provide adequate alerts and/or evacuation routes to this stretch of Turkey Creek Streamway Trail users during high water events.

Table 4-9 lists trail segments to be replaced or relocated per the FS recommended plan.

Table 4-9. Trail Segments Proposed for Replacement

Approximate Station Location (Per FS Stationing)	LB or RB	Trail Replaced or Relocated	Proposed Approximate Length to be Replaced or Relocated (feet)
Station 3.569	RB	Replaced	100
Station 3.569	LB	Replaced	370
Station 3.349	LB	Relocated	100
Station 3.250	LB	Replaced	700
Station 2.679	LB	Relocated	200*

*200 feet will be demolished; 250 feet is required to be relocated to provide levee crossing.

4.10.5 Access to Private Property

Access to private property will be required to construct the recommended plan. Private property within the recommended plan's project footprint may have permanent impacts to their current access conditions. The following impacts to private property during and after construction will be further assessed during design:

- Access ramp location impacts (potential impacts to properties with proposed levee segments)
- Impacts to private parking (potential impacts to the western extents of commercial properties along the RB)
- Impacts to loading dock access (potential impacts to 6230, 6234, 6236, and 6240 Merriam Drive).

4.10.6 Traffic Control

Traffic control will be further evaluated during design. Traffic control will include vehicular and pedestrian plans and include re-routing of pedestrian traffic along Turkey Creek Streamway Trail during certain phases of construction activities. Coordination between the sponsor, commercial properties, residential properties, and the public will be required.

4.11 BORROW

No borrow sources were identified during FS, and a subsurface investigation was not performed for any proposed borrow location because it was determined that the volume needed for this project would be small enough that it would be more cost-effective to use a commercial borrow source. This will be investigated again during design, to determine if this remains accurate, or if it is more cost-effective, or reduces project risk to include a borrow source with this project.

4.12 REAL ESTATE

The City of Merriam, Kansas, owns fee over approximately 7.5 acres along the project area. These lands are available for the project but are not sufficient to cover all improvements. The city has also granted the use of city parks in the area to mitigate tree loss. The MDD also owns fee over 9.2 acres along the channel and will be available for use on the project.

Required estates for project purposes outlined by FS will include permanent levee and flood protection easement, temporary work area easements, fee acquisition, and utility easements. The entire project will affect 121 tracts. Parcels JP35000000-0009, JP28000033-0024D and JF241212-3006 will need to be acquired in fee due to the location of the buildings and the need for space for the levee and/or floodwall. Parcel JF241212-3005 is where the detention basin will be constructed. The elevation of this property will be adjusted and will require removal of the building. No disposal action is needed for this project.

Parcel JF241212-3005, a commercial property, is the location of the proposed detention basin, which will require a fee acquisition. The proposed placement of flood control structures run through the center of parcel JP35000000-0009, a commercial property. Parcel JF241212-3006, a commercial property, has improvements near the current bank, leaving little room for construction of flood control structures. A small storage building is located within feet of the bank on parcel JF84000009-0001A, a commercial property. The shed will be the only structure removed from this property. The one residential property impacted is JP28000033-0024D. The flood control structures proposed run through the parcel and require a full acquisition. Total estimated costs for these five P.L. 91-646 relocations are $250,000.00.

The unidentified public utilities could be an issue as most of the areas of interest are currently heavily developed and have been so for close to one hundred years.

There are four properties that require fee acquisition and one shed that will have to be acquired. The estimated cost includes the estimate for the purchase of these properties and an estimated cost for P.L. 91-646 relocation costs of $9,645,342.00. This includes land acquisition; utility/facility relocation; and lands, easements, rights-of-way, relocations, and disposal area (LERRD) administrative costs.

4.13 HAZARDOUS, TOXIC, AND RADIOACTIVE WASTE

A records search for potential contaminated sites along the reach of Turkey Creek addressed by this DDR, was performed in 2009, and updated in 2015 and again in 2023. Based on the findings from the records search, there are no known superfund sites that will impact the proposed work. In addition, no solid waste facilities, such as former landfills, were identified at locations that would impact the project. There were several spills reported near Turkey Creek along the reach addressed by this DDR, but they were quickly remediated and should not have any significant impact to the project. There are six properties with current or former underground storage tanks (USTs), with the status of the tanks unknown on two of those properties, 5560 Merriam Drive, and 5745 Merriam Drive, no longer in use. There is no known existing contamination associated with any of the current or former tank locations, although some contamination was encountered and remediated when the tanks that were closed and removed.

Two areas of concern were discovered during a February 2023 soils investigations. Buried railroad tires, stained soil, and debris were discovered while advancing a boring on 5726 Merriam Drive. This may present foundation issues for the proposed levee along this reach as well as issues with construction of a retention basin proposed on this property. The other issue discovered during the February 2023 soil investigations was petroleum contamination located on City property just north of 5920 Merriam Drive at a

depth of 10 to 11 feet below the ground surface. Sample results indicated total petroleum hydrocarbon (TPH) levels above residential values for mid-range hydrocarbons but below non-residential levels. Volatile organic compound (VOC) analysis results were all non-detect, which suggests the source of contamination is very old and the underlying constituents driving the health-based TPH risk levels have degraded over time.

Overall, the risk of encountering significant environmental/hazardous, toxic, and radioactive waste (HTRW) impacts during construction appear relatively low based on Federal and State environmental records. However, there remains uncertainty about activities that occurred prior to Federal/State regulations being enacted and records kept. An example of this is the debris encountered at 5726 Merriam Drive and the petroleum contamination just north of 5920 Merriam Drive, both discovered during subsurface investigations performed in February 2023. Neither of these sites were expected to have contamination issues based on environmental records. It is not possible nor cost effective to identify all potential environmental subsurface issues along a greater than one mile portion of the channel addressed in this DDR. The density of borings and other investigative techniques would be both schedule and cost prohibitive due to the potentially isolated nature of contamination. Therefore, it is recommended that the contractor be required to develop contingency plans to address unexpected environmental conditions and contingency funding be set aside so unforeseen conditions can be addressed without significant project delays.

As for the two recently discovered sites with environmental concerns, 5726 Merriam Drive and just north of 5920 Merriam Drive, it is recommended additional investigations be performed to characterize the nature and extent of the environmental issues more fully on these properties. This will help to determine the engineering measures that should be included in the design and help refine cost impacts to the project. For the debris at 5726 Merriam Drive, it is likely all debris below the proposed features will need to be removed and disposed of at a permitted facility. For the petroleum contamination encountered just north of 5920 Merriam Drive, based on the TPH levels, it does not appear any remedial activities would need to occur. However, it is good practice to segregate any contaminated soil from clean soil excavated and dispose of it at a permitted landfill rather than reusing the soil on site or at other project locations. In this case it would likely be classified as a special waste that could be disposed of in a Resource Conservation and Recovery Act (RCRA) Subtitle D landfill, which is the type of landfill that accepts typical household waste.

There were no sites of HTRW concerns identified south of 61st Street within the limits of proposed DDR activities.

Refer to Appendix D for more detailed information on specific sites identified and figures showing the location of the properties of concern relative to project features.

4.14 ENVIRONMENTAL CONSIDERATIONS

As described in the FS and Environmental Assessment (EA), approximately 3.6 acres of trees would be removed to implement project designs. Based on the fox squirrel habitat suitability index modeling 2.7 average annual habitat units would be required in compensation. Mitigation requirements could be accomplished through a mitigation bank/in-lieu fee or planted on-site/in-kind within the watershed. In-kind mitigation would be accomplished by planting 185 hard mast producing trees over a 7-acre area, as this type of mitigation would closely replicate the open-canopy mast producing trees with little underbrush habitat that would be lost. This acreage was determined with the fox squirrel habitat suitability index model.

Other pertinent environmental considerations include threatened and endangered species, environmental justice, and cultural resources due to the timespan of the study and EA and current phase of the project. A search of the U.S. Fish and Wildlife Service (USFWS) Information and Planning Consultation webpage indicates the listing of the northern long-eared bat (endangered), Mead's milkweed (threatened), and the monarch butterfly (candidate) in addition to the pallid sturgeon that was assessed in the project's EA. Therefore, additional coordination with the USFWS may be required. Coordination with USFWS may determine that potential impacts to some or all these species are negligible. Additionally, while the

project's EA evaluated potential environmental justice impacts, there have since been updated requirements, guidance, and resources for analyzing these impacts. Reevaluation of environmental justice impacts will likely be necessary to confirm if there are any previously unconsidered effects. Furthermore, a review of archeological sites and/or architectural resources listed in or eligible for the National Register of Historic Places (NRHP) would be required as well as consultation with the Kansas State Historic Preservation Office (SHPO) and relevant federally-recognized Native American Tribes. Once the project area has been defined, field survey may be required to evaluate the potential for intact cultural resources, including architecture.

4.15 PRELIMINARY COST ESTIMATE UPDATE

As part of this FS validation effort, the cost estimate from the FS was escalated to today's prices. The PDT also performed an initial CSRA. As the project reaches 35 percent design completion, the cost estimate will be updated based on findings within this document. See Appendix H for more information.

4.16 ADDITIONAL DATA REQUIREMENTS

Below is a summary of all the additional data the PDT will collect as 35 percent design completion is reached.

4.16.1 Topographic and Real Estate Survey

An AE survey task order has been awarded to provide a topographic survey of the project area, as well as ground penetrating radar (GPR) surveys to properties suspected of having USTs within the project limits. The survey began in February 2023 with a final survey deliverable is expected in July 2023.

As part of this project, USACE will complete the boundary survey of the adjacent properties for use in design.

4.16.2 Subsurface Investigation

Boring logs are available for two historical borings drilled within the current project area. As part of the FS, an Integrated Environmental Assessment included investigation 20 square miles of the Upper Turkey Creek watershed. Several pilot borings were conducted along Turkey Creek with the majority placed near the Kansas River located at U.S. 69 Highway and I-35 and eastward. Two borings drilled in 2002, labeled DC-4 and DC-5, were conducted in the current project area.

These two soil borings revealed mostly silty clay fill and weathered shale to auger refusal on limestone at approximately 13 feet below ground surface. The subsurface investigation continued with rock coring to between 25.9 and 27.6 feet below ground surface.

Additional subsurface soil investigation and laboratory testing of soil samples were recommended as part of the design. These additional soil borings were performed to define the soil stratigraphy and depth to bedrock for the project area for underseepage, slope stability, bearing capacity, and settlement calculations. The borings were also performed to provide Standard Penetration Test (SPT) blow counts to estimate soil strength and disturbed and undisturbed samples for laboratory testing.

It was recommended that 17 soil borings be conducted along the proposed levee system alignment. Borings were to be drilled to auger refusal on bedrock, expected at 12- to 15-feet below grade. Disturbed soil samples were to be taken in glass jars at approximately every five feet in depth. Approximately 20 relatively undisturbed Shelby tube samples were to be taken. Boring locations are shown in Appendix B.4. Boring and lab testing results will be included in future design documents.

4.16.3 Laboratory Testing of Soil Samples

The laboratory test results included in the FS are for soil samples taken from areas that were part of the larger area reviewed during the FS that are outside of the current project area. These tests included

Atterberg limits and grain size distribution. The following laboratory tests were recommended as part of design:

4.16.3.1 Unified Soil Classification System

Performed in accordance with ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes, including testing for grain size, moisture, and Atterberg limits. Soil classification will be used to determine material types for underseepage, slope stability, bearing capacity, and settlement calculations. For underseepage calculations, this will allow permeability ratios and elevations of impervious and pervious blanket materials to be assigned. It is estimated that 34 samples will be tested for USCS soil classification.

4.16.3.2 Moisture Content Test

Performed in accordance with ASTM D2216, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass. Moisture content is used in global slope stability, bearing capacity, and settlement calculations. It is estimated that 34 samples will be tested for moisture content.

4.16.3.3 Soil Dry Unit Weight Test

Determined on relatively undisturbed cohesive soil samples from five-inch diameter Shelby tubes in accordance with the requirements in EM 1110-2-1906, Appendices I and II. Soil unit weight is used in underseepage, global slope stability, bearing capacity, and settlement calculations. It is estimated that ten samples will be tested for unit weight.

4.16.3.4 Consolidated Undrained Triaxial Shear Strength Testing with Pore Pressure Measurements (R bar test)

Performed on relatively undisturbed cohesive soil samples from nominal five-inch diameter Shelby tubes in accordance with ASTM D4767, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils. Consolidated undrained (CU) test results will be used to assign shear strength for cohesive soil for steady state seepage analysis and for rapid drawdown analysis. It is estimated that five CU tests will be performed.

4.16.3.5 Unconsolidated Undrained Triaxial Shear Strength Testing

Performed on relatively undisturbed cohesive soil samples from nominal five-inch diameter Shelby tubes in accordance with ASTM D2850. Unconsolidated undrained (UU) test results will be used to assign shear strength for cohesive soil for end of construction analysis. It is estimated that three UU tests will be performed.

4.16.3.6 Direct Shear

Performed on disturbed cohesionless soil samples, remolded to assumed field density in accordance with ASTM D3080, Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions. Direct shear results will be used to assign internal friction angles for sands for stability analyses. It is estimated that three direct shear tests will be performed.

4.16.3.7 Consolidation Testing

Performed on relatively undisturbed cohesive soil samples from nominal five-inch diameter Shelby tubes in accordance with ASTM D2435, Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading. Consolidation testing will be performed on cohesive soils in the embankment and the pervious blanket to estimate the anticipated settlement due to the levee construction. It is estimated that five consolidation tests will be performed.

4.16.3.8 Unconfined Compression of Rock Core

Performed on rock cores in accordance with ASTM D2938, Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens. Unconfined compression strength of rock will be used for evaluation of various alternatives to support the floodwall on bedrock therefore not relying on the limestone block creek wall for support.

SECTION 5
Criteria

5.1 U.S. ARMY CORPS OF ENGINEERS CRITERIA

Tables 5-1 to 5-3 provide USACE criteria to be used during design.

Table 5-1. Engineering Manuals

Reference Number	Title	Date
EM 1110-2-1416	River Hydraulics	15 Oct 1993
EM 1110-2-1417	Flood-Runoff Analysis	31 Aug 1994
EM 1110-2-1601	Hydraulic Design of Flood Control Channels	30 Jun 1994
EM 1110-1-1804	Geotechnical Investigations	1 Jan 2001
EM-1110-2-1902	Slope Stability	31 Oct 2003
EM-1110-2-1906	Laboratory Soils Testing	20 Aug 1986
EM 1110-2-1913	Design and Construction of Levees	30 Apr 2000
EM 1110-2-2100	Stability Analysis of Concrete Structures	1 Dec 2005
EM 1110-2-2104	Strength Design for Reinforced Concrete Hydraulic Structures	30 Nov 2016
EM 1110-2-2107	Design of Hydraulic Steel Structures	1 Aug 2022
EM 1110-2-2502	Floodwalls and Other Hydraulic Retaining Walls	1 Aug 2022
EM 1110-2-2504	Design of Sheet Pile Walls	31 Mar 1994
EM 1110-2-2902	Conduits, Culverts, and Pipes associated with Dams and Levee Systems	31 Dec 2020
EM 1110-2-2906	Design of Pile Foundations	15 Jan 1991
EM 1110-2-3104	Structural and Architectural Design of Pumping Stations	30 Jun 1989

Table 5-2. Engineering Regulations

Reference Number	Title	Date
ER 1110-2-1405	Hydraulic Design for Local Flood Protection Projects	30 Sep 1982
ER 1110-1-1807	Drilling in Earthen Embankment Dams and Levees	31 Dec 2014

Table 5-3. Engineering and Construction Bulletin

Reference Number	Title	Date
ECB 2018-14	Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects, Revision 2	19 Aug 2022
ECB 2019-8	Managed Overtopping of Levee Systems	24 Apr 2019

5.2 FEDERAL AND STATE CRITERIA

Tables 5-4 to 5-10 provide a list of non-USACE criteria that will be used during design.

Table 5-4. State of Kansas Criteria

	Title	Date
	Kansas Department of Transportation Road Design Manual	May 2014

Table 5-5. City of Merriam, Kansas, Criteria

	Title	Date
	City of Merriam Manual of Infrastructure Standards	2019
	City of Merriam Code of Ordinances, Chapter 26-Environment	Current Edition as of March 2023

Table 5-6. American Concrete Institute

Reference Number	Title	Date
ACI 318-19	Building Code Requirements for Structural Concrete	2022

Table 5-7. American Institute of Steel Construction

Reference Number	Title	Date
ANSI/ AISC 360-16	Specification for Structural Steel Buildings	2016

Table 5-8. American Society of Civil Engineers

Reference Number	Title	Date
ASCE/SEI 7-16	Minimum Design Loads and Associated Criteria for Buildings and	2017

	Other Structures	

Table 5-9. American Public Works Association

Reference Number	Title	Date
APWA 5600	Storm Drainage Systems & Facilities	2011

Table 5-10. National Oceanic and Atmospheric Administration

Reference Number	Title	Date
NOAA Atlas 14	Point Precipitation Frequency, Volume 8, Version 2.0: Midwestern States (Colorado, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Oklahoma, South Dakota, Wisconsin)	2013
NOAA TP-40	Rainfall Frequency Atlas of the Eastern United States for Duration from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years	Jan 1963

THIS PAGE IS INTENTIONALLY BLANK.

List of Acronyms and Abbreviations

1D	one-dimensional
2D	two-dimensional
ACP	Auger Cast Piles
AE	Architect-Engineering
AEP	annual exceedance probability
APWA	American Public Works Association
CSRA	cost and schedule risk analysis
CU	consolidated undrained
DIP	ductile iron pipe
EA	Environmental Assessment
ECB	Engineering and Construction Bulletin
EM	engineering manual
FEMA	Federal Emergency Management Agency
FRM	Flood Risk Management
FS	Feasibility Study
GPR	ground penetrating radar
H&H	hydraulics and hydrology
HEC	Hydrologic Engineering Center
HEC-HMS	Hydrologic Modeling System
HEC-RAS	River Analysis System
HTRW	hazardous, toxic, and radioactive waste
KDHE	Kansas Department of Health and Environment
LB	left bank
LERRD	lands, easements, rights-of-way, relocations, and disposal area
LiDAR	light detection and ranging
LF	load factor
LL	life loss
MDD	Merriam Drainage District
NAVD 88	North American Vertical Datum of 1988
NED	National Economic Development
NOAA	National Oceanic and Atmospheric Administration
NRHP	National Register of Historic Places
O&M	operations and maintenance
pcf	pounds per cubic foot
PDT	project delivery team
PE	polyethylene
psf	pounds per square foot
psi	pounds per square inch

PVC	polyvinyl chloride
RB	right bank
RCB	reinforced concrete box
RCP	reinforced concrete pipe
RCRA	Resource Conservation and Recovery Act
RM	river mile
SF	shallow footings
SHPO	State Historical Preservation Office
SPT	Standard Penetration Test
TOL	top of levee
TP	Technical Paper
TPH	total petroleum hydrocarbon
U.S.	United States
USACE	U.S. Army Corps of Engineers
USFWS	U.S. Fish and Wildlife Service
UST	underground storage tanks
UU	unconsolidated undrained
VE	value engineering
VOC	volatile organic compound
WSE	water surface elevation

THIS PAGE IS INTENTIONALLY BLANK.

APPENDIX A
Plates from Feasibility

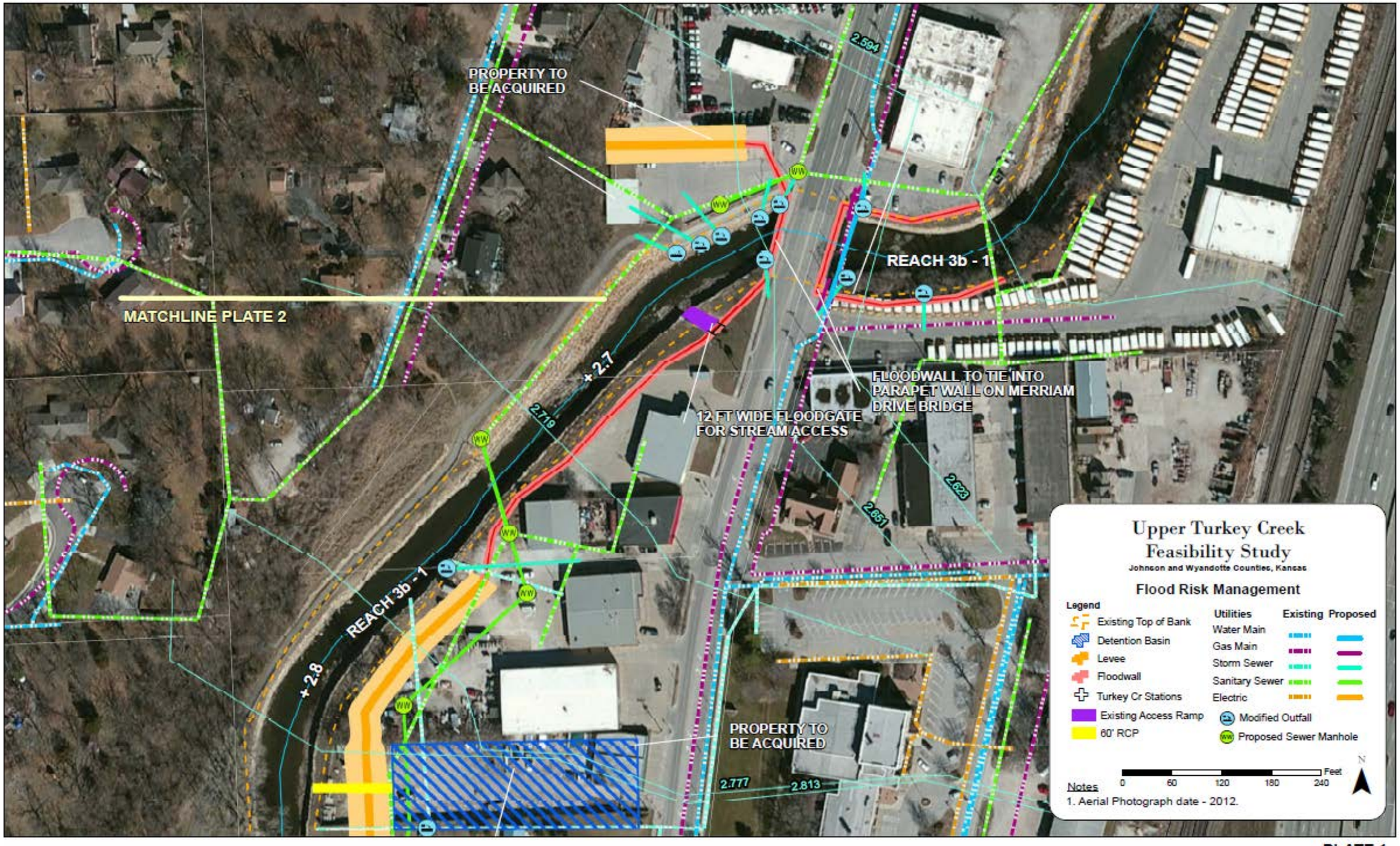


Figure 7-1: Recommended Plan (Alternative 2d) - Plate 1

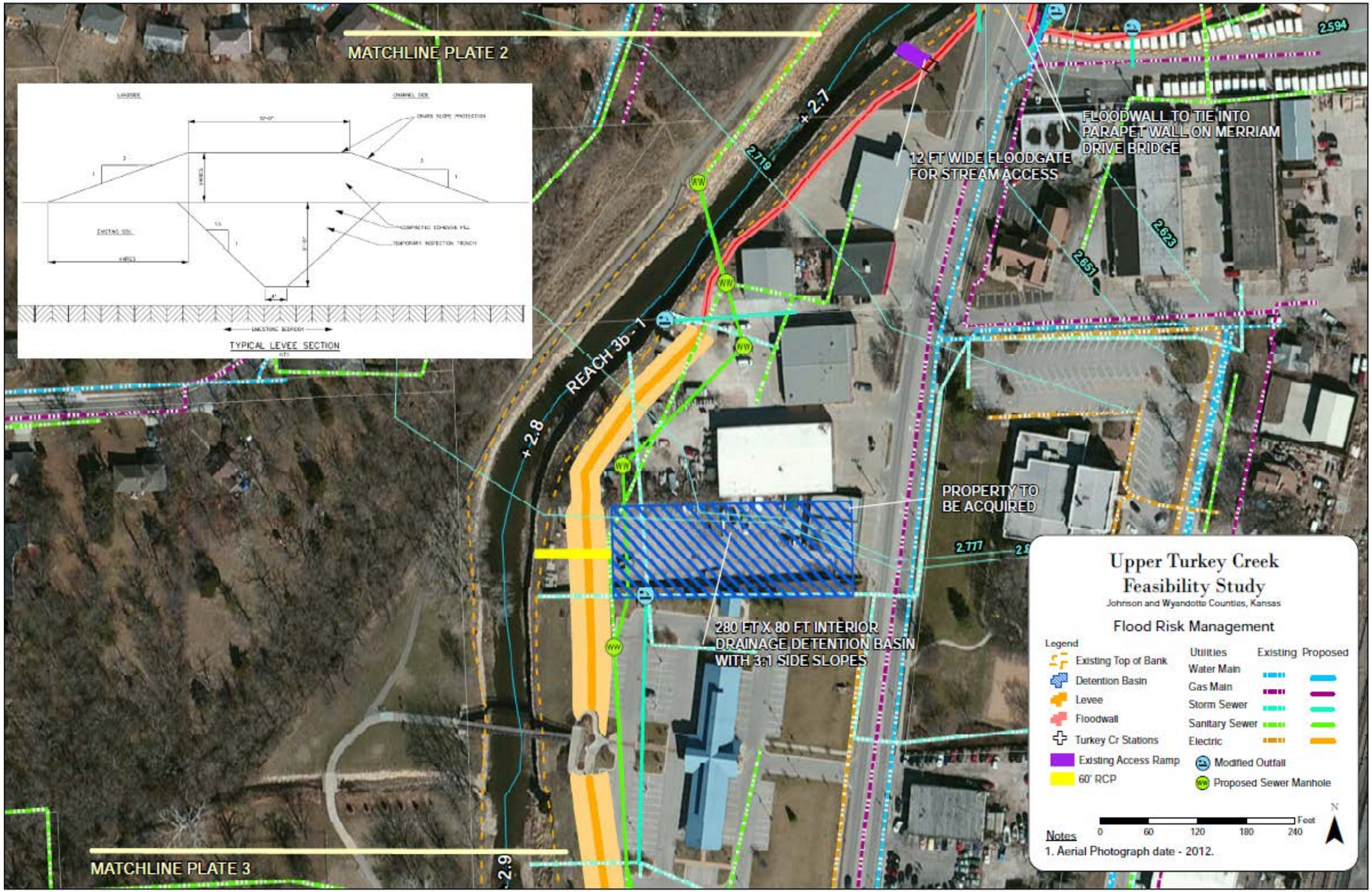


Figure 7-2: Recommended Plan (Alternative 2d) - Plate 2

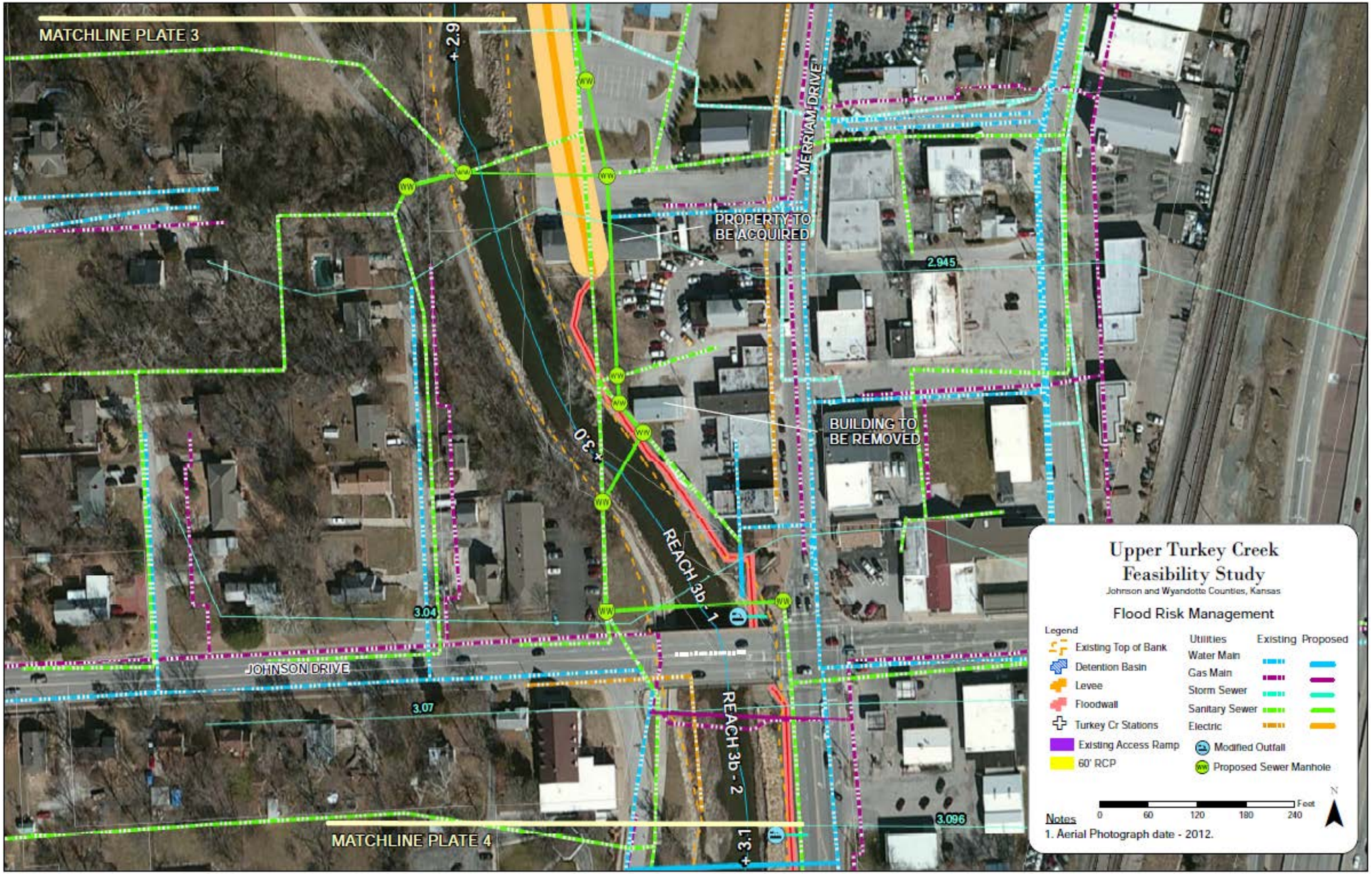


Figure 7-3: Recommended Plan (Alternative 2d) - Plate 3

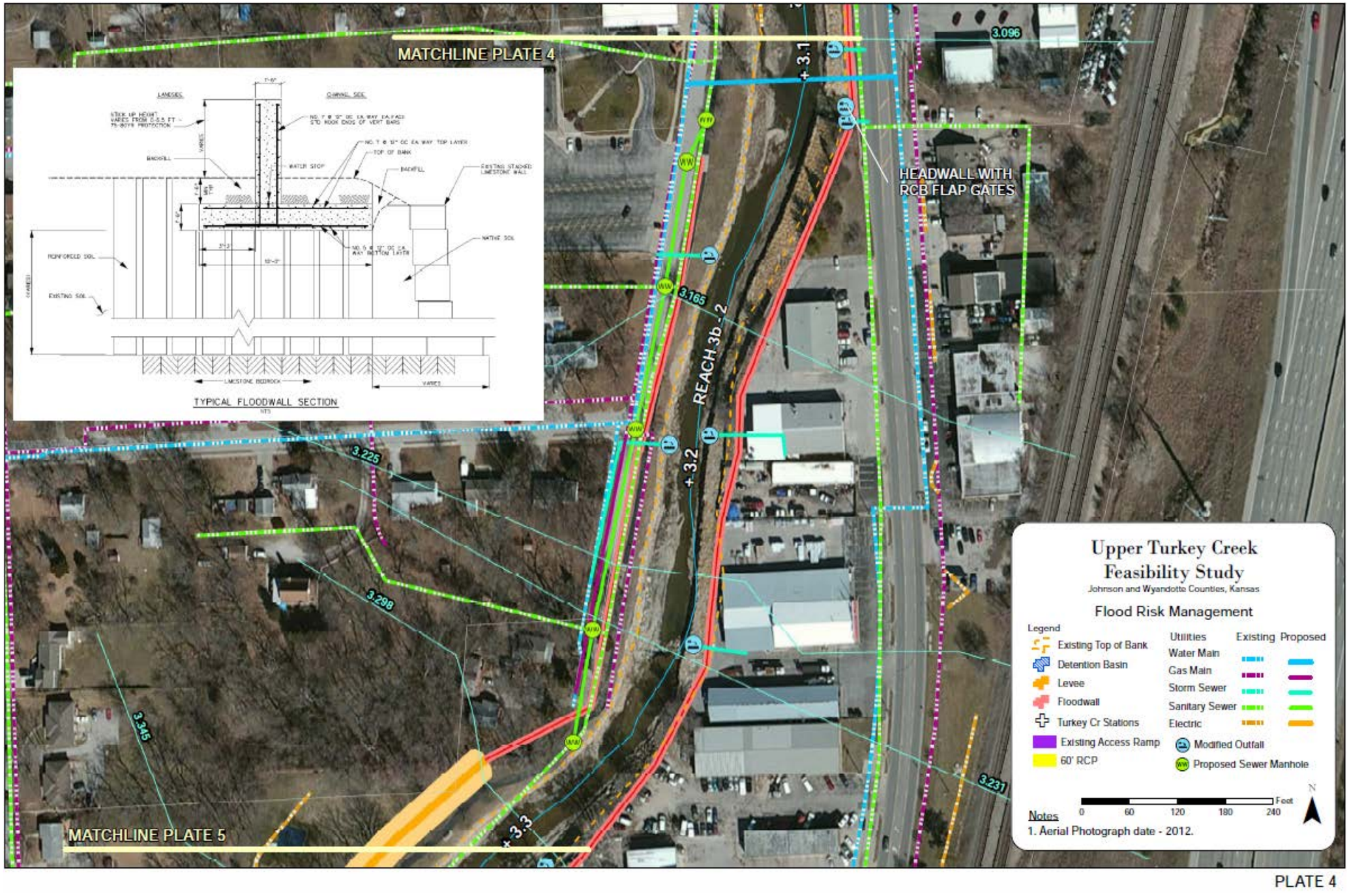


Figure 7-4: Recommended Plan (Alternative 2d) - Plate 4

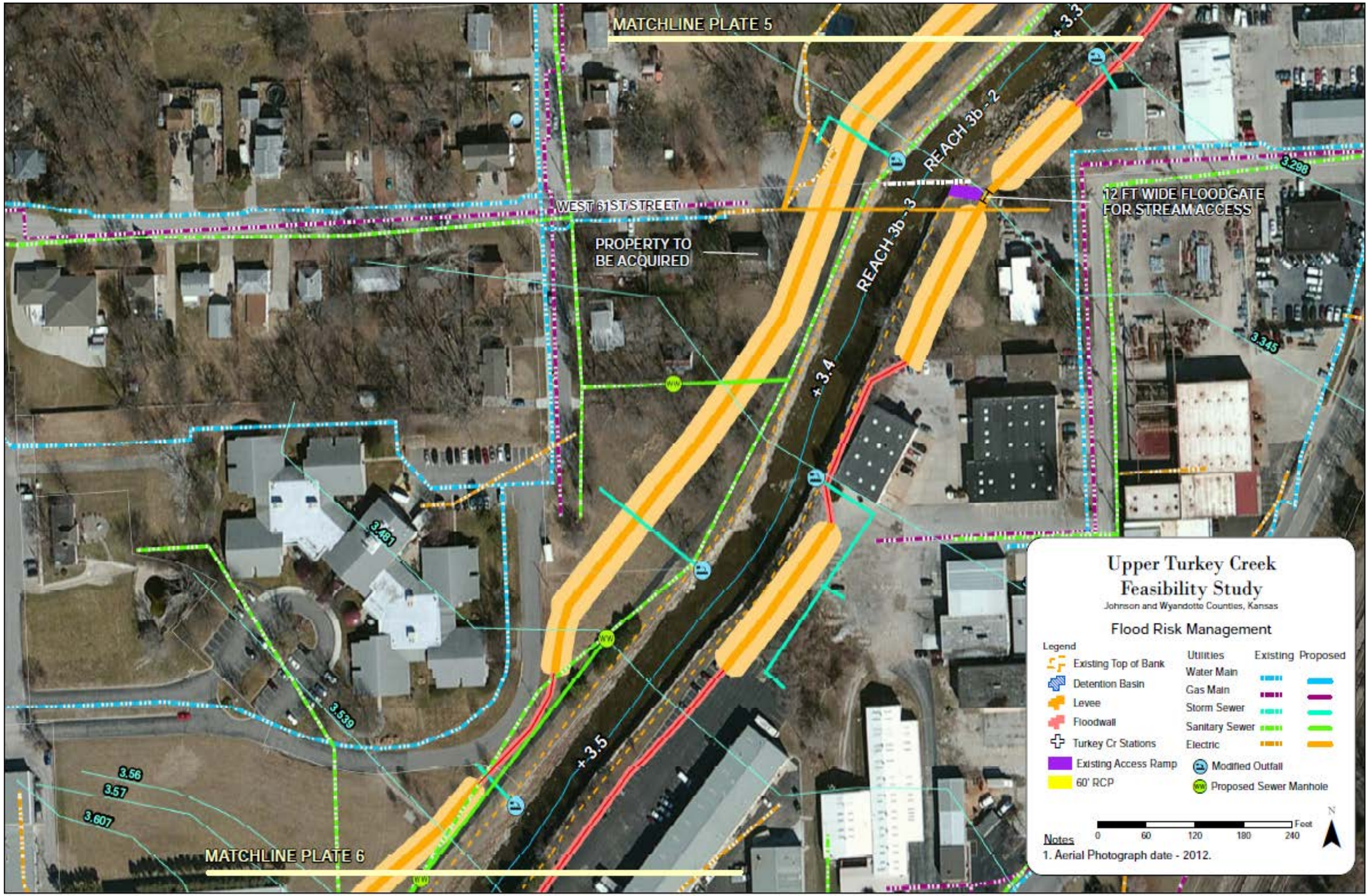


Figure 7-5: Recommended Plan (Alternative 2d) - Plate 5

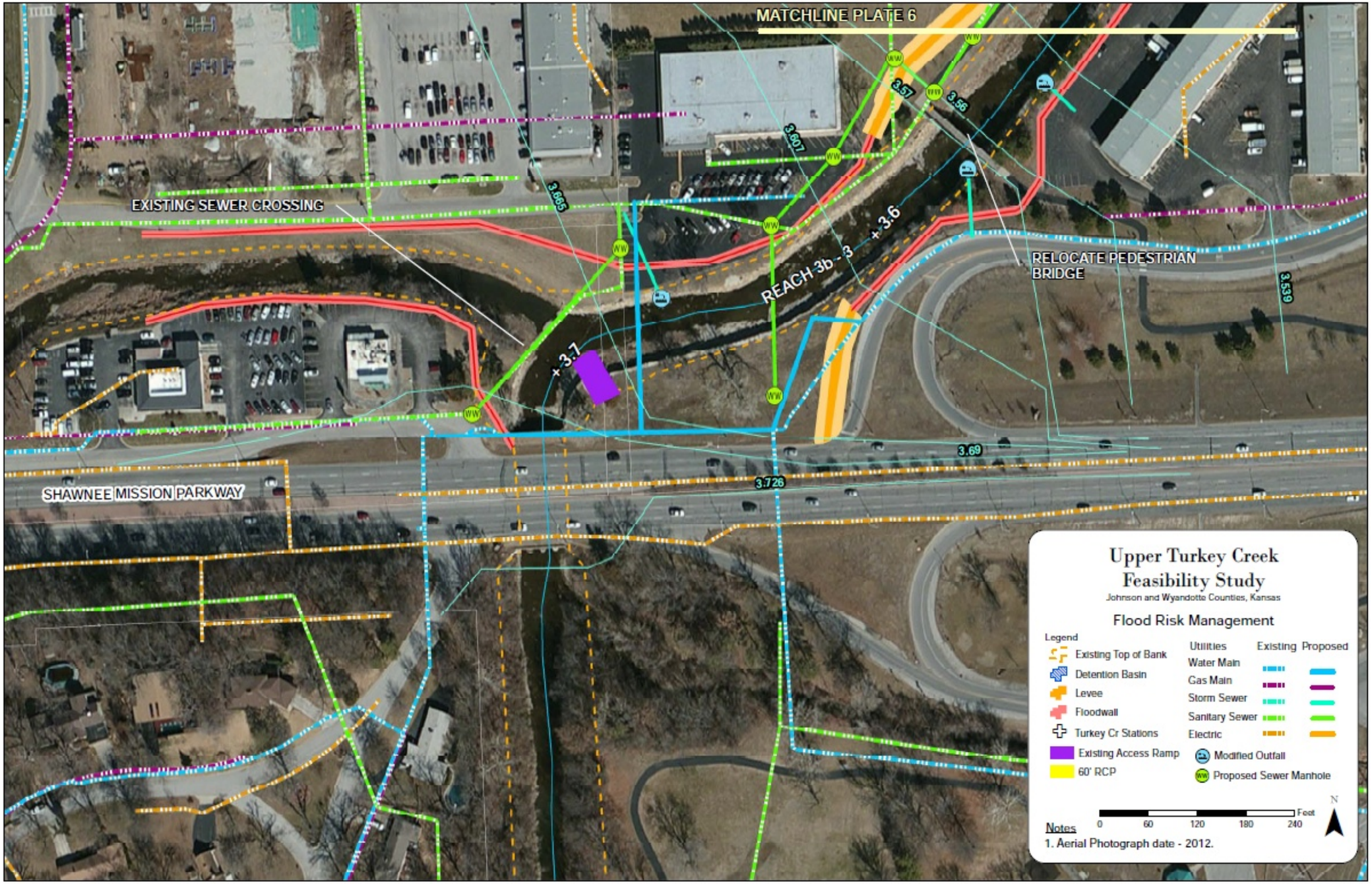


Figure 7-6: Recommended Plan (Alternative 2d) - Plate 6

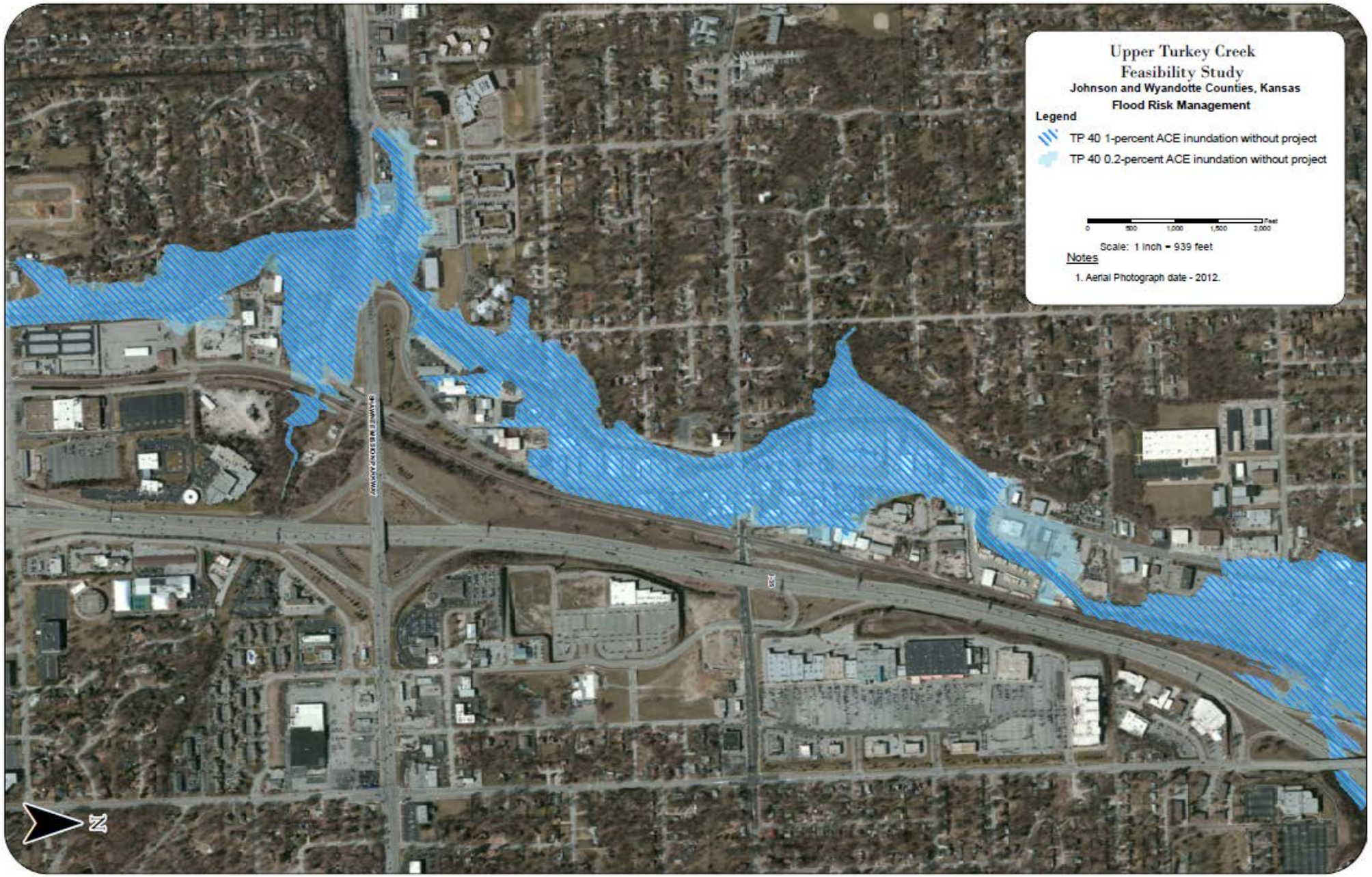
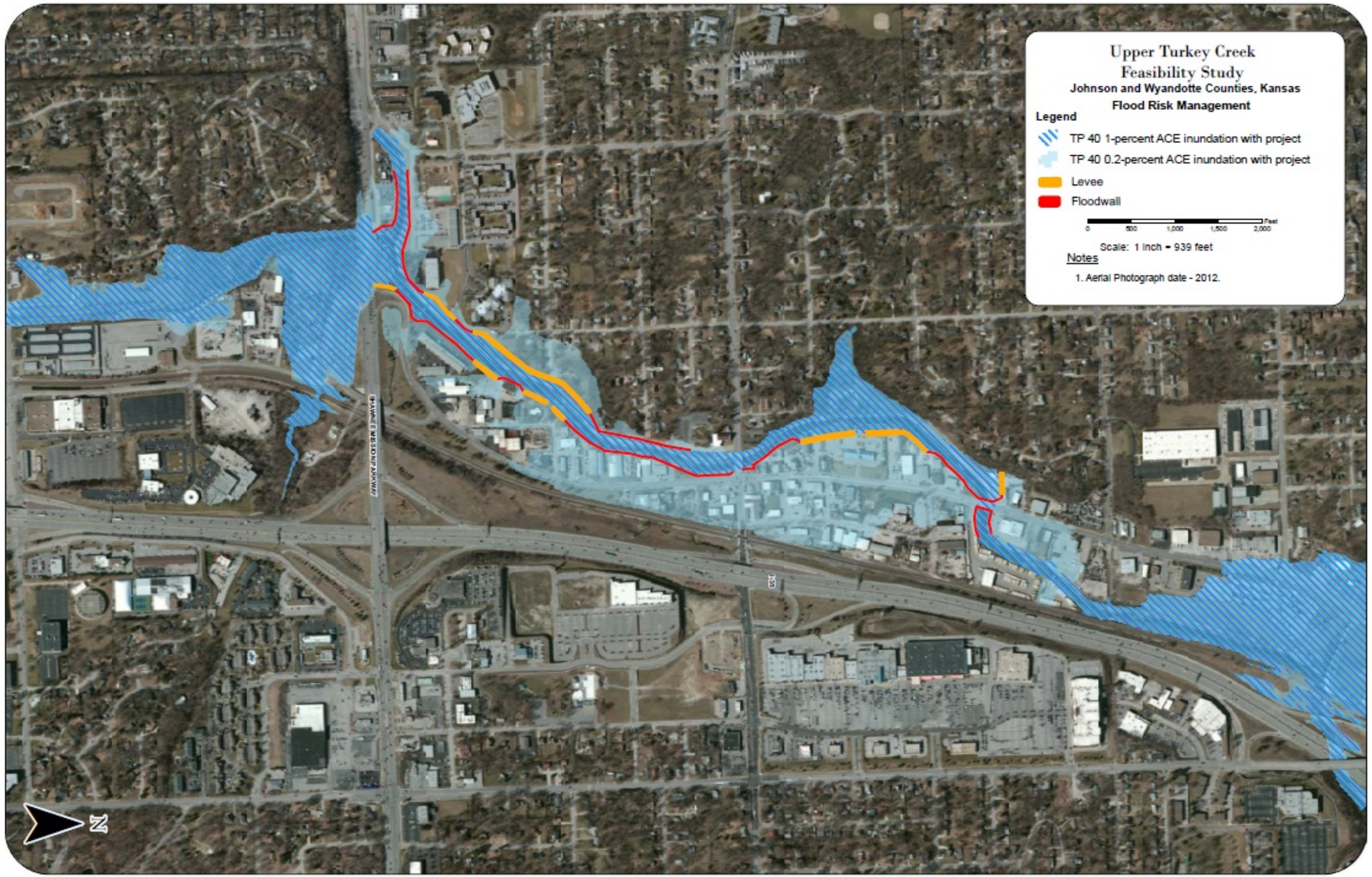


Figure 7-7: Inundation Map Without Project (TP-40) - Plate 7



7-8: Inundation Map With Project (TP-40) - Plate 8

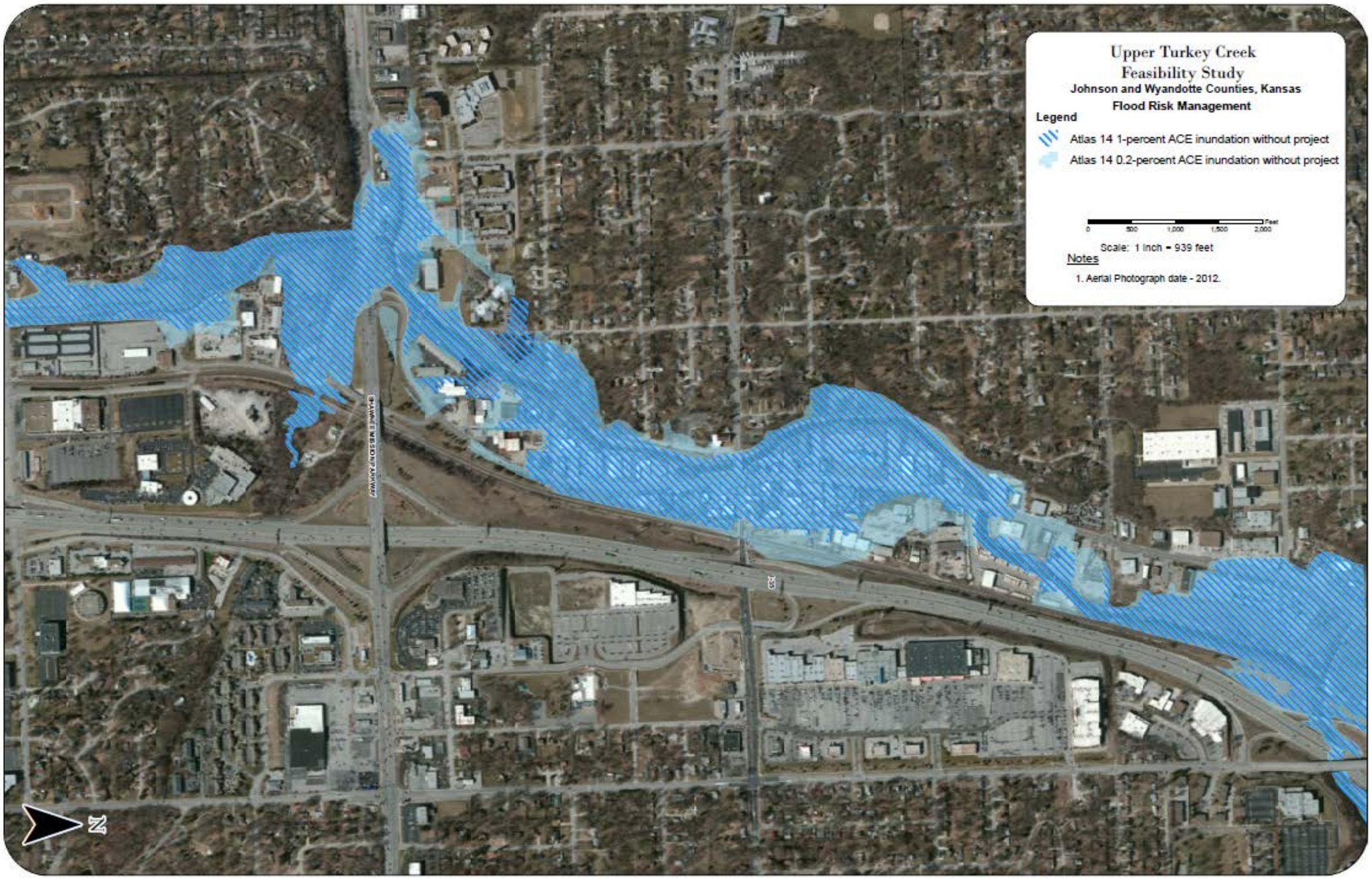


Figure 7-9: Inundation Map Without (Atlas 14) - Plate 9

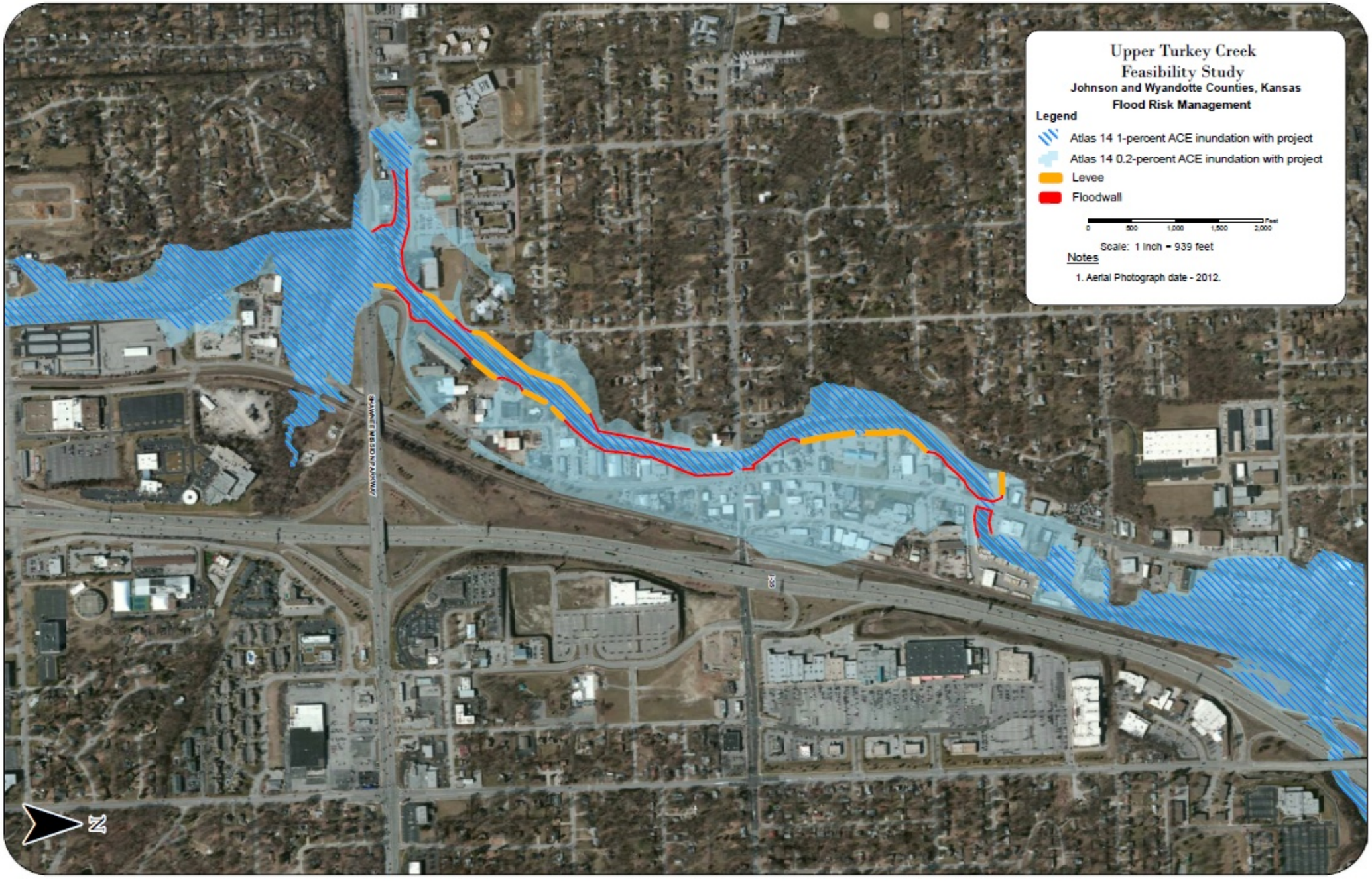


Figure 7-10: Inundation Map With (Atlas 14) - Plate 10

THIS PAGE IS INTENTIONALLY BLANK.

APPENDIX B
Geotechnical

APPENDIX B – GEOTECHNICAL

B.1 Underseepage
B.2 Stability
B.3 Settlement
B.4 Boring Locations

B.1 Underseepage Notes

Analyzed vertical underseepage gradient for proposed Upper Turkey Creek levee based on current information and assumptions as detailed below:

Existing Topography – Initially used cross section information from AE Feasibility Study analysis at River Mile 2.813 as that was judged in the study to be a critical area. Information was difficult to interpret, so supplemented with Google Earth at approximate River Mile 2.825 and other assumptions. Per Google Earth, prevailing landside grade in that area is approximately EL 905 ft.

Levee Cross Section – Feasibility Study stated that max levee height is 6 feet (assumed 6 feet from prevailing landside grade). Assumed 6 ft tall levee with 10 ft crown and 1V:3H side slopes down to EL 905. Measured 16 feet from EL 905 on the riverside to the creek bank.

Levee Subsurface Profile – Based on Borings DC-4 and DC-5, the two borings completed in the area of the current project during Feasibility, impervious blanket is continuous to bedrock. In other areas, a zone of foundation sand may exist between the impervious clay blanket and bedrock. Even if the impervious blanket is continuous to bedrock throughout the levee unit, vertical underseepage pressures may build up through joints and seams in the bedrock. This may be modeled as a zone of foundation sand. For sensitivity checks, moved blanket bottom up in profile, adding pervious sand blanket.

Underseepage was analyzed using the NWK underseepage spreadsheet based on blanket theory. SEEP/W seepage analyses were calibrated to the results of the spreadsheet and then used as parent analyses to provide pore water pressures for SLOPE/W stability analyses.

Table B.1-1: Underseepage Spreadsheet Parameters

Blanket Unit Weight	115 pcf, based on value used for similar soil at other projects
Top of Levee Elevation	911 ft, set to 6 ft (max levee height) above prevailing landside grade of 905 feet at cross section
Riverside Ground Elevation	905 ft at riverside levee toe, set to match prevailing landside elevation
Landside Ground Elevation	905 ft, prevailing landside grade
Landside Tailwater Elevation	905 ft, same as landside ground elevation
River Blanket Bottom Elevation	Based on two borings, impervious blanket is continuous to bedrock. For sensitivity checks, moved blanket bottom up in profile, adding pervious sand blanket.
Land Blanket Bottom Elevation	Based on two borings, impervious blanket is continuous to bedrock. For sensitivity checks, moved blanket bottom up in profile, adding pervious sand blanket.
Top of Bedrock Elevation	Based on two borings, bedrock is located 12.5-12.9 ft below grade (EL 895.71 and 915.15). Used 13 ft below landside grade at cross section (EL 905-13 = EL 892).
Permeability Ratio	Boring logs showed silty clay. Using 400 based on 1962 MRD conference values.
Riverside Seepage Length (L1)	Measured approximately 16 ft from EL 905 on riverside to creek bank
Levee Seepage Length (L2)	For 6 ft max tall levee with 10 ft crest and 1V:3H side slopes, levee width of 46 ft

Table B.1-2: Sensitivity to Impervious/Pervious Blanket Thickness

Impervious Blanket Thickness	Pervious Blanket Thickness	Vertical Underseepage Gradient FS
12	1	3.19
11	2	2.56
10	3	2.20
9	4	1.92
8	5	1.67
7.5	5.5	1.56
7	6	1.45
6	7	1.25
5	8	1.05
4.5	8.5	0.95

Levee Unit Name

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

Case Analyzed - Water at Levee Top

ROWS IN THIS TAB CAN BE COPIED DOWN AS MANY TIMES AS DESIRED. HOWEVER, THE WELL ANALYS

Blanket Unit Weight = 115.0 pcf (saturated)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	Land Side Tailwater Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{br}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{br}	Impervious Blanket Thickness (ft) Levee z_{bo}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d
UTC River Mile 2.825	911.0	905	905	905	897	897	892	6.0	400	400	8	8	8	5

6' max tall levee
assumed 6' above prevailing LS grade of 905

can have sand foundation of up to 5.5' and top of levee loading and still have vert underseepage gradient FS of ~1.6

Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_r	Factor c_l	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_o	Head at Toe (elevation msl) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Effective Stress Uplift / Blanket Rupture (i_c/i_o)	average horizontal gradient between entrance and levee toe (i_h)
16	46	infinity	0.007906	0.007906	15.92	126.49	4.03	909.03	0.50	0.84	1.67	0.10

Figure B.1-1: Underseepage Analysis

B.2 Stability Notes

Evaluated levee slope stability for one assumed cross section

Existing Topography – Initially used cross section information from AE Feasibility Study analysis at River Mile 2.813 as that was judged in the study to be a critical area. Information was difficult to interpret, so supplemented with Google Earth at approximate River Mile 2.825 and other assumptions. Per Google Earth, prevailing landside grade in that area is approximately EL 905 ft.

Levee Cross Section – Feasibility Study stated that max levee height is 6 feet (assumed 6 feet from prevailing landside grade). Assumed 6 ft tall levee with 10 ft crown and 1V:3H side slopes down to EL 905.

Creek Bank – Measured 16 feet from EL 905 on the riverside to the creek bank at EL 892 for cross section on Google Earth. Used that for underseepage analysis and for steady state seepage check of landside slope. For sudden drawdown evaluation of riverside slope, checked 1V:1H, 1V:2H, and 1V:3H slopes from riverside levee toe to creek bed for sensitivity.

Levee Subsurface Profile – Based on Borings DC-4 and DC-5, the two borings completed in the area of the current project during Feasibility, impervious blanket is continuous to bedrock. Checked sensitivity of underseepage calculations to thinner impervious blanket (adding foundation sand between impervious blanket and bedrock). Results indicated satisfactory underseepage factor of safety could be obtained for up to 5.5 feet of foundation sand. Assumed 5 feet of foundation sand between bedrock and impervious blanket for stability calculations. The foundation sand did not appear to significantly impact the stability analyses as far as resistance to sliding. The higher vertical underseepage gradient due to the thinner impervious blanket; however, does have a significant impact on slope stability.

Underseepage Calibration – Using 5 feet of foundation sand results in hydraulic grade line (HGL, or water total head, as defined in SEEP/W) at bottom of blanket at landside levee toe of 909.03 ft on the underseepage spreadsheet. Calibrated the Geoslope model by adjusting saturated horizontal hydraulic conductivity for impervious blanket and impervious fill in SEEP/W until this HGL (water total head contour) was obtained at the landside toe. For this cross section, hydraulic conductivity of 7e-06 ft/sec was used for clay.

SEEP/W Boundary Conditions – A water total head equal to the top of new levee is applied on the riverward slope, and a water total head equal to the landside grade is applied to the landside edge of the model. A potential seepage face is applied to the landward levee slope with water rate equal to 0 ft^3/sec, and a no-flow boundary is placed at bedrock with a water rate equal to 0 ft^3/sec. Added a layer of foundation sand 0.1 ft thick above bedrock in the channel so that could have water total head boundary equal

to top of new levee in the channel and on riverside slope and still have no-flow boundary in bedrock on channel bed.

Riverward Extent of Model – Usually taken at least halfway into a river. For this analysis, used 40 feet, approximate full width of channel in this area.

Landward Extent of Model – Taken to 2,500 feet landward of the new levee centerline. Extending the boundary farther appears to have no influence on the calculation results.

Hydraulic Conductivity of Sand – Used 0.003 ft/sec. This value is typically used for NWK levees such as the Argentine Levee Raise on the Kansas River (see Argentine DDR), the floodwall modification project at South Topeka on the Kansas River (see South Topeka DDR), and for design of Missouri River levee L-476 (see DM-1, p. I-15). This is also within the typical range used in Das Principles of Geotechnical Engineering, Table 5.1, p. 164, 1998.

Permeability Ratio Check – A permeability ratio of K_f/K_b of 400 typical of ML-CL materials was used in the underseepage blanket theory spreadsheet. This equates to hydraulic conductivity of 7.5e-06 ft/sec for impervious blanket (K_b), using 0.003 ft/sec for sand (K_f). The 7e-06 ft/sec used for calibrating SEEP/W to the blanket theory spreadsheet results equates to a permeability ratio of 428.

Sand Drains – A 2 ft sand drain was placed at the base of the landside levee slope to improve the steady state stability factor of safety to the required 1.4.

Steady State Material Properties – Properties used for this preliminary analysis are based on soil properties used in the Argentine Levee Raise design. Material properties will be adjusted after lab testing is completed. Consolidated Undrained (CU) triaxial shear strength testing with pore pressure measurements will be used for selection of drained shear strengths for in-situ soils for steady state analysis. As it is not planned to identify borrow sources during design, strength properties for impervious fill will be assumed. The properties used in the current analysis are as follows:

Table B.2-1: Steady State Material Properties

Material	Unit Weight (pcf)	Cohesion (pcf)	Phi (degrees)
Impervious Fill (Embankment)	120	0	26
Impervious Blanket	115	0	26
Foundation Sand	115	0	32

End of Construction Material Properties – End of construction case has not been analyzed for this preliminary report. Unconsolidated Undrained (UU) triaxial shear strength testing will be used for selection of undrained shear strengths for in-situ soils for end of construction analysis.

Sudden Drawdown Material Properties - Properties used for this preliminary analysis are based on soil properties used in the Argentine Levee Raise design. Material properties will be adjusted after lab testing is completed. The effective stress plots of the Consolidated Undrained (CU) triaxial shear strength testing with pore pressure measurements will be used for selection of drained shear strengths for in-situ soils for steady state analysis/first stage calculations (Basic Strength on SLOPE/W). The total stress plots of the same test will be used for selection of undrained shear strengths for second stage calculations (R Envelope on SLOPE/W).

Table B.2-2: Sudden Drawdown Material Properties

Material	Unit Weight (pcf)	Basic		R Envelope	
		Cohesion (pcf)	Phi (degrees)	Cohesion (pcf)	Phi (degrees)
Impervious Fill (Embankment)	120	0	26	17	300
Impervious Blanket	115	0	26	17	300
Foundation Sand	115	0	32		

Sudden Drawdown Water Surfaces – Used top of levee for PZ1 and landside grade for PZ2.

SLOPE/W Settings – Pore water pressure conditions are taken from the parent SEEP/W analysis. Entry and exit point ranges were defined for the slip surface.

(Under Define Project/Advanced)
Analysis Type: Spencer
Minimum slip surface depth: 5 ft
Number of slices: 100
Maximum number of iterations: 100
Tolerable difference in FS: 0.001

(Under Define Slip Surface Entry and Exit Range)
Number of Increments over Range: 20
Number of Radius Increments: 20

Defining Slip Surfaces – Expanded limits of slip surface range if critical slip surface was at the edge of the range. After critical surface was determined, contracted limits of search around the critical surface to refine the search.

Failure Definition – For preliminary analyses, defined steady state failure as a slip surface including any portion of the levee crest. Also defined sudden drawdown failure as a slip surface including any portion of the crest.

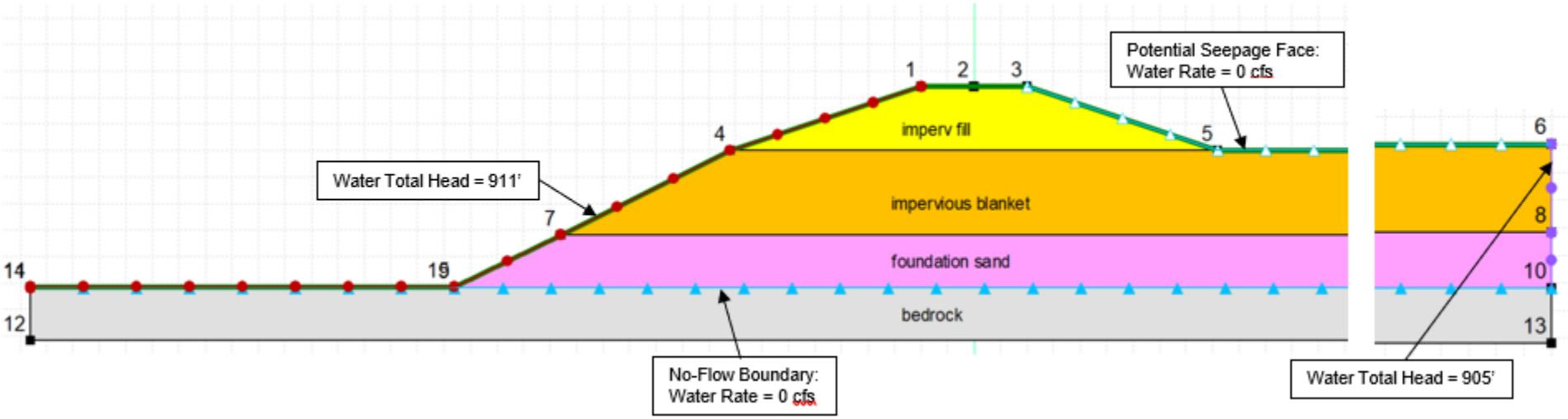


Figure B.2-1: Steady State Seepage – SEEP/W Definition

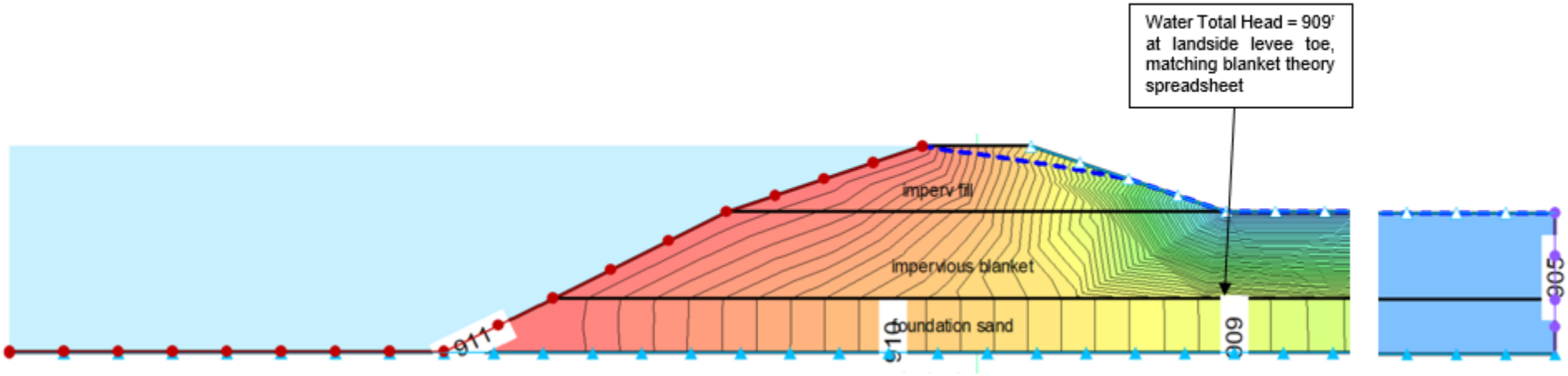


Figure B.2-2: Steady State Seepage – SEEP/W Results

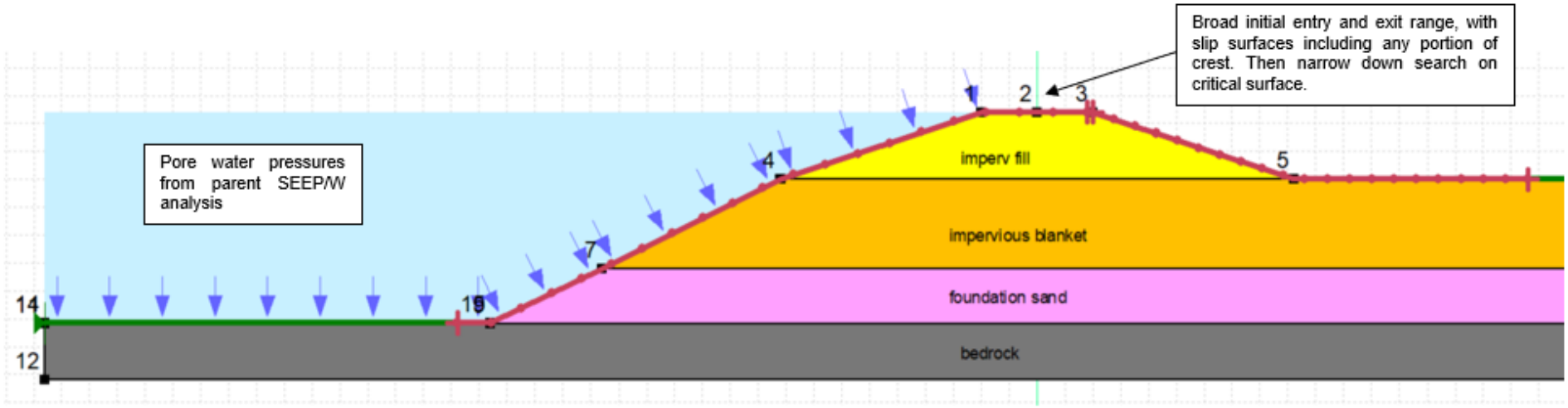


Figure B.2-3: Steady State Seepage – SLOPE/W Definition

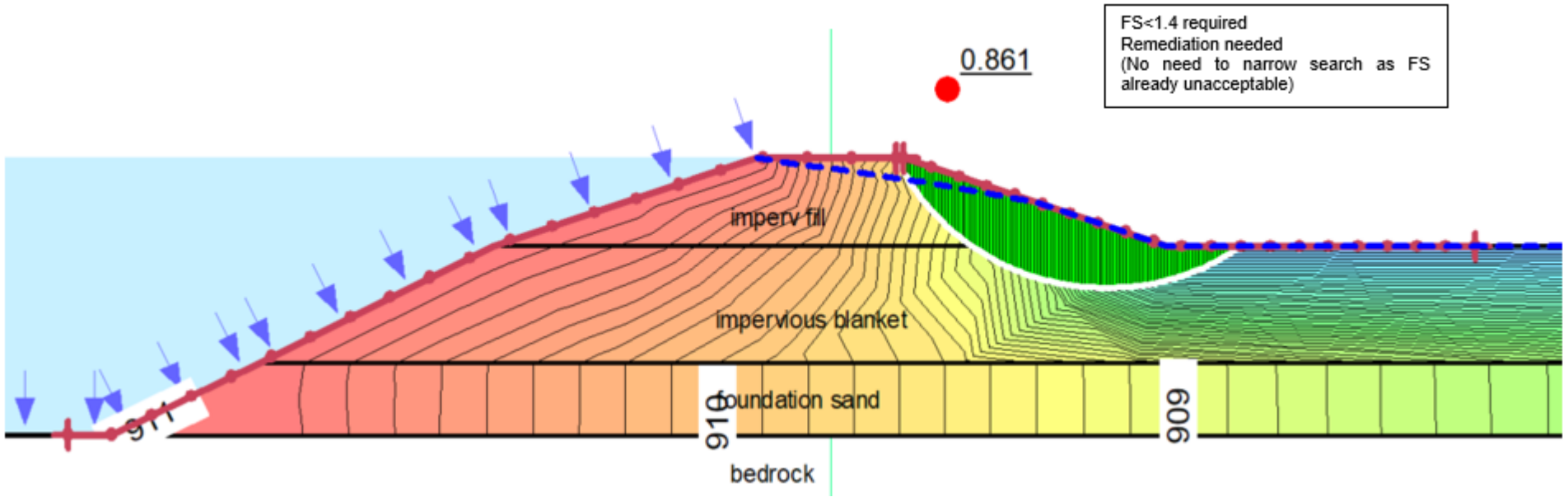


Figure B.2-4: Steady State Seepage – SLOPE/W Results

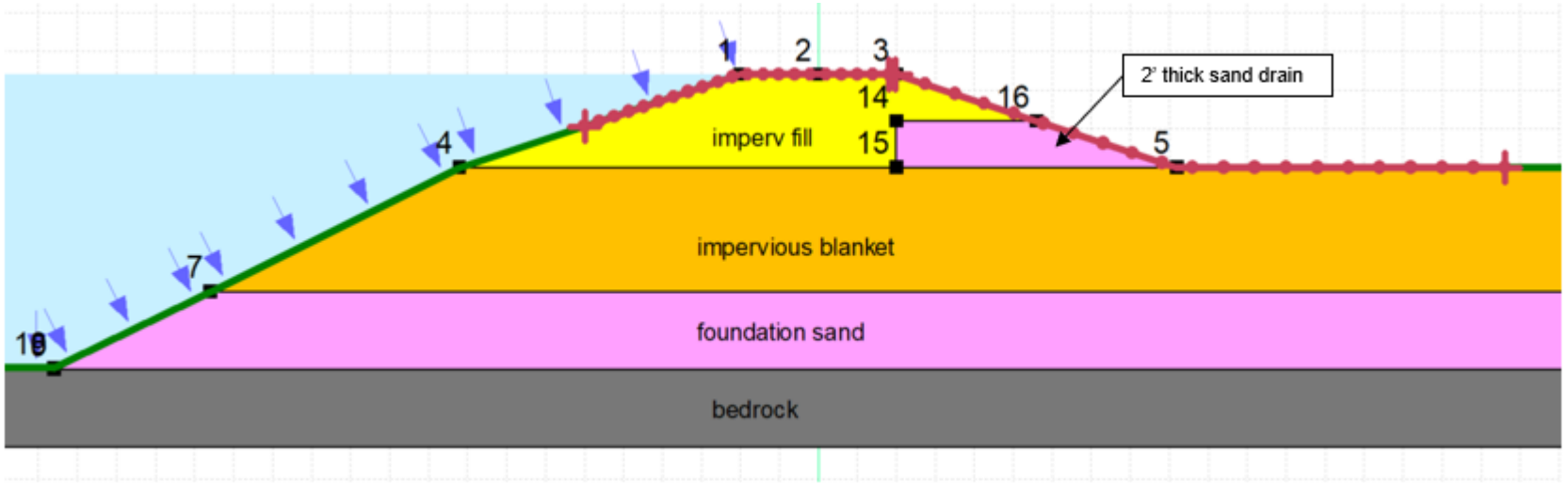


Figure B.2-5: Steady State Seepage – SLOPE/W Definition – Try 2' thick sand drain

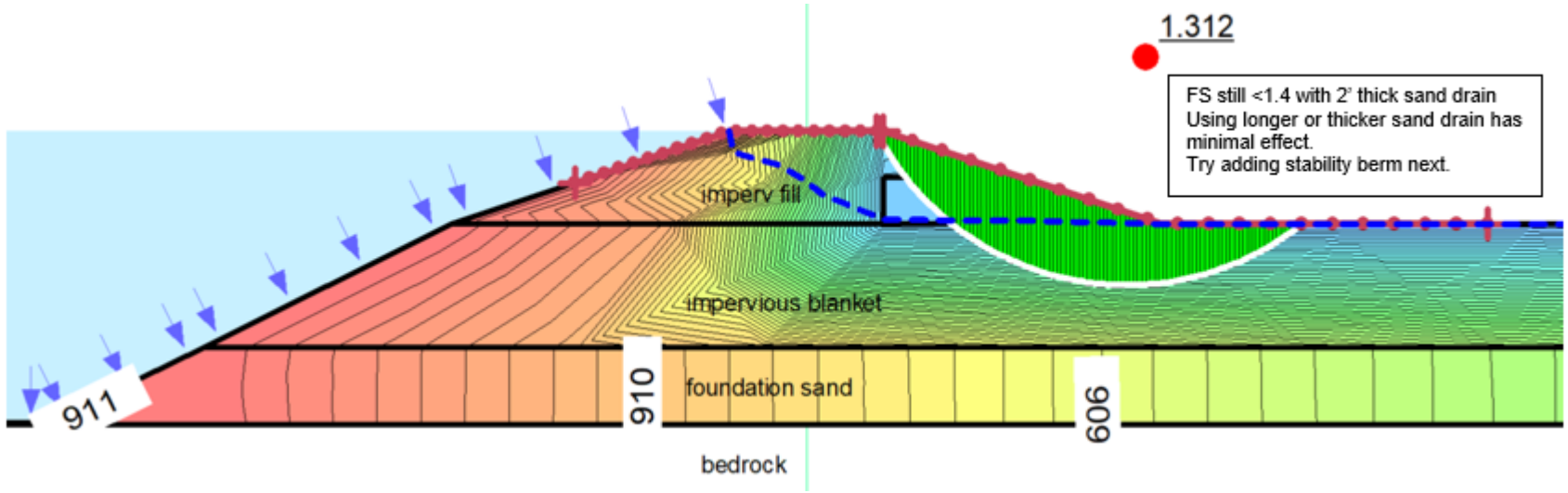


Figure B.2-6: Steady State Seepage – SLOPE/W Results – Try 2' thick sand drain

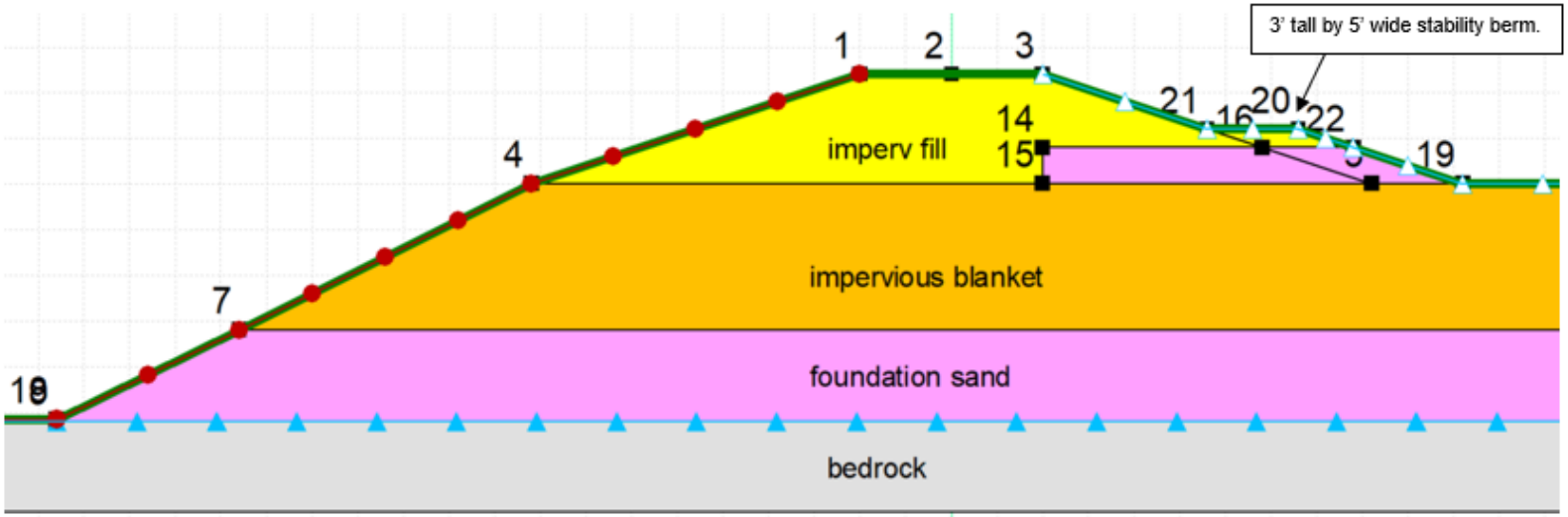


Figure B.2-7: Steady State Seepage – SLOPE/W Definition – Try 2' thick sand drain and 3'x5' stability berm

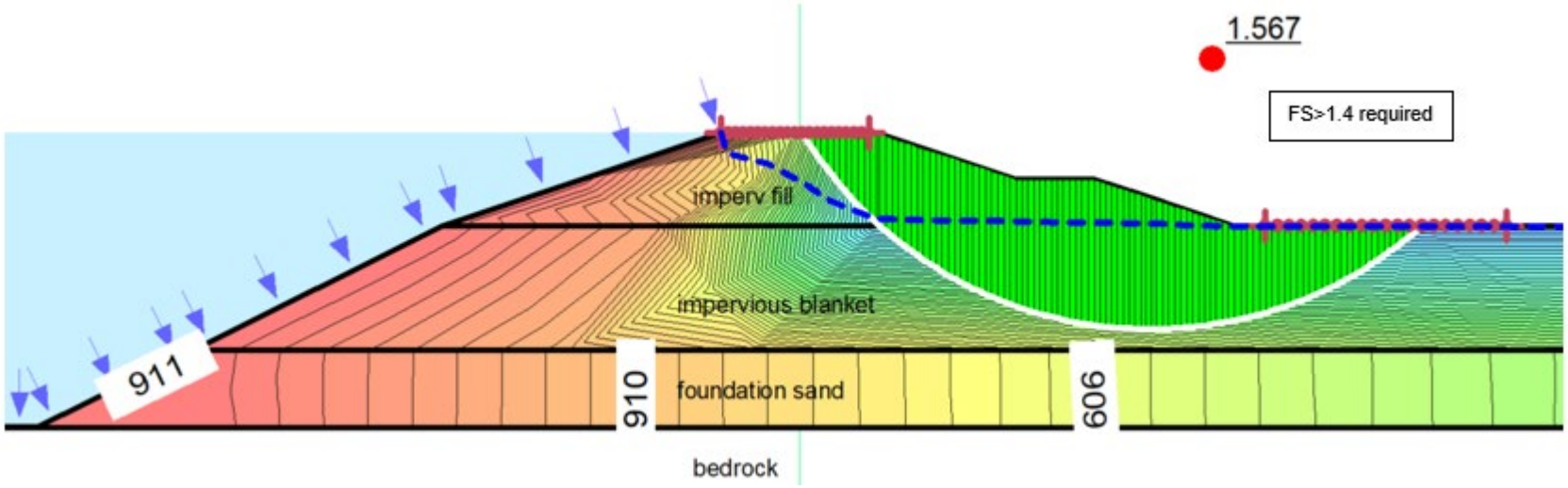


Figure B.2-8: Steady State Seepage – SLOPE/W Results – Try 2' thick sand drain and 3'x5' stability berm

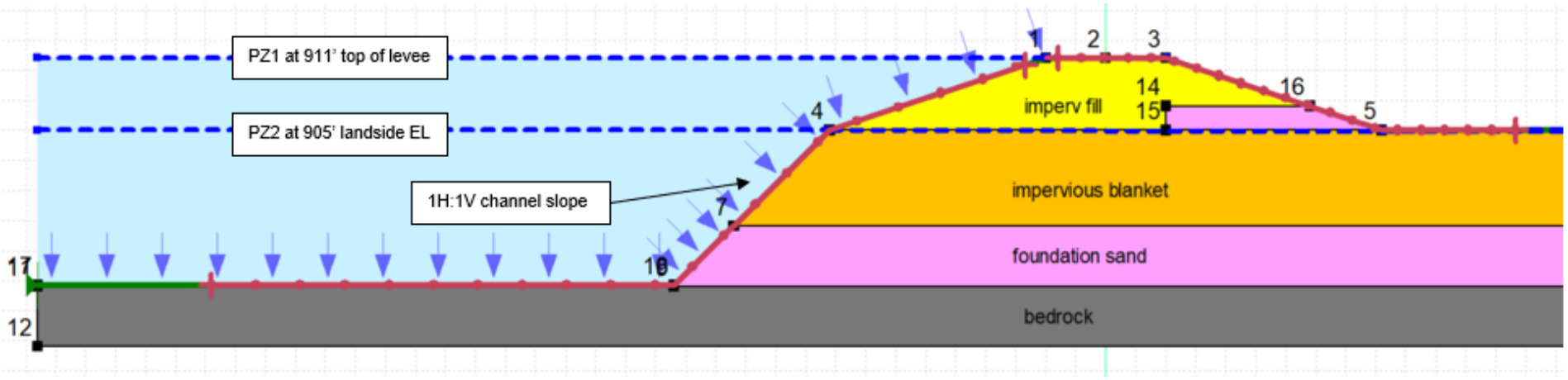


Figure B.2-9: Sudden Drawdown with 1V:1H Channel – SLOPE/W Definition

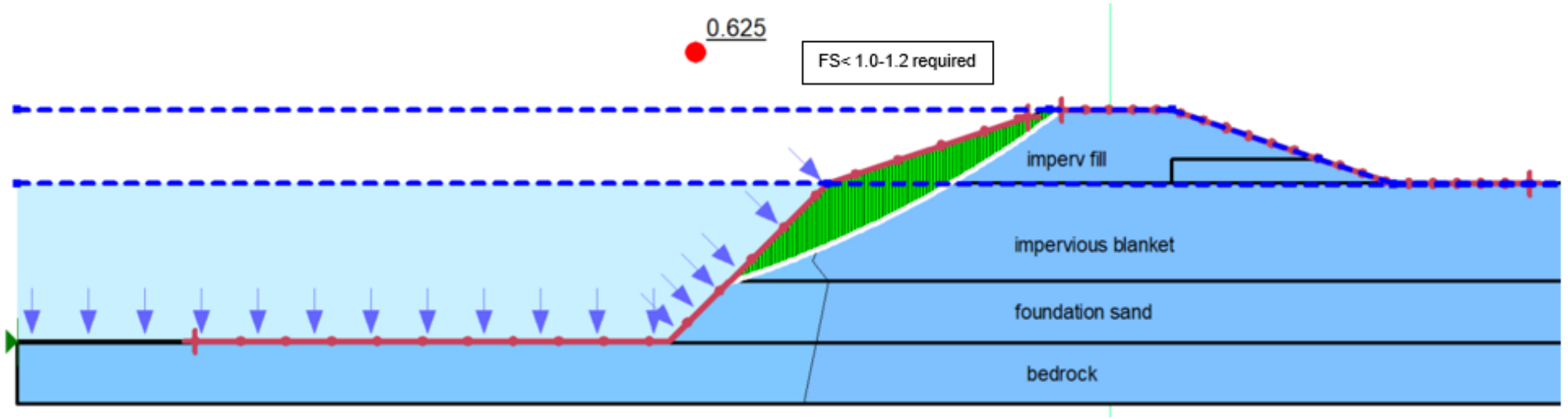


Figure B.2-10: Sudden Drawdown with 1V:1H Channel – SLOPE/W Results

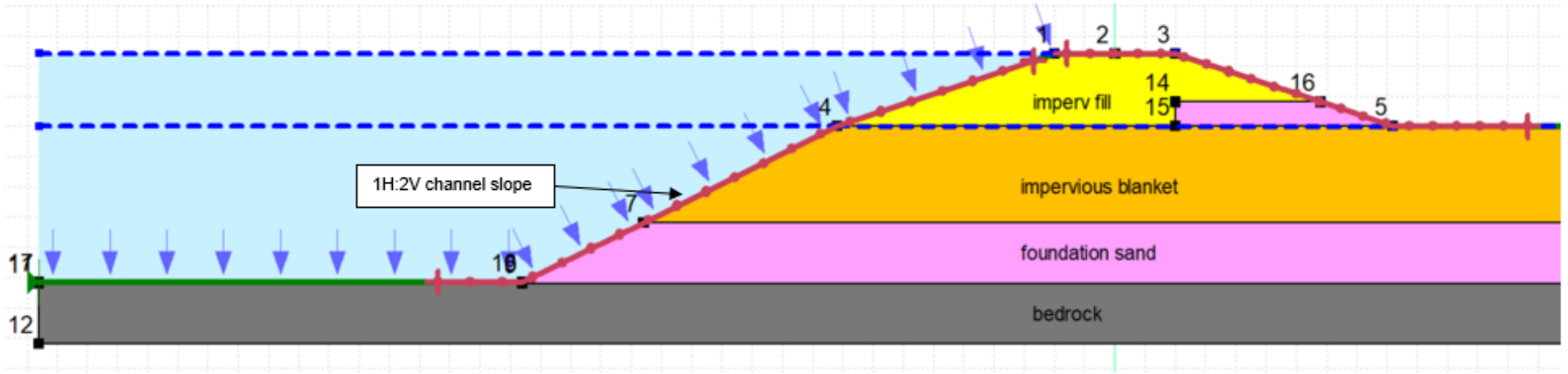


Figure B.2-11: Sudden Drawdown with 1V:2H Channel – SLOPE/W Definition

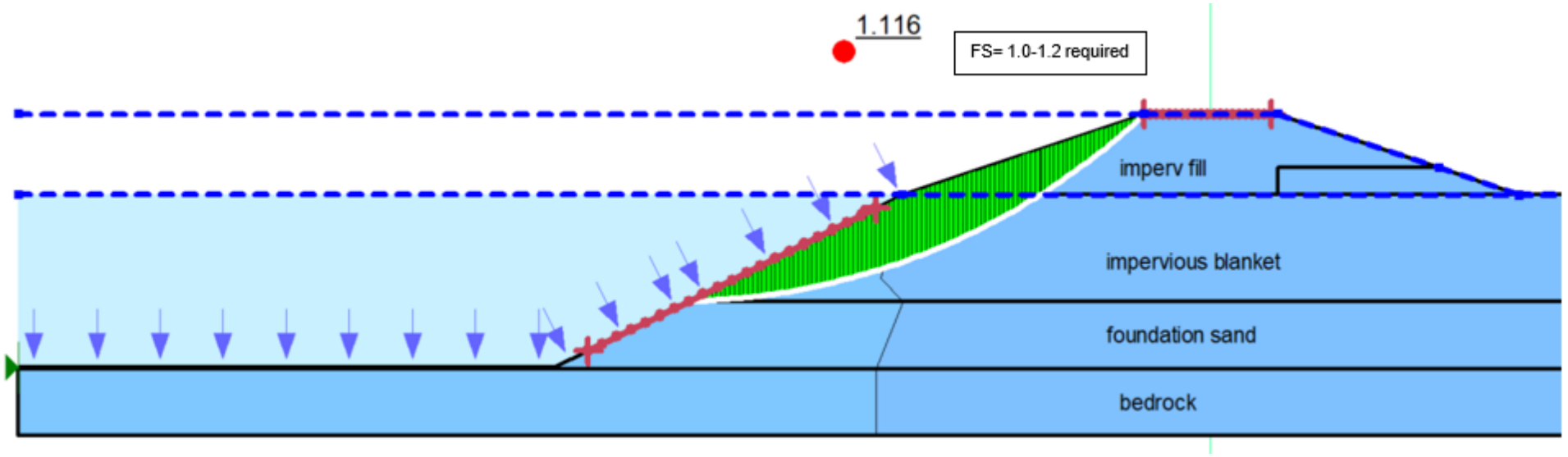


Figure B.2-12: Sudden Drawdown with 1V:2H Channel – SLOPE/W Results

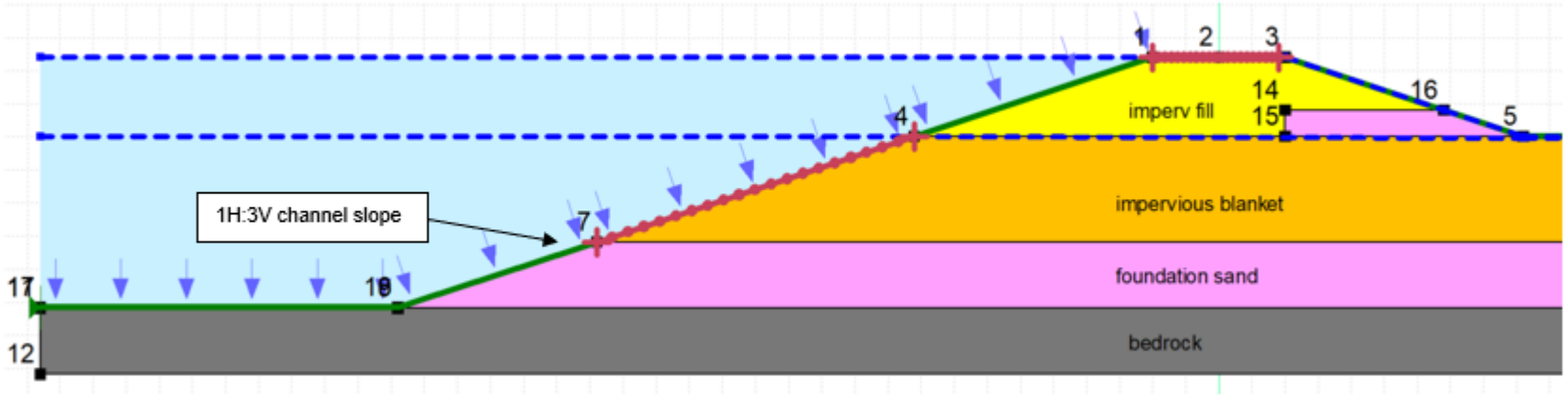


Figure B.2-13: Sudden Drawdown with 1V:3H Channel – SLOPE/W Definition

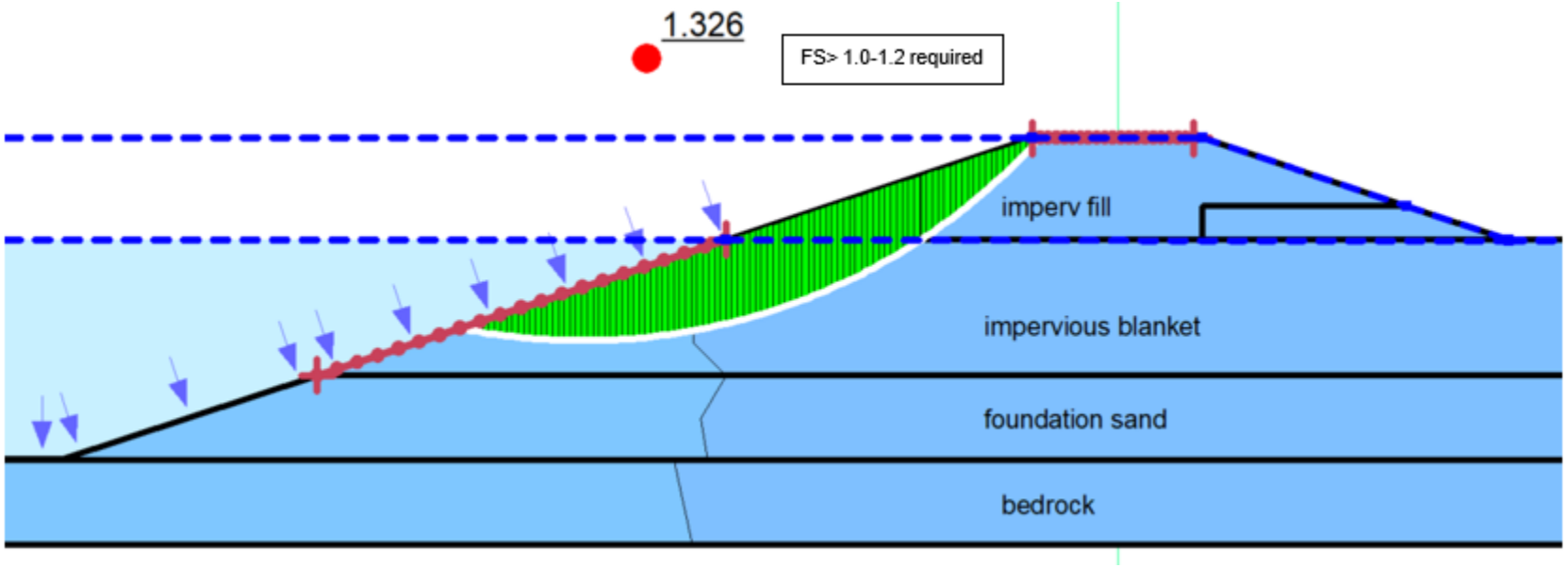


Figure B.2-14: Sudden Drawdown with 1V:3H Channel – SLOPE/W Results

B.3 Settlement Notes

Using assumed levee dimensions, soil profile from the two available soil borings, and soil parameters from Argentine Levee, maximum consolidation settlement of subgrade soils due to levee construction is estimated to be approximately 1.6 inches.

Settlement will be calculated during design using site-specific subsurface investigation and lab testing.

Assumptions and methods used for the initial calculations are given below.

Stress Increase – determined stress increase in clay blanket due to new embankment

H (levee height) = 6 ft max

Soil unit weight = 115 pcf

B1 = ½ crest width = 5 ft

B2 = slope width = 3*6 ft for 1V:3H slope = 18 ft

Consolidation Settlement – determined consolidation settlement of clay blanket due to new embankment

Assumed insignificant settlement within new embankment

C_r (Rebound Index) = 0.045 used Argentine value

C_c (Compression Index) = 0.32 used Argentine value

e_o (Initial Void Ratio) = 1.089 used Argentine value

Depth to Water = 13 ft below existing grade, used creek bed

Determined maximum past pressure based on overburden stress times overconsolidation ratio (OCR) rather than overconsolidation margin or unaltered preconsolidation pressure. This method has been used on several Kansas City District levee projects, including Dodson and Argentine.

OCR (overconsolidation ratio) – used chart developed from KC Levees testing (see Figure B.3-1)

This chart considered consolidation test data from Argentine as well as other Kansas River projects with similar soils, including Armourdale, CID KS, and Manhattan (see test data in Figure B.3-2). Adjusted depth of sample relative to landside grade (since many

borings went through the levee embankment) and modified overburden pressure relative to landside grade (zero at landside grade), adjusting the OCR. Omitted samples from the compacted embankment. Developed an OCR profile for OCR vs. Depth below LS Grade.

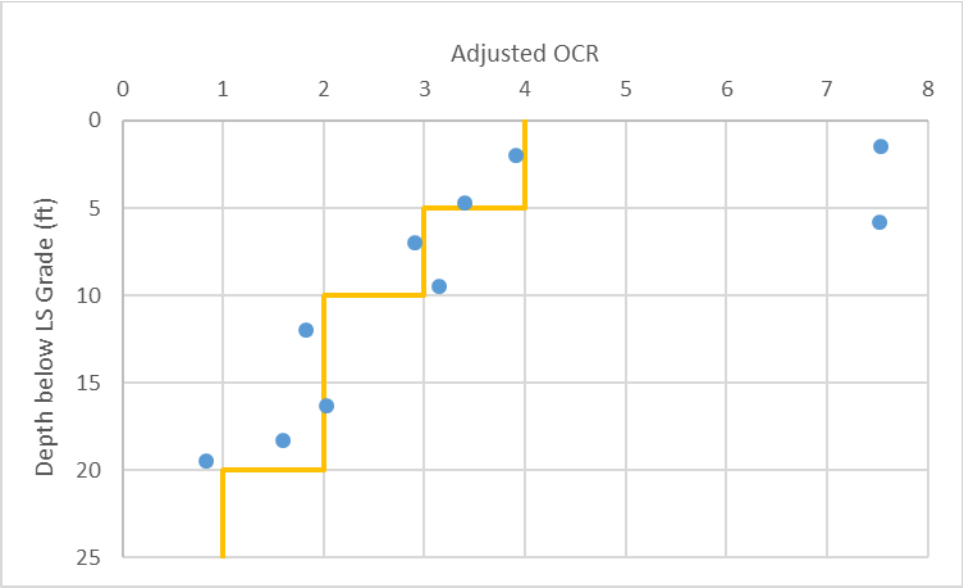


Figure B.3-1: OCR Profile

Used this OCR profile to develop maximum past pressures at various depths in consolidation calculations, multiplying the overburden pressure at that depth by the OCR for that depth.

Levee	Boring	Levee Station	Sample	Top of Boring EL	On crest?	Landside Toe EL	Depth from (ft bgs)	Depth to (ft bgs)	Depth from Landside Grade	Depth to	Overburden Pressure (tsf)	Overburden if no levee (tsf)	Preconsol Pressure (tsf)	OCR	OCR if no levee	Compression Index, C_c	Rebound Index, C_r	Initial Void Ratio, e_o	Soil Class
Argentine	ADU-551	181+00	W1	775.1	Y	758	3	5	-14.1	-12.1	0.24	0	2.71	11.4		0.004	0.011	0.385	CL
Argentine	ADU-549	111+00	W1	776.3	Y	762	3	5	-11.3	-9.3	0.24	0	1.81	7.6		0.04	0.005	0.495	SM
ARMOURDALE	ADU-659	91+20	U3	772.5	Y	748	18	20	-6.5	-4.5	1.1	0	2.9	2.6		0.08	0.008	0.488	CL
CID KS	ADU-502	52+00	U3	763	Y	746	13	15	-4.0	-2.0	0.90	0.00	2.51	2.8		0.33	0.063	1.069	
CID KS	ADU-501	48+00	U3	763	Y	744	18	20	-1.0	1.0	1.20	0.00	1.52	1.3		0.14	0.02	0.768	
ARMOURDALE	ADU-695	253+25	U3	764.5	Y	746	18	20	-0.5	1.5	1.1	0	3.19	2.9		0.2	0.028	0.788	CL
ARMOURDALE	ADU-706	296+50	U2	752.5	Y	744	8	10	-0.5	1.5	0.5	0	1.54	3.1		0.29	0.028	1.011	CH
ARMOURDALE	ADU-646	3+50UE	U2	774.5	Y	757	18	20	0.5	2.5	1.15	0.08625	0.65	0.6	7.536232	0.25	0.035	1.181	CL
Manhattan	ADU-175	200+00	W2	1021.0	assume y	1007.0	15	17	1.0	3.0	0.38	0.115	0.45	1.2	3.913043	0.18	0.012	1.017	SM
Argentine	ADU-549	111+00	W2	776.3	Y	762	18	20	3.7	5.7	1.14	0.27025	0.92	0.8	3.404255	0.32	0.045	1.089	CL
Argentine	ADU-545	28+30	W1	779.2	Y	772	12	14	4.8	6.8	0.78	0.3335	2.51	3.2	7.526237	0.08	0.015	0.524	CL
Manhattan	ADU-176	220+00	W1	1018.0	assume y	1009.0	15	17	6.0	8.0	0.38	0.4025	1.17	3.1	2.906832	0.2	0.014	0.845	CL-ML
CID KS	AD-519	127+00	W4	759.5	Y	748	20	22	8.5	10.5	1.32	0.54625	1.72	1.3	3.148741	0.07	0.01	0.797	
Manhattan	ADU-180	269+30	W1	1019.0	assume y	1006.0	24	26	11.0	13.0	0.59	0.69	1.26	2.1	1.826087	0.11	0.011	0.738	CL
CID KS	ADU-522	138+00	U6	759.7	Y	745	30	32	15.3	17.3	1.85	0.93725	1.9	1	2.027207	0.04	0.007	0.762	
CID KS	ADU-524	132+00	U6	761.2	Y	751	27.5	29.5	17.3	19.3	1.74	1.05225	1.68	1	1.596579	0.11	0.016	0.895	CL
CID KS	ADU-490	23+00	W6	760.5	Y	750	29	31	18.5	20.5	1.80	1.12125	0.94	0.5	0.83835	0.09	0.01	0.83	

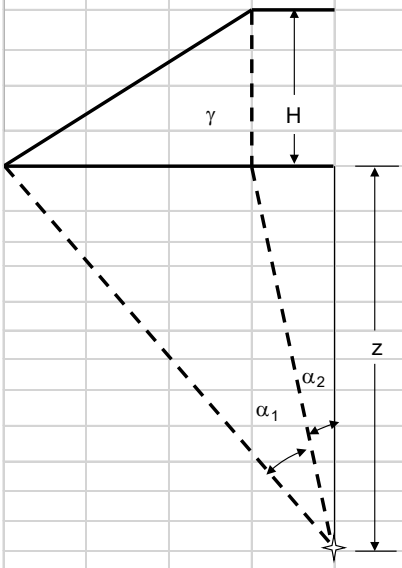
Figure B.3-2: Consolidation Test Data Used for OCR Profile

Find stress due to new embankment

STRESS INCREASE UNDER AN EMBANKMENT - CENTERLINE

Das Foundations, 1999 (pp 233-234)

Project:	Upper Turkey Creek	
Designer :	DHJ	11/23/2022
Checker :		



$\Delta p = (q_o/\pi\,[((B_1 + B_2)/B_2)(\alpha_1 + \alpha_2) - (B_1/B_2)(\alpha_2)])$ (eq. 4.20)

$\alpha 1 = \tan^{-1}((B_1 + B_2)/z) - \tan^{-1}(B_1/z)$ radians (eq. 4.21)

$\alpha 2 = \tan^{-1}(B_1/z)$ radians (eq. 4.22)

H (feet)	γ (pcf)	q_o (psf)	z (feet)	B_1 (feet)	B_2 (feet)
6	115	690	13	5	18

α_1 = 0.68917 radians

α_2 = 0.36717 radians

Δp =	274.055	psf	CL due to One Side
Δp =	548.11	psf	CL due to Both Sides

Left Side

H (feet)	γ (pcf)	q_o (psf)	z (feet)	B_1 (feet)	B_2 (feet)	α_1 (rad)	α_2 (rad)	Only from one side Δp (psf)	Include Both Δp (psf)
6	115	690	1.625	5	18	0.2437	1.25656	344.38	**688.75**
6	115	690	4.875	5	18	0.56388	0.79806	333.53	**667.06**
6	115	690	8.125	5	18	0.67956	0.55165	311.88	**623.75**
6	115	690	11.375	5	18	0.69737	0.41414	286.67	**573.34**

Figure B.3-3: Stress Increase

Consolidation Settlement	
Project:	Upper Turkey Creek
Location:	Merriam, Kansas
Comments:	Settlement of clay blanket due to levee construction

C_r =	0.045	
C_c =	0.32	
e_o =	1.089	
$\gamma_{moist/sat}$ =	115	pcf

Designer :	DHJ	11/23/2022
Checker :		

Compressible Sublayer Depth below Grade			Depth below LS Grade	Sublayer Thickness	Depth to Water	Effective Overburden	OCR based on depth below LS grade	Max past pressure (overburden * OCR)	Stress Increase	Final Pressure	Rebound Curve $C_r/(1+e)$	Virgin Curve $C_c/(1+e)$	Test For Overconsol (P_c>initial)	Test For OC1 (P_{final}<P_c), rebound only	recompress for OC1	recompress portion of OC2	virgin compress for OC2	virgin compress for NC	Strain			Settlement		
Top	Bottom	Average	Average	H_o	h_w	P_o'		P_c adjusted	ΔP	P_f	$C_{\varepsilon r}$	$C_{\varepsilon c}$			ε_{rz}	ε_{rz}	ε_{cz}	ε_{cz}	Recompress Sum ε_{rz}	Virgin Compress Sum ε_{cz}	Total ε_z	ε_z*Ho	Per Layer	Total
ft	ft	ft	ft	ft	ft	psf		psf	psf	psf					Y/Y	Y/N	Y/N	N/N	ft/ft	ft/ft	ft/ft	feet	inches	inches
0	3.25	1.625	1.625	3.25	13	186.88	4	747.50	688.75	876	0.0215	0.1532	Yes	No	NA	0.0129692	0.0105	NA	0.0130	0.0105	0.0235	0.076	0.916	0.92
3.25	6.5	4.875	4.875	3.25	13	560.63	4	2242.50	667.06	1228	0.0215	0.1532	Yes	Yes	0.007333	NA	NA	NA	0.0073	0.0000	0.0073	0.024	0.286	1.20
6.5	9.75	8.125	8.125	3.25	13	934.38	3	2803.13	623.75	1558	0.0215	0.1532	Yes	Yes	0.004784	NA	NA	NA	0.0048	0.0000	0.0048	0.016	0.187	1.39
9.75	13	11.375	11.375	3.25	13	598.33	2	1196.65	573.34	1172	0.0215	0.1532	Yes	Yes	0.0062872	NA	NA	NA	0.0063	0.0000	0.0063	0.020	0.245	1.63
																					Settlement =		1.63	inches

NC: Normally Consolidated

$$S_c = C_c H_c/(1+e_o) \log ((p_o + \Delta p)/p_o)$$

uses virgin compression curve

OC1: Overconsolidated but not loaded beyond Preconsolidation Pressure

$$S_c = C_s H_c/(1+e_o) \log ((p_o + \Delta p)/p_o)$$

uses recompression curve

OC2: Overconsolidated and loaded beyond Preconsolidation Pressure

$$S_c = C_s H_c/(1+e_o) \log p_c/p_o + C_c H_c/(1+e_o) \log ((p_o + \Delta p)/p_o)$$

uses both curves

Definitions

P_c	Max past pressure based on overburden*OCR
P_o'	Effective Overburden Pressure, γ x z
ΔP	Stress Increase from Fill or Footing Pressure
P_f	Final Pressure at given depth, $P_o' + \Delta p$
ε_{rz}	Recompressive Strain
ε_{cz}	Compressive Strain
C_r	Recompression Index
C_c	Virgin Compression Index
H_o	Sublayer Thickness
e_o	Initial Void Ratio

Preconsolidation Margin (see Coduto, 2001, p. 69)

$P_m' = P_c - P_o'$

P_o' for sample at depth of sample

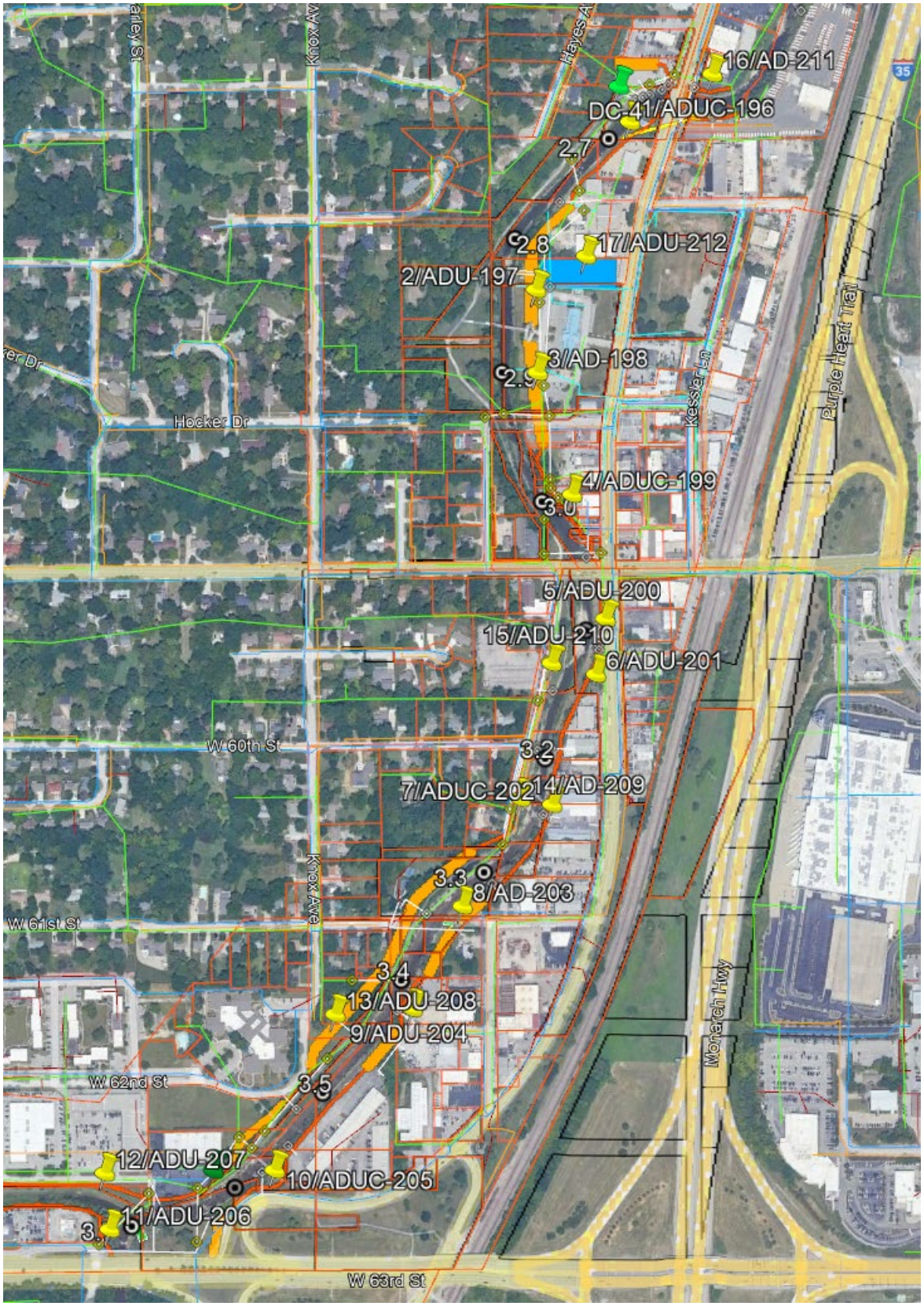
for Normally Consolidated, $P_m' = 0$

Adjusted Preconsolidation Pressure

P_c adjusted = $P_m' + P_o'$ at depth being analyzed

Figure B.3-4: Consolidation Settlement

B.4 Boring Locations



THIS PAGE IS INTENTIONALLY BLANK.

APPENDIX C
Civil

Drainage Structure Existing Vs. Proposed Conditions

	Existing Conditions				Proposed Conditions			Recommended Plan Per FS				Current Recommended Plan						
	Outfall Number	River Sta[1] (mile)	Bank (Left/Right)	Dia (inch)	Dia (inch)	Cross Under	Notes	Riverside Flapgate	Riverside Structures	Landside Structures	Pipes	Updated since FS	Riverside Flapgate	Riverside Structures	Landside Flapgate	Landside Structures	Pipe Demolition	Pipe Construction
REACH 3		3.711	Shawnee Mission Parkway															
	1	3.661	Right	30	NA	NA	No Change - does not cross floodwall or levee											
	2	3.660	Left	24	24	Floodwall		1	Gate Box	1-60" MH	35'-24" RCP	No	1	Gate Box		1-60" MH	remove 35ft existing pipe	35'-24" RCP
	3	3.578	Right	24	24	Floodwall		1	Gate Box	1-60" MH	50'-24" RCP	No	1	Gate Box		1-60" MH	remove 50ft existing pipe	50'-24" RCP
		3.568	Pedestrian Bridge															
	4	3.548	Right	18	24	Floodwall		1	Gate Box	1-60" MH	50'-24" RCP	No	1	Gate Box		1-60" MH	remove 50ft existing pipe	50'-24" RCP
	5	3.520	Left	36	36	Floodwall		1	Gate Box	1-60" MH	50'-36" RCP	**Yes**	1	Gate Box	**1**	**Gate Box**	**remove 56ft existing pipe**	50'-36" RCP
	6	3.465	Right	12	24	NA	Combine with outlet @ 3.422			1-48" MH	50'-24" RCP	No				1-48" MH	remove 50ft existing pipe	50'-24" RCP
	7	3.450	Left	36	36	Levee		1	Gate Box	1-60" MH	50'-36" RCP	**Yes**	1	Gate Box	**1**	**Gate Box**	**remove 56ft existing pipe**	50'-36" RCP
	8	3.441	Right	24	24	NA	Combine with outlet @ 3.422			1-48" MH	100'-24" RCP	No				1-48" MH	remove 25ft existing pipe	100'-24" RCP
	9	3.422	Right	12	24	Floodwall		1	Gate Box	1-60" MH	50'-24" RCP	No	1	Gate Box		1-60" MH	remove 50ft existing pipe	50'-24" RCP
	10	3.355	Left	24	24	NA	Combine with outlet @ 3.350			1-48" MH	30'-24" RCP	No				1-48" MH	remove 25ft existing pipe	30'-24" RCP
	11	3.350	Left	42	42	Levee		1	Gate Box	1-60" MH	50'-42" RCP	**Yes**	1	Gate Box	**1**	**Gate Box**	**remove 56ft existing pipe**	50'-48" RCP
REACH 2	12	3.299	Right	24	24	Floodwall		1	Gate Box	1-48" MH	50'-24" RCP	No	1	Gate Box		1-48" MH	remove 50ft existing pipe	50'-24" RCP
	13	3.228	Right	24	24	Floodwall		1	Gate Box	1-48" MH	50'-24" RCP	No	1	Gate Box		1-48" MH	remove 50ft existing pipe	50'-24" RCP
	14	3.226	Left	24	24	NA	Combine with outlet @ 3.194			1-48" MH	170'-24" RCP	No				1-48" MH	remove 25ft existing pipe	170'-24" RCP
	15	3.195	Right	36	36	Floodwall	Combine with outlet @ 3.190			1-48" MH	30'-36" RCP	No				1-48" MH	remove 25ft existing pipe	30'-36" RCP
	16	3.194	Left	12	24	Floodwall		1	Gate Box	1-48" MH	50'-24" RCP	No	1	Gate Box		1-48" MH	remove 50ft existing pipe	50'-24" RCP
	17	3.190	Right	12	36	Floodwall		1	Gate Box	1-48" MH	50'-36" RCP	**Yes**	1	Gate Box	**1**	**Gate Box**	**remove 56ft existing pipe**	50'-36" RCP
	18	3.150	Left	36	36	Floodwall		1	Gate Box	1-60" MH	50'-36" RCP	**Yes**	1	Gate Box	**1**	**Gate Box**	**remove 56ft existing pipe**	50'-36" RCP
	19	3.112	Right	54	54	Floodwall	Add 54" flap gate at floodwall	1	Concrete Collar		30'-54" RCP	**Yes**	1	Concrete Collar	**1**	**Gate Box**	**remove 56ft existing pipe**	30'-54" RCP
	20	3.109	Right	3-7'x7'	3-7'x7'	Floodwall	Add 3-7'x7' flap gates and headwall	3				**Yes**	3	Headwall	**3**	**Gate Box**	**remove 20ft existing pipe**	**20'-7'x7' RCB**
	21	3.098	Right	12	24	Floodwall		1	Gate Box	1-48" MH	50'-24" RCP	No	1	Gate Box		1-48" MH	remove 50ft existing pipe	50'-24" RCP
	22	3.090	Left	12	12	NA	No Change - does not cross floodwall or levee											
		3.058	Johnson Drive Bridge															
REACH 1	23	3.042	Right	12	24	Floodwall	Add 24" flap gate at floodwall	1		1-60" MH	50'-24" RCP	No	1			1-60" MH	remove 50ft existing pipe	50'-24" RCP
	24	3.039	Left	12		NA	No Change - does not cross floodwall or levee					No						
	25	3.037	Left	12		NA	No Change - does not cross floodwall or levee					No						
	26	3.024	Left	12		NA	No Change - does not cross floodwall or levee					No						
	27	3.020	Left	12		NA	No Change - does not cross floodwall or levee					No						
	28	2.990	Left	12		NA	No Change - does not cross floodwall or levee					No						
	29	2.964	Left	12		NA	No Change - does not cross floodwall or levee					No						
	30	2.953	Left	12		NA	No Change - does not cross floodwall or levee					No						
	31	2.930	Left	72		NA	No Change - does not cross floodwall or levee					No						
	32	2.890	Right	18	18	NA	Combine with Detention Basin outlet @ 2.8			1-48" MH	200'-18" RCP	No				1-48" MH	remove 70ft existing pipe	200'-18" RCP
		2.870	Pedestrian Bridge															
	33	2.854	Right	36	36	NA	Combine with Detention Basin outlet @ 2.8			1-48" MH	125'-36" RCP	No				1-48" MH	remove 70ft existing pipe	125'-36" RCP
	34	2.830	Right	12	12	NA	Combine with Detention Basin outlet @ 2.8			1-48" MH	125'-12" RCP	No				1-48" MH	remove 70ft existing pipe	125'-12" RCP
	45*	2.800	Right	None	60	Floodwall	Detention Basin outlet	1	Gate Box				1	Gate Box	1	**Gate Box**		
	35	2.756	Right	24	24	Floodwall		1		1-60" MH	50'-24" RCP	No	1			1-60" MH	remove 50ft existing pipe	50'-24" RCP
	36	2.668	Left	24	12	Floodwall	Combine with outlet @ 2.650			1-48" MH	60'-12" RCP	No				1-48" MH	remove 25ft existing pipe	60'-12" RCP
	37	2.662	Left	8	12	Floodwall	Combine with outlet @ 2.650			2-48" MH	60'-12" RCP	No				2-48" MH	remove 25ft existing pipe	60'-12" RCP
	38	2.659	Left	8	12	Floodwall	Combine with outlet @ 2.650			1-48" MH	50'-12" RCP	No				1-48" MH	remove 25ft existing pipe	50'-12" RCP
	39	2.655	Left	8	12	Floodwall	Combine with outlet @ 2.650			1-48" MH	30'-12" RCP	No				1-48" MH	remove 25ft existing pipe	30'-12" RCP
	40	2.655	Right	24	24	Floodwall		1	Gate Box	1-48" MH	50'-24" RCP	No	1	Gate Box		1-48" MH	remove 50ft existing pipe	50'-24" RCP
	41	2.650	Left	12	48	Floodwall		1	Gate Box	1-60" MH	60'-48" RCP	**Yes**	1	Gate Box	**1**	**Gate Box**	**remove 66ft existing pipe**	60'-48" RCP
		2.636	Merriam Drive Bridge															
	42	2.632	Right	12	24	Floodwall		1	Gate Box	1-48" MH	25'-24" RCP	No	1	Gate Box		1-48" MH	remove 25ft existing pipe	25'-24" RCP
	43	2.632	Left	12	24	Floodwall		1	Gate Box	1-48" MH	25'-24" RCP	No	1	Gate Box		1-48" MH	remove 25ft existing pipe	25'-24" RCP
	44	2.609	Right	12	24	Floodwall		1	Gate Box	1-48" MH	25'-24" RCP	No	1	Gate Box		1-48" MH	remove 25ft existing pipe	25'-24" RCP

Note 1: Stationing is approximate and based on GIS plotted locations

* Outfall Number 45 is used to indicate the Detention Basin outlet as a proposed condition.

Bold text inidcates updates from 'Recommended Plan Per FS' as shown in 'Current Recommended Plan'

25 : Total riverside flap gates
19 : Total riverside gate boxes

11 : Total landside flap gates
9 : Total landside gate boxes
21 : Total landside 48" Manholes
6 : Total landside 60" Manholes

APPENDIX D
Hazardous, Toxic, and Radioactive Waste

ENVIRONMENTAL LAWS AND REGULATIONS

Various Federal and State legislation regulates the proper use, disposal, and clean up of hazardous materials and waste.

Resource Conservation and Recovery Act (RCRA)

Passed by Congress in 1976, this law gives the Environmental Protection Agency the authority to regulate hazardous waste disposal. This is accomplished by tracking the hazardous waste from its generation to its disposal. In 1984, there were several amendments made to this act, called the Hazardous and Solid Waste Act (HSWA). The amendments were enacted to include underground storage tank regulation, all releases of hazardous waste to the environment, and mandates for corrective action at hazardous waste facilities.

Toxic Substance Control Act (TSCA)

Also enacted by Congress in 1976, this law deals with all chemicals produced or imported into the United States. It requires that the chemical be adequately tested for toxicity prior to its commercial release to the public. In 1979, this act was amended to include the prohibition of the manufacture and distribution polychlorinated biphenyls (PCBs). PCBs are required to be labeled and disposed of properly. In the Statutes of Kansas, K.A.R. 28-55-5 states that PCB treatment, storage and disposal facilities shall comply with all applicable requirements of 40 CFR 761 subpart D, as in effect on November 1, 1986 and 40 CFR 264 subparts B, C, D, G and H, as in effect on November 1, 1986.

Comprehensive Environmental Response, Compensation and Liability Act (CERCLA)

Nicknamed Superfund, this law was enacted to start a trust fund for the payment of remediation at abandoned hazardous waste sites. This law was passed in 1980. The issue of abandoned or uncontrolled sites has grown through the years. The EPA manages the National Priority List, which lists certain CERCLA sites of the highest priority. By law, the State of Kansas also maintains a registry list of all abandoned and uncontrolled hazardous waste sites within the state. Superfund Amendments and Reauthorization Act (SARA) amended CERCLA in 1984. The amendments included more state and community involvement in the remediation process, increased focus on human health problems posed by contaminated sites, selection of remedies that permanently cleaned the site, and consideration of standards and other requirements in State and other Federal laws and regulations.

Federal Facility Compliance Act (FFCA)

In 1992, Congress passed the Federal Facility Act, which requires the Department of Energy (DOE) to develop and submit Site Treatment Plans for the development of capacity and technologies for treating mixed waste. All facilities that the DOE stores or generates these wastes must have such a plan. It also requires schedules for bringing new treatment facilities into operation.

State Regulations

The State of Kansas provides regulations for the use, management, and disposal of hazardous waste. These regulations are found in Article 31 on Hazardous Waste Management with additional statutes on Public Health in Chapter 65 and Hazardous Waste in Regulation Article 34. Environmental Use Controls are mentioned in K.S.A 65-1, 221 to 65-1, 235 and gives the state authority to assist existing programs to help address environmental contamination in a cost-effective manner and restrict the use of land where controls exist. It also empowers the State to enforce the regulations, defines hazardous waste, and controls the management, treatment, and disposal of hazardous waste. Statute Article 34 Solid Waste and Regulation Article 55 establishes criteria for facilities that handle and dispose of PCB waste. Article 28 contains regulations pertaining to the protection of surface waters and the water supply from environmental contamination and establishes water quality standards.

RECORDS SEARCH

A records search for potential contaminated sites along the reach of Turkey Creek addressed by this DDR, was performed in 2009, and updated in 2015 and again in 2023. The records search consisted reviewing various Federal and State of Kansas databased for past and present environmental activities and enforcement actions at properties adjacent to the creek. The information sources reviewed as part of this records search included:

- USEPA Enviromapper - GIS database identifying Superfund sites, sites were hazardous releases have occurred, and facilities regulated by the USEPA that handle materials designated as hazardous waste.

- USEPA CERCLIS Database - The Comprehensive Environmental Response, Compensation, and Liability Information System (CERCLIS) database contains general information on sites across the nation and US territories including location, status, contaminants, and actions taken. It provides information on sites being assessed under the Superfund Program, hazardous waste sites and potential hazardous waste sites.

- KDHE Identified Sites – Contaminated sites managed by the Bureau of Remediation (except for leaking underground or above ground petroleum storage tanks) are included in the Identified Sites List (ISL).

- KDHE AST and UST Assessment Database – this database provides information on site where leaks have occurred, or a tank has been closed.

- KDHE Database of Registered USTs

- KDHE Environmental Interest Finder – Kansas consolidated from the Identified Sites List, the AST and UST Assessment Database, and the Database of Registered USTs into an interactive online GIS system. Information in this system was reviewed in February 2023. A link to the KDHE interactive online GIS system is:

  https://maps.kdhe.state.ks.us/keif/

- USEPA Cleanups In My Community (CIMC) Map/Database – EPA and it's state, territory and tribal partners have developed a variety of cleanup programs to assess and, where necessary, clean up these contaminated sites. CIMC is intended to provide information about sites that might need to be cleaned up, are being cleaned up, or have been cleaned, in a particular community or anywhere in the United States. The scope of sites included in the CIMC GIS database are: Brownfields Properties, Brownsfields Grants Jurisdictions, RCRA Corrective Action, Superfund, Federal Facilities and Federal Agency Hazardous Waste Compliance Docket, Removals/Responses, and Recovery Act Funded Cleanups. The link to the CIMC Map/Database is:

  https://ofmpub.epa.gov/apex/cimc/f?p=cimc:MAP:0:::71:P71_WELSEARCH:AL|State|AL|||true|true|true|true|true|true||-1|sites|N|basic

FINDINGS

Below are the findings from the records search:

Superfund Sites

There were no Superfund sites identified in areas of potential soil disturbance.

Sites identified on CERCLIS Database

The CERCLIS Database identified no sites being investigated or remediated for hazardous waste within the areas of potential soil disturbance.

KDHE Identified Sites

There were no identified sites in the proximity of potential soil disturbance.

Solid Waste Facilities

There are no known solid waste facilities in the project area.

Registered and Closed Underground Storage Tanks (USTs)

There are six properties along the project reach that formerly or currently have USTs. These are described below, working from downstream to upstream.

- 5568 Merriam Dr, Merriam, KS
  State of Kansas records identify this location as either previously or currently having a tank. No other information is provided on the tank size or contents other than it is out of use. The proposed levee is approximately 75-ft from the building on the property and the location of the tank is unknown. If the tank on this property did have a past leak, there is the potential petroleum contamination may be encountered when excavating the inspection trench. It is recommend a boring be advanced in the location of the proposed levee to determine contaminated soil will need to be addressed during construction. See Appendix D, Figure D-1 for location.

- 5639 Merriam Drive, Merriam, KS
  This property contained one-300 gallon waste oil UST, a 10,000 gallon gasoline UST, and a 2,000 gallon diesel UST. This USTs and the associated distribution piping were removed. During removal, 20 CY of contaminated soil was removed near the 300 gallon waste oil tank and 150 CY of contaminated soil was removed near the gasoline and diesel tanks. This property also has two existing 10,000 gallon USTs that are in use. The presumed location of the existing tank is shown in Appendix D, Figure D-1. They were installed in 2019 and have all the latest protective measures to detect and contain any leakage. The potential of encountering petroleum contamination during construction of the proposed floodwall is believed to be low since contaminated soil was removed during closure of the older tanks. Due to the location of the existing tanks, it does not appear construction of the floodwall will be impacted.

- 5730 Merriam Drive, Merriam KS
  Three 560 gallon gasoline USTs were removed from this property. The KDHE records indicate no contaminated was encountered. (see Appendix D, Figure D-2 for location). The building associated with this property was demolished to develop the Farmers Market.

- 5745 Merriam Dr, Merriam, KS
  Kansas records indicate there was a UST on this property but status except to indicate it is out of use. The location, size, and former contents of the tank are also unknown. The property is more than 300 feet from the proposed levee, so it is unlikely there will be any impact to construction of the levee.

- 5903 Merriam Dr, Merriam, KS
  There are two 10,000 gallon gasoline USTs and one 8,000 gallon diesel UST being actively used on this property (see Appendix D, Figure D-3). This is no indication of past leaks from these tanks and they have spill basins, so it is unlikely contaminated soil will be encountered during installation of the proposed floodwall and gas main.

- 6032 Merriam Dr, Merriam, KS
  There was a 500 gallon waste oil UST removed from this property in 1990. Kansas records indicate the tank did not leak and there was not contaminated soil. Therefore, there is not expected to be any contamination encountered during installation flood wall near this property.

Spills

There were a number of spills report along the reach of Turkey addressed by this DDR. Most were associated with minor leaks of mineral oil from transformers and contamination associated with the spill was cleaned up. The only significant spill reported is described below:

- 61st and Merriam Ln, Merriam, KS
  On April 11th, 2022, and vehicle accident knocked down three utility poles and impacted four transformers. It was estimated that 91 pints of mineral oil from the leaked from the transformers and 95% of the oil went down a storm drain and into Turkey Creek. The oil did not contain Polychlorinated Biphenyls (PCBs), so it is not anticipated this was negatively impact construction of the proposed levee in this area. See Appendix D, Figure D-5 for location.

Other Areas of Concern

There were two areas of concern identified during investigations performed in February of 2023. A proposed levee crosses the west side of this property and it is also the location of a proposed retention basin. These are described below:

- 5726 Merriam Dr, Merriam, KS
  In February 2023 railroad ties and debris were encountered while advancing soil boring ADU-212 (see Appendix D, Figure D-2 for location). During advancing of a soil boring in February 2023, railroad ties and debris was encountered. The debris was described as having very poor strength properties as well has having a foul odor. The low strength properties may impact the foundation of the proposed levee and due to the foul odor would not be suitable for the base of the retention basin. All debris removed as part of the levee inspection trench and excavation of the retention basin will need to be disposed of in a permitted landfill. It is recommended additional investigation be performed to determine the nature and extent of the debris to assess the impact to the project.

- 5920 Merriam Dr, Marriam, KS
  In February 2023, petroleum contamination was encountered at a depth of 10 to 11 feet while advancing Boring ADU-201. The source of this contamination is unknown, however a historical aerial photo from 1941 shows a turn in and exit to the site with a small structure. It is speculated this may have been an former gas station but no records could be found. The current use of the property is auto repair. A soil sample was collected and tested for Total Petroleum Hydrocarbon (TPH) and Volatile Organics (VOCs). The results are in Table 1. The only range exceeding the KDHE Tier 2 RSK cleanup level of 250 mg/kg was the Mid Range Hydrocarbons at 316 mg/kg. However, this result is below the Non-Residential Scenario of 350 mg/kg. The VOC analysis was non-detect, which could suggest this is an old spill and the underlying constituents that drive risk associated with TPH have degraded. Since this is a non-residential property, the contaminated soil would likely not need to be cleaned up. However, it would be good practice to segregate contaminated soil during excavation and dispose of it in a permitted landfill. Most petroleum contaminated soil is classified as a Special Waste and typically can be disposed of in a RCRA Subtitle D landfill, which is the same type of landfill that accepts household trash. Additional testing would be necessary to meet disposal facility requirements.

Facilities Handling Hazardous Waste

There are a number of facilities located along the Turkey Creek channel that handle or produce hazardous wastes. These facilities are regulated by USEPA. Although they handle hazardous wastes, there have been no reported spills or areas requiring cleanup that would have an impact on the project.

CONCLUSION/RECOMMENDATIONS

Based upon the findings from the records search performed, there are no known Superfund sites that will impact the proposed work. In addition, no solid waste facilities, such as former landfills, were identified at locations that would impact the project. There were several spills reported near Turkey Creek along the reach addressed by this DDR, but they were quickly remediated and should not have any significant impact to the project. There are six properties with current or former USTs, with the current status of the tanks unknown on two of those properties, 5560 Merriam Dr and 5745 Merriam Dr. There is no known existing contamination associated with any of the current or former tank locations, although some contamination was encountered and remediated when the tanks that were closed and removed.

Two areas of concern were discovered during a February 2023 soils investigations. Buried railroad tires and debris were discovered while advancing a boring on 5726 Merriam Dr. This may present foundation issues for the proposed levee along this reach as well as issues with construction of a retention basin proposed on this property. The other issue discovered during February 2023 soil investigations was petroleum

contamination on 5920 Merriam Dr at a depth of 10 to 11 feet below the ground surface. Sample results indicated TPH levels above residential values for Mid Range Hydrocarbons but below non-residential levels. VOC analysis results were all non-detect, which suggests the source of contamination is very old and the underlying constituents driving the health-based TPH risk levels have degraded over time.

Overall, the risk of encountering significant environmental/HTRW impacts during construction appear relatively low based on Federal and State environmental records. However, there remains uncertainty about activities that occurred prior to Federal/State regulations being enacted and records kept. An example of this is the debris encountered at 5726 Merriam Dr and the petroleum contamination at 5920 Merriam Dr, both discovered during subsurface investigations performed in February 2023. Neither of these sites were expected to have contamination issues based on environmental records. It is not possible nor cost effective to identify all potential environmental subsurface issues along a greater than one mile stretch of the channel addressed in this DDR. The density of borings and other investigative techniques would be both schedule and cost prohibitive, due to the potentially isolated nature of contamination. Therefore, it is recommended that the Contractor be required to develop contingency plans to address unexpected environmental conditions and contingency funding be set aside so unforeseen conditions can be addressed without significant project delays.

As for the two recently discovered sites with environmental concerns, 5726 Merriam Dr. and 5920 Merriam Dr, it is recommended additional investigations be performed to more fully characterize the nature and extent of the environmental issues on these properties. This will help to determine the engineering measures that should be included in the design and help refine cost impacts to the project. For the debris at 5726 Merriam Dr, it is likely all debris below the proposed features will need to be removed and disposed at a permitted facility. For the petroleum contamination encountered at 5920 Merriam Dr, based on the TPH levels, it does not appear any remedial activities would need to occur. However, it is good practice to segregate any contaminated soil from clean soil excavated and dispose of it at a permitted landfill rather than reusing the soil on site or at other project locations. In this case it would likely be classified as a Special Waste that could be disposed of in a RCRA Subtitle D landfill, which is the type of landfill that accepts typical household waste.

There were no sites of HTRW concern identified south of 61st St within the limits of proposed DDR activities.

Table 1
Summary of Soil Analytical Data
Turkey Creek
Merriam, Kansas
Terracon Project No. 02221476

Sample ID			ADU-201A	KDHE TIER 2 RSK Soil Pathway Residential Scenario	KDHE TIER 2 RSK Soil Pathway Non-Residential Scenario	KDHE TIER 2 RSK Soil to Groundwater Pathway Non-Residential Scenario
Collection Depth Below Ground Surface (feet)			10-11			
Collection Date			3/2/2023			
Analyte	**Method**	**Units**				
Total Petroleum Hydrocarbons (TPHs)						
LRH (C5-C8)	8015	mg/kg	**69.5**	**550**	**950**	**150**
MRH (C9-C18)	8015M	mg/kg	**316**	**250**	**350**	**150**
HRH (C19-C35)	8015M	mg/kg	**18.6**	**6,000**	**27,500**	**13000**

Source

Laboratory analysis performed by Pace Analytical, Lenexa, Kansas
Tier 2 Kansas Department of Health and Environment (KDHE) Risk-Based Standards (RSK) Manual - 6th Version (July 2021)
TPH RSKs are discussed in Policy # BER-041 (September 1, 2015, revised May 24, 2017)

Notes

Soil concentrations in milligrams per kilogram (mg/kg)
KSTPH – Kansas Total Petroleum Hydrocarbons
LRH – Low Range Hydrocarbons (carbon ranges C5 C8)
MRH – Mid Range Hydrocarbons (carbon range C9 C18)
HRH – High Range Hydrocarbons (carbon range C19 C35)
Bold = Detection reported above the laboratory reporting limit
Yellow highlighted concentrations are greater than the applicable KDHE residential RSKs
NA = Not Analyzed
<0.005 = Not detected above the listed laboratory analytical reporting limit

Lab Qualifiers

J = Estimated concentration above the adjusted method detection limit and below the adjusted reporting limit.

USGS Background metals don't really help as residential RSKs are less stringent

United States Geological Survey (USGS) county averages (September 30, 2008)
USGS Website: http://mrdata.usgs.gov/geochem/doc/averages/countydata.htm
*Johnson County background levels calculated by using the USGS average concentration plus two standard deviations

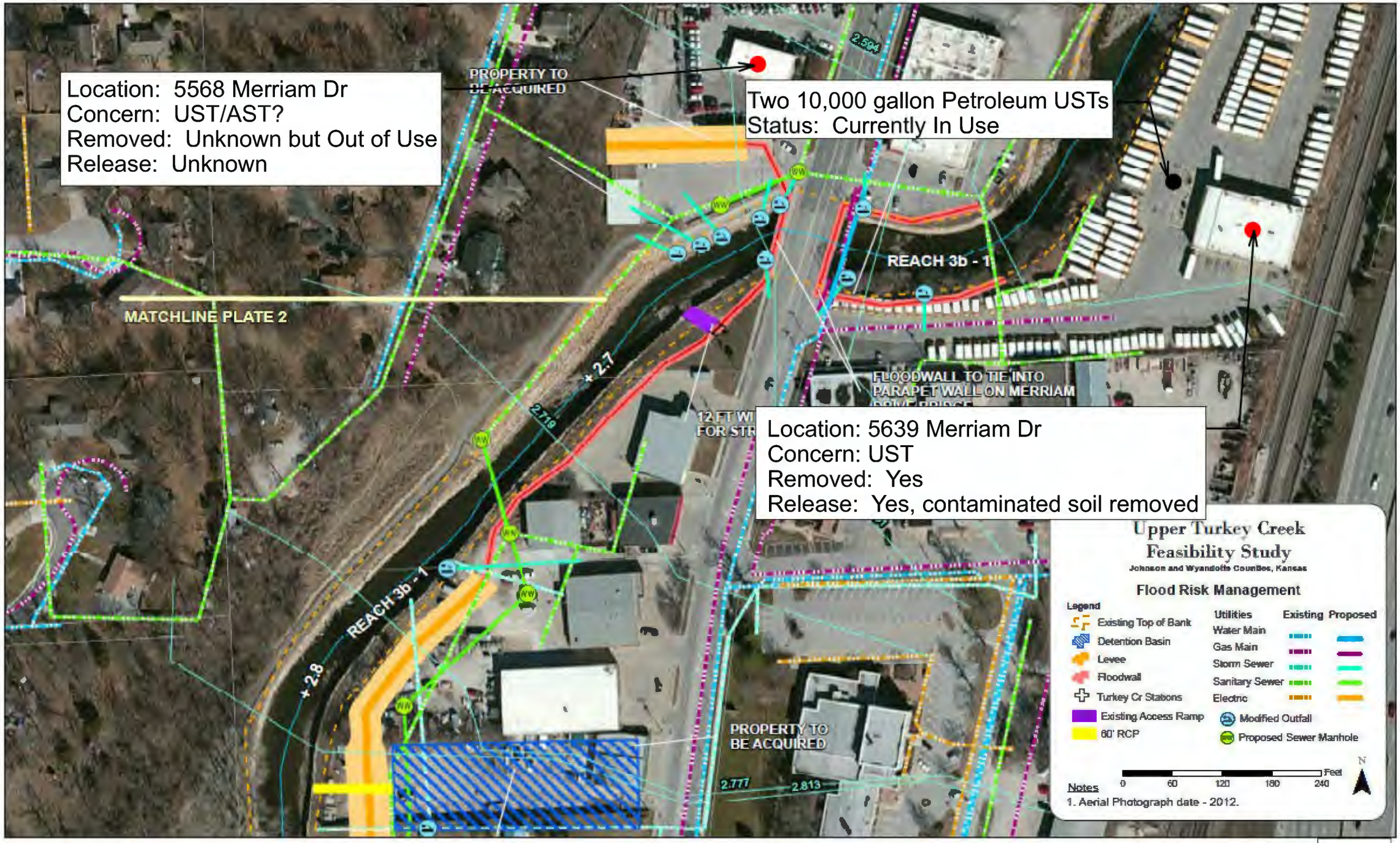

Location: 5568 Merriam Dr
Concern: UST/AST?
Removed: Unknown but Out of Use
Release: Unknown
PROPERTY TO BE ACQUIRED
Two 10,000 gallon Petroleum USTs
Status: Currently In Use
REACH 3b - 1
MATCHLINE PLATE 2
FLOODWALL TO TIE INTO PARAPET WALL ON MERRIAM
Location: 5639 Merriam Dr
Concern: UST
Removed: Yes
Release: Yes, contaminated soil removed
REACH 3b - 1
PROPERTY TO BE ACQUIRED
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure D-1

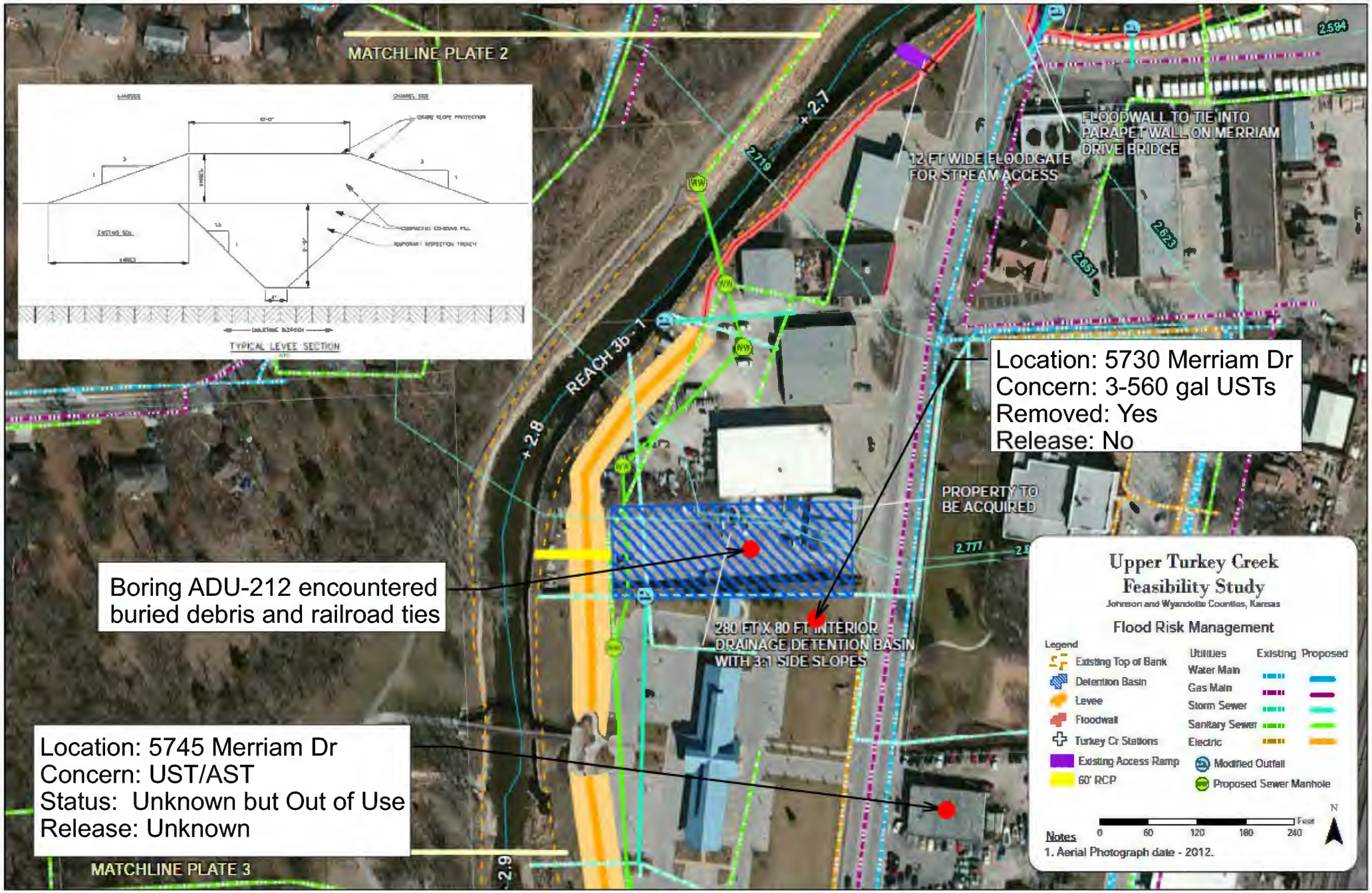

MATCHLINE PLATE 2
TYPICAL LEVEE SECTION
+2.7
2.719
REACH 3b - 1
+2.8
2.594
FLOODWALL TO TIE INTO PARAPET WALL ON MERRIAM DRIVE BRIDGE
12 FT WIDE FLOODGATE FOR STREAM ACCESS
2.623
2.651
Location: 5730 Merriam Dr
Concern: 3-560 gal USTs
Removed: Yes
Release: No
PROPERTY TO BE ACQUIRED
2.777
Boring ADU-212 encountered buried debris and railroad ties
280 FT X 80 FT INTERIOR DRAINAGE DETENTION BASIN WITH 3:1 SIDE SLOPES
Location: 5745 Merriam Dr
Concern: UST/AST
Status: Unknown but Out of Use
Release: Unknown
MATCHLINE PLATE 3
2.9
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60' RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Notes
1. Aerial Photograph date - 2012.
0
60
120
180
240
Feet
N

Figure D-2

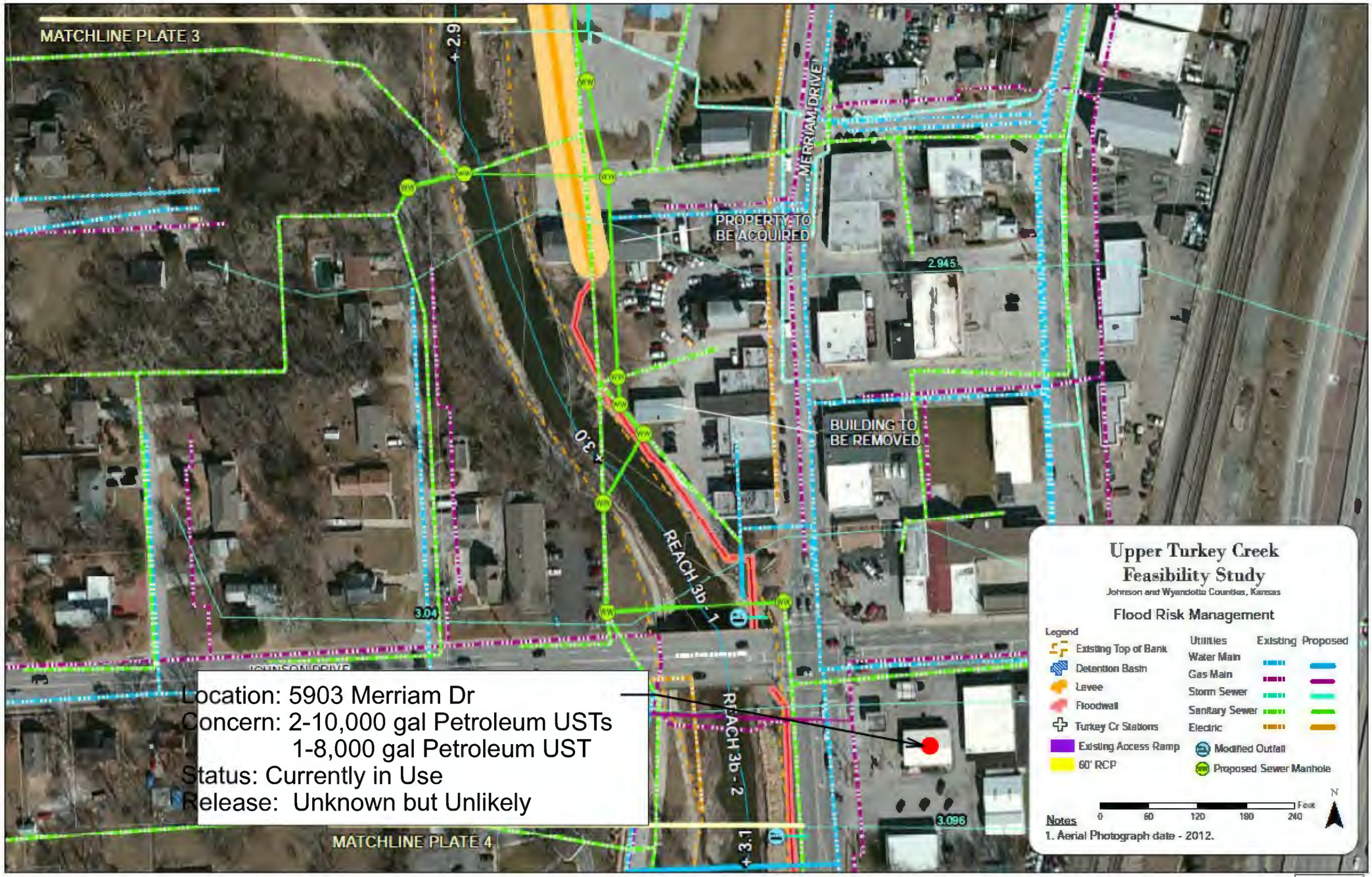

MATCHLINE PLATE 3
+ 2.9
MERRIAM DRIVE
PROPERTY TO BE ACQUIRED
2.945
BUILDING TO BE REMOVED
+ 3.0
REACH 3b - 1
3.04
REACH 3b - 2
3.096
+ 3.1
MATCHLINE PLATE 4
Location: 5903 Merriam Dr
Concern: 2-10,000 gal Petroleum USTs
1-8,000 gal Petroleum UST
Status: Currently in Use
Release: Unknown but Unlikely
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure D-3

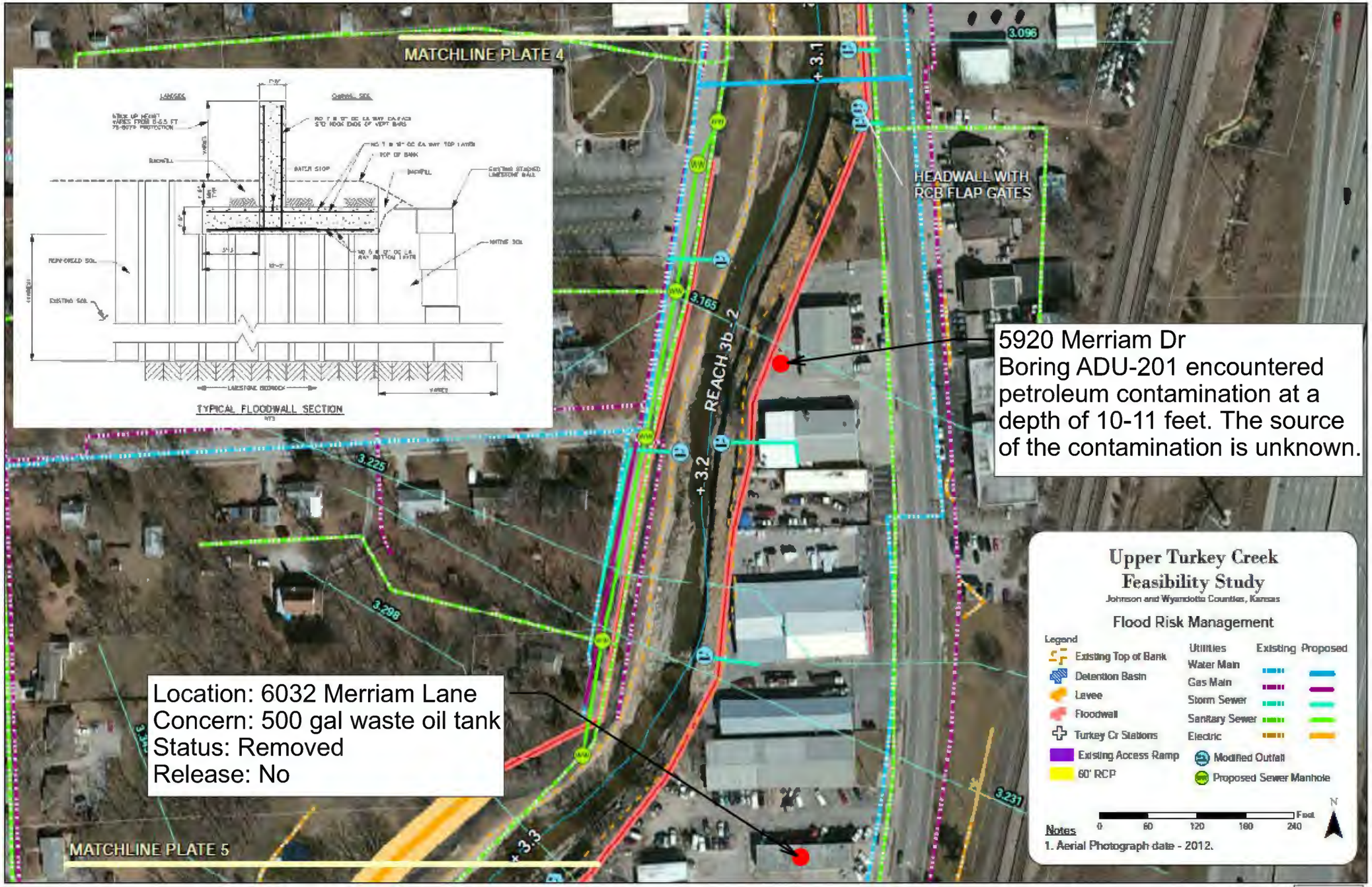

MATCHLINE PLATE 4
TYPICAL FLOODWALL SECTION
HEADWALL WITH RCB FLAP GATES
REACH 3b_2
3.096
3.1
3.165
3.2
3.225
3.298
3.3
3.231
5920 Merriam Dr
Boring ADU-201 encountered petroleum contamination at a depth of 10-11 feet. The source of the contamination is unknown.
Location: 6032 Merriam Lane
Concern: 500 gal waste oil tank
Status: Removed
Release: No
MATCHLINE PLATE 5
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure D-4

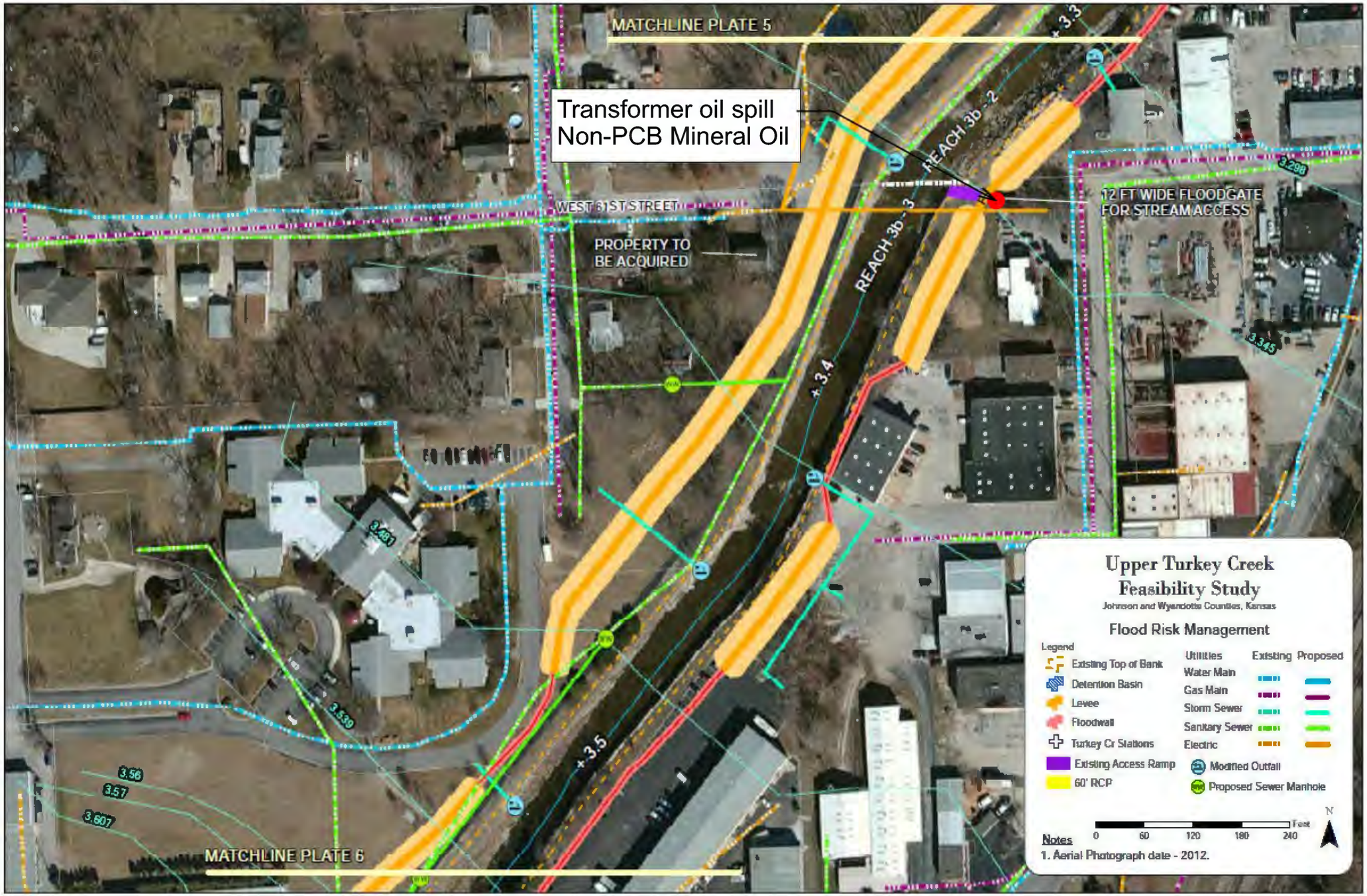
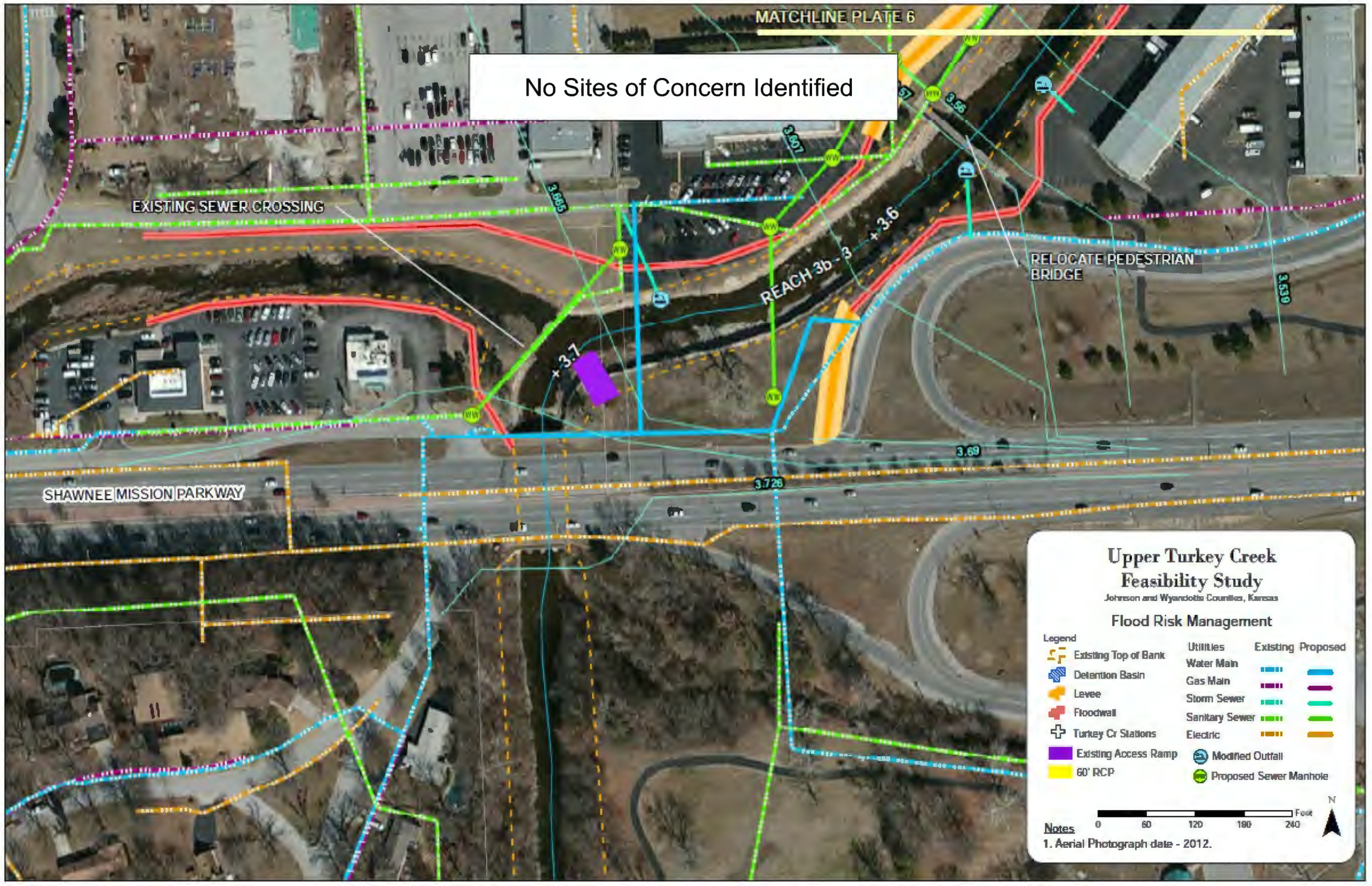

MATCHLINE PLATE 5
Transformer oil spill
Non-PCB Mineral Oil
WEST 81ST STREET
PROPERTY TO
BE ACQUIRED
REACH 3b - 2
REACH 3b - 3
12 FT WIDE FLOODGATE
FOR STREAM ACCESS
+3.3
+3.4
+3.5
3.298
3.345
3.481
3.339
3.56
3.57
3.607
MATCHLINE PLATE 6
Upper Turkey Creek
Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60" RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure D-5



No Sites of Concern Identified
MATCHLINE PLATE 6
EXISTING SEWER CROSSING
REACH 3b - 3
+3.6
+3.7
RELOCATE PEDESTRIAN BRIDGE
SHAWNEE MISSION PARKWAY
3.685
3.607
3.56
3.539
3.69
3.726
Upper Turkey Creek Feasibility Study
Johnson and Wyandotte Counties, Kansas
Flood Risk Management
Legend
Existing Top of Bank
Detention Basin
Levee
Floodwall
Turkey Cr Stations
Existing Access Ramp
60' RCP
Utilities
Existing
Proposed
Water Main
Gas Main
Storm Sewer
Sanitary Sewer
Electric
Modified Outfall
Proposed Sewer Manhole
Feet
0
60
120
180
240
N
Notes
1. Aerial Photograph date - 2012.

Figure D-6

THIS PAGE IS INTENTIONALLY BLANK.

APPENDIX E
Hydraulics and Hydrology

APPENDIX E – HYDROLOGY & HYDRAULICS

E.1 Hydraulic Modeling Configuration & Parameterization
E.2 Feasibility Study Hydrology – Northeast Johnson County Watershed Study
E.3 Validation
E.4 Existing Conditions Comparison

E.1 Hydraulic Model Configuration & Parameterization

HEC-RAS version 6.3.1 was used for the two-dimensional hydraulic analysis. A graphic of the modeled mesh extent, calibration regions, and boundary conditions are presented in Figure 1.

E.1.1 Terrain

A LIDAR terrain surface collected in 2020 with a 2-foot cell size was provided by the sponsor and was used as the base mesh for the project site. A channel survey was performed by USACE personnel in December 2022 and January 2023. This survey was internally processed into station-elevation data and was used to generate a terrain surface with a 2-foot cell size using RASMapper. This survey was merged with the LIDAR terrain to provide a thorough representation of the Upper Turkey Creek channel and floodplain for modeling and mapping purposes corresponding with this project.

E.1.2 Mesh Development

The two-dimensional model consists generally of a 6.25-foot mesh within the stream channel, and a 25-foot mesh along the stream overbanks. A range of mesh sizes varying from approximately 3.125-25 feet were adopted. Smaller mesh sizing was generally adopted in areas of instability or where an increased level of detail is desired. Larger mesh sizing was generally adopted in areas of stability or lower hydraulic complexity or where a lower level of detail is acceptable. Hand-editing of the mesh was performed to aid in stability and to ensure that channel features and changes in channel or floodplain cross-section are properly simulated. Breaklines were placed at the toes of the right and left streambanks throughout Turkey Creek, at the approximate top-of-bank of the right and left streambanks, along the approximate centerline of tributaries, and along elevated roadway and railway embankments.

E.1.3 Bridge Modeling

No bridge drawings or schematics were identified in a literature review of the Feasibility Study documents or project directories to validate that the bridge and culvert cross-sections within the Feasibility Study are consistent with the array of structures that currently cross Turkey Creek within the project reach. Bridge drawings, stream crossing schematics, and/or surveys were obtained for 67th Street, Shawnee Mission Parkway, Johnson Drive, Merriam Drive, the DS Bus Lines bridge, the BNSF Trestle, the I-35 Antioch Road On-Ramp, and I-35 over Turkey Creek. Pedestrian bridges at 67th Street, near Shawnee Mission Parkway, and at the Merriam Marketplace were also incorporated in the modeling based on provided drawings. Each of these structures was transcribed via spreadsheet into the HEC-RAS bridge/culvert editor as a 2D connection. Each stream crossing structure was individually assessed, and modeling routines were adapted according to modeling validation efforts and the computed water surface profiles. Cross sections from the Feasibility Study were transcribed as profile lines to this analysis for comparison of water surface elevations. Bridge roughness values were assigned as 0.035 for the main channel region of each cross-section, and 0.04 for the overbank areas. Weir coefficient values of 2.0 were assigned for each stream crossing structure, as it is not anticipated that with each of these structures having guardrails or barriers that they will be hydraulically efficient in transmitting downstream flows once overtopped.

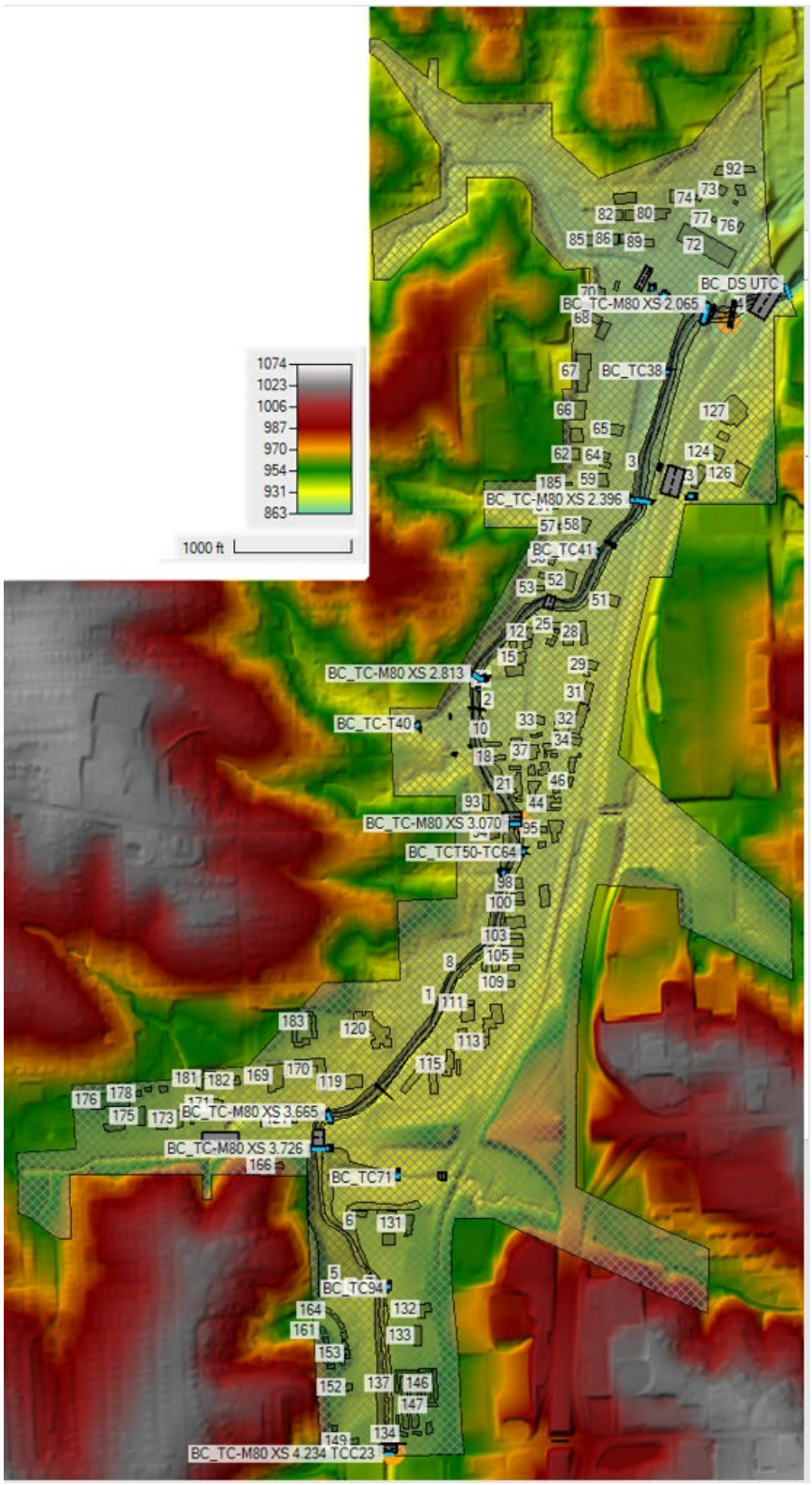


Figure 1. Extent of mesh (hatched area) with roughness calibration regions (gray numbered areas) and boundary conditions (light blue lines) identified. South (lower) end of image is 67th Street, and northeast edge (top right) is I-35.

The low flow methods adopted for each structure were selected by evaluating the presence of bridge piers, and then selecting the appropriate drag coefficient (C_d) based on the pier shapes according to the HEC-RAS Hydraulic Reference Manual. A drag coefficient (C_d) of 1.2 was selected for bridges with circular piers or rounded reinforced concrete box (RCB) inlets, while a C_d of 2.0 was selected for bridge piers or RCB inlets with a square upstream shape. The "Highest Energy Answer" for the low flow method was selected for use.

The high flow methods adopted for each structure were selected by evaluating the bridge deck shape and deck cover type and/or materials. Structures which were anticipated to experience a pressurization during a high flow due to a flat deck surface and a no through-deck ventilation to equalize the below deck pressure with the atmosphere were assigned with the "pressure and/or weir" high flow method. This generally included road bridges. Structures which were not anticipated to experience pressurization during a high flow due to a curved shape and/or due to spaced deck cover materials (e.g. spaced apart deck slats) were assigned with the "energy only (standard step)" high flow method. This generally included railroad trestles and pedestrian bridges. A summary of the bridge structures which cross Turkey Creek and their locations according to the Feasibility Study stationing are presented in Table 1.

A sensitivity analysis was performed for the high flow methods for only the pedestrian bridges and railroad trestle to examine the effect of the change to the water surface profile for the 1% and 0.5% AEP Atlas 14 unsteady flow events. A similar sensitivity analysis was not performed for the road structures due to the high confidence in their overtopping performance resulting in pressure and/or weir flow. This sensitivity analysis for the pedestrian and railroad bridges was performed due to the observed sensitivity of the water surface profile to the presence of bridge cross-sections within the flow conveyance area.

For the 1% AEP flow event, the difference in water surface profiles along the model stream alignment was within -0.04 foot at all pedestrian and railroad bridge locations, with the maximum observed value difference in the vicinity of the structure being -0.05 foot immediately upstream of the Shawnee Mission Parkway Pedestrian Bridge at model station 3.57. These values are within the uncertainty of the water surface profile. Anomalous differences in the water surface profiles at the DS Bus Lines Bridge (River Station 2.479) and Shawnee Mission Parkway Road Bridge (River Station 3.711), which are not pedestrian or railroad bridges, were observed and could not be explained. The difference between the two models was the high flow method at the pedestrian and railroad bridges only. In both cases, the differences are within +/- 0.14 foot which are also considered to be within the uncertainty of the water surface profile. A comparison of the water surface profiles and the computed differences in water surface profiles is presented in Figure 2.

For the 0.5% AEP flow event, the difference in water surface profiles along the model stream alignment was within -0.1 foot at all pedestrian and railroad bridge locations, with the maximum observed value difference in the vicinity of the structure being -0.1 foot in the immediate vicinity of the 67th Street Pedestrian Bridge at model station 4.238. These values are within the uncertainty of the water surface profile. Similar to the 1% sensitivity analysis, anomalous differences in water surface profile were observed at Merriam Drive and at the Shawnee Mission Parkway Road Bridge. In the case of the Shawnee Mission Parkway Road Bridge, the water surface profile immediately downstream of the bridge achieves a maximum difference of -0.29 foot, while immediately upstream of Merriam Drive, the water surface profile for the sensitivity analysis was -0.06 foot below the existing conditions water surface profile. A comparison of the water surface profiles and the computed differences in water surface profiles is presented in Figure 3.

In reviewing the sensitivity analysis profiles overlaid against one another within RASMapper, the differences at the pedestrian and railroad bridges were within the anticipated uncertainty of the

modeling at values of approximately +/- 0.1 foot or less. In the instances of the anomalous values at bridges that were not a part of the sensitivity analysis, the differences in value between the existing conditions and sensitivity analysis simulations are within a reasonable range. These anomalous differences could be due to a variety of factors, including the potential for 3-dimensional effects to flow due to the presence of the bridges, and the potential for debris or other non-aqueous objects within the stream. While debris or other objects were not simulated by the modeling, the sensitivity analysis validated that the adopted assumptions with respect to the high flow methods were reasonable, and that the adopted bridge modeling algorithms produce very similar results within the range of anticipated design profiles for the Upper Turkey Creek flood risk reduction project.

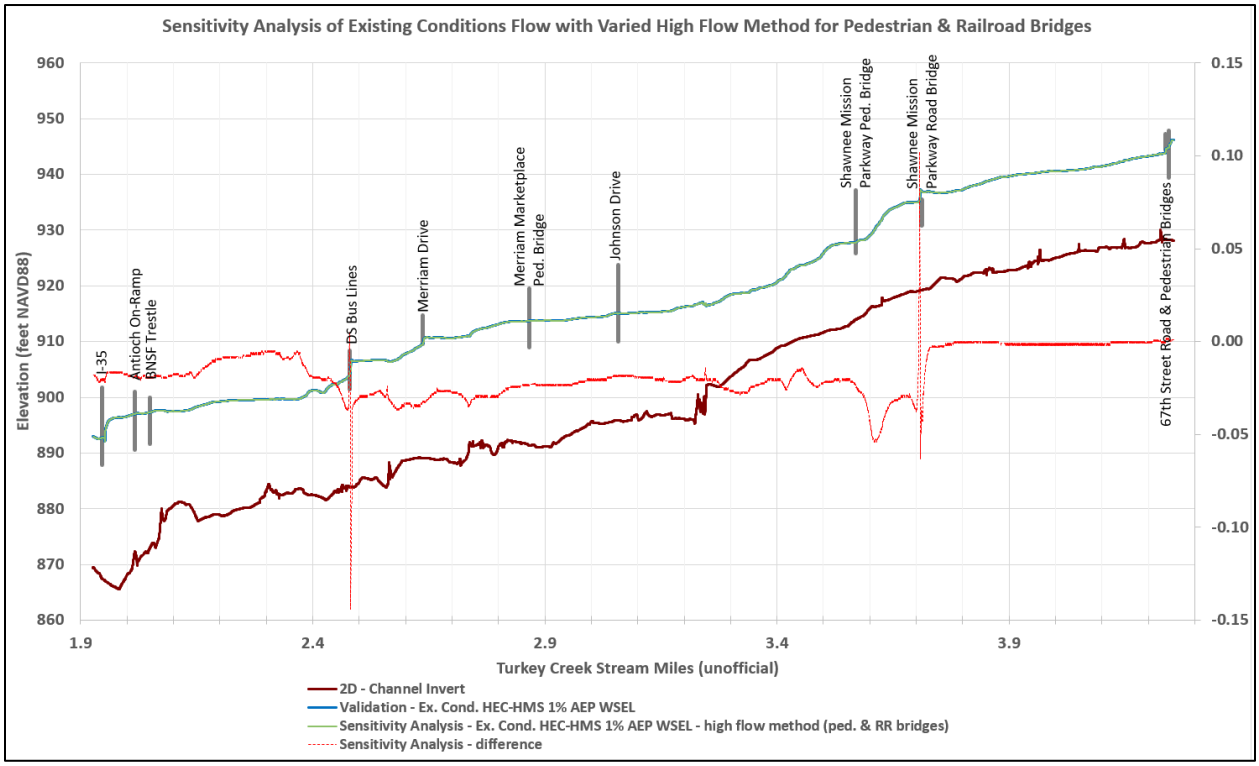


Figure 2. Sensitivity analysis of high flow methods for 1% AEP unsteady flow hydrograph for pedestrian & railroad bridges. Red line indicates difference between the existing conditions and sensitivity analysis water surface profiles, and y-axis difference values are along the right edge of the plot in units of feet. Left y-axis indicates maximum water surface elevation for both profiles. Brown line indicates the channel invert elevation along the stream model alignment, and its elevation is represented along the left y-axis.

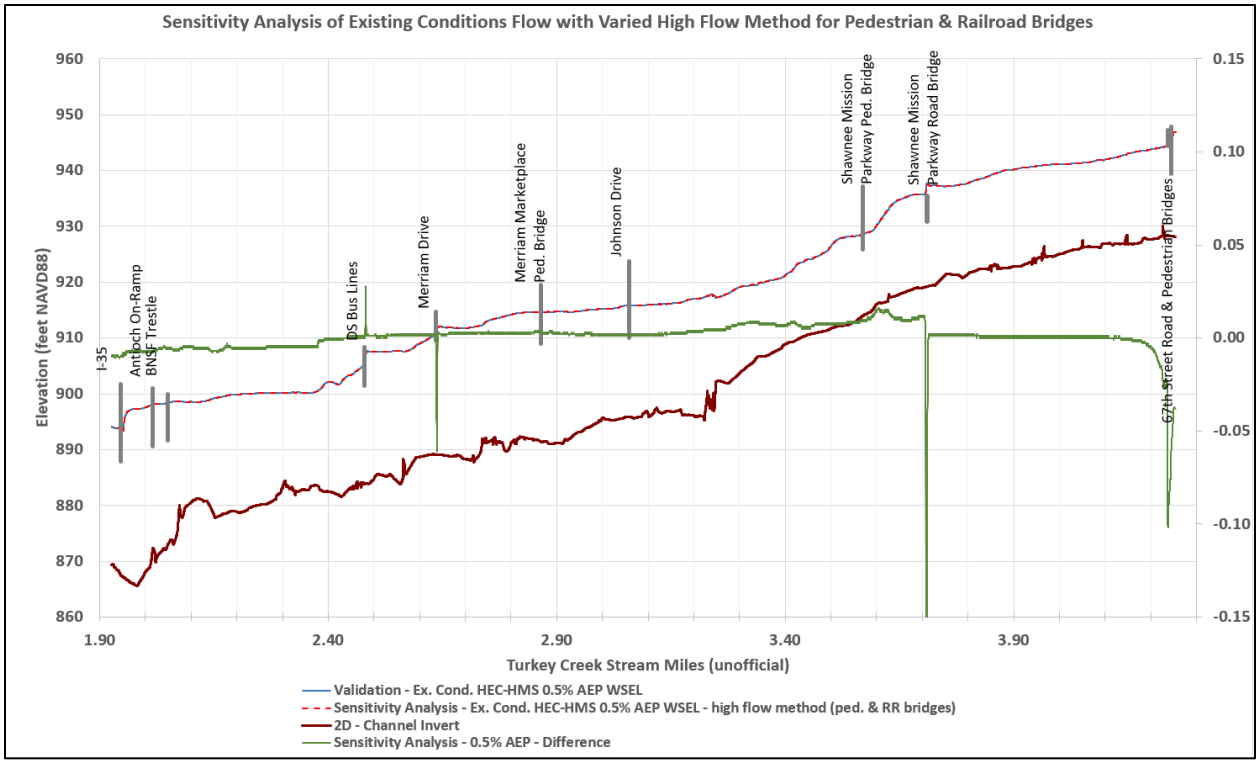


Figure 3. Sensitivity analysis of high flow methods for 0.5% AEP unsteady flow hydrograph for pedestrian & railroad bridges. Green line indicates difference between the existing conditions and sensitivity analysis water surface profiles, and y-axis difference values are along the right edge of the plot in units of feet. Left y-axis indicates maximum water surface elevation for both profiles. Brown line indicates the channel invert elevation along the stream model alignment, and its elevation is represented along the left y-axis.

E.1.4 Manning's Roughness Coefficients

A channel bed Manning roughness value of 0.035 and a streambank roughness value ranging from 0.04-0.045 were implemented as calibration regions based on the Feasibility Study modeling and validation with three 2017 flow events. Building structure footprints within the mesh area were identified and assigned a Manning's value of 0.2 to approximate the flow impedance these features are anticipated to represent during an out-of-bank flow event. This value is consistent with the values recommended for "Developed, High Intensity" description according to the 2D HEC-RAS User's Manual. Overbank roughness values were adopted based on the 2019 National Land Cover Dataset (NLCD), which is a digital raster file generated by the Multi-Resolution Land Characteristics Consortium (MRLCC)[1] that classifies land cover types. This digital raster file identifies land cover types based on specific ranges of light emissions returns from a range of sampling methods. Recommended Manning's roughness ranges for the specific land cover types classified by the NLCD dataset are presented in Table 2-1 of the HEC-RAS 2D User's Manual. The values adopted for this effort are presented in Table 2.

[1] The MRLCC is a group of federal agencies who generate land cover information at the national scale for a wide variety of modeling applications. Their website is https://www.mrlc.gov/.

Table 1. Summary of Bridges and Structures Crossing Turkey Creek.

Bridge Identifier	Feasibility Study River Station	Low Flow Method*	High Flow Method	Notes:
67th Street	4.244	Energy, Momentum (C_d = 1.2)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	
67th Street Pedestrian Bridge	4.238	Energy (no piers)	Energy Only (Standard Step)	Not included in 1D model; walkway has gaps to equalize pressure if submerged
Shawnee Mission Parkway	3.711	Energy, Momentum (C_d = 2.0)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	
Shawnee Mission Parkway Pedestrian Bridge	3.570	Energy (no piers)	Energy Only (Standard Step)	Not included in 1D model; walkway has gaps to equalize pressure if submerged
Channel Grade Control	3.23	N/A	N/A	Included in 2D model as a terrain feature
Johnson Drive	3.058	Energy, Momentum (C_d = 1.2)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	
Merriam Marketplace Pedestrian Bridge	2.866	Energy (no piers)	Energy Only (Standard Step)	Not included in 1D model; walkway has gaps to equalize pressure if submerged
Merriam Drive	2.636	Energy, Momentum (C_d = 2.0)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	
DS Bus Lines	2.479	Energy (no piers)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	Surveyed; no plans available
BNSF Trestle	2.049	Energy, Momentum (C_d = 1.2)	Energy Only (Standard Step)	Bridge deck has gaps to equalize pressure if submerged
I-35 Antioch On-Ramp	2.016	Energy, Momentum (C_d = 1.2)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	
I-35	1.946	Energy, Momentum (C_d = 1.2)	Pres. and/or Weir (Sub. Inlet + Outlet C_d = 0.8)	

* Highest energy answer adopted for all Low Flow Method computations

Table 2. 2016 National Land Cover Dataset (NLCD) values present within the simulated area, description, adopted Manning's roughness, and recommended ranges according to the 2D HEC-RAS User's Manual Table 2-1.

NLCD Value	Land Cover Description	Adopted Manning's roughness	Table 2-1 Recommended Range
11	Open Water	0.035	0.025 – 0.05
21	Developed, Open Space	0.035	0.03 – 0.05
22	Developed, Low Intensity	0.06	0.06 – 0.12
23	Developed, Medium Intensity	0.08	0.08 – 0.16
24	Developed, High Intensity	0.12	0.12 – 0.20
31	Barren Land Rock-Sand-Clay	0.027	0.023 – 0.030
41	Deciduous Forest	0.15	0.10 – 0.20
43	Mixed Forest	0.1	0.08 – 0.20
52	Shrub-Scrub	0.09	0.07 – 0.16
71	Grassland-Herbaceous	0.035	0.025 – 0.05
81	Pasture-Hay	0.035	0.025 – 0.05
82	Cultivated Crops	0.035	0.02 – 0.05
90	Woody Wetlands	0.08	0.045 – 0.15
95	Emergent Herbaceous Wetlands	0.06	0.05 – 0.085

E.1.5 Floodplain and Tributary Incorporation

Tributary stormwater culverts and/or conduits were implemented in select locations based on aerial imagery and approximate elevation and dimension measures were based on the LIDAR dataset and aerial imagery. These features were incorporated into the modeling to ensure that the volume represented by the tributary ditches and channels were attached to the Turkey Creek mainstem as they would be in the field. Surveyed elevations for these features were not collected, as detailed water surface profiles are not desired along the reaches of the respective tributary features, and the performance of the individual culverts and/or conduits is not being evaluated for this effort. During the site visit, no flap gates were identified on conduit and/or culvert features throughout the proposed project reach. As a result, conduits and culverts for the existing conditions simulations did not include any flap gates or closure features restricting backflow.

E.1.6 Boundary Condition Implementation and Sensitivity Analysis

The boundary conditions energy grade line (EG) slope values adopted for the Turkey Creek stream channel within the mesh were based upon the median value for the 50% through 0.5% friction slopes computed at the upstream (4.254) and downstream (1.927) cross-sections of the Feasibility Study 1-dimensional model. The median friction slope value was adopted based on the anticipated use of a varied flow hydrograph and the desire for the value to be representative throughout the simulation. The adopted upstream EG value was 0.004909, while the downstream EG value was 0.002583.

Boundary conditions for the modeling scenarios using the unsteady flow hydrographs were placed according to the cross-section stationing indicated in the Feasibility Study modeling, or near the node locations identified by GIS layers for the HEC-1 analysis. Cross-sections 2.065, 2.396,

2.813, 3.070, 3.665, 3.726, and 4.234 were adopted as boundary condition locations, consistent with the adopted flow change locations in the steady flow hydraulic modeling from the Feasibility Study. Similar to the upstream and downstream boundary condition locations, the median friction slope values for the 50% through 0.5% frequency events from the 1D Feasibility Study HEC-RAS model were adopted at the respective flow change cross-section locations.

The steady flow values corresponding with the 1% and 0.5% annual exceedance probabilities (AEP) indicated in the Feasibility Study model were implemented as constant flows within the Unsteady Flow Data Editor. Only these values were simulated as the design flow profile and levee/floodwall alignment were based on these values, respectively.

Unsteady flow hydrographs generated from the Feasibility Study hydrologic models were also simulated as part of the updated existing conditions analysis. Nodes from the hydrologic model identified as TCC23, TC94, TC-T71, TC-T59, TC66, TC-T50, TC64, TC79, TCT40, TC57, TC48, TC41, TC-T30, TC19, and TC38 were implemented as boundary conditions emptying into Turkey Creek according to their locations on Figure 6-1 of the “Northeast Johnson County Watershed Study” (2005). The EG slopes at the respective hydrologic node locations were based on the slope computed from the terrain approximately 10 to 25 feet upstream to downstream of the respective node location.

A sensitivity analysis of upstream and boundary conditions was performed to validate that the modeling extents are sufficient to not be significantly influenced by minor discrepancies in the adopted boundary condition slopes as compared to what exists for the reach of analysis. The sensitivity analysis consisted of three evaluations using an order of magnitude greater and less than the adopted value, as well as a decreased value within the same order of magnitude. The sensitivity analyses used upstream and downstream Turkey Creek EG slopes of 0.01 (BCTest1), 0.001 (BCTest2), and 0.0001 (BCTest3), and are presented in Figure 4.

The model was not particularly sensitive to variations in the upstream boundary condition EG slope; however, the model was highly sensitive to downstream boundary condition EG slopes less than those adopted for the modeling. The reach of analysis for the proposed levee and floodwall reach design, according to the cross-section reach adopted for the Feasibility Study, was from cross-section 3.665 (Station 3100 in Figure 3) at the upstream end to 2.594 (Station 8760 in Figure 3) at the downstream end.

Based on the sensitivity analyses, all three upstream water surface profiles approximately converge to within an absolute value of 0.1 foot difference from the existing conditions water surface by cross-section 4.05 (approximately Station 920 in Figure 2), which is upstream of the reach of interest at cross-section 3.665. At approximately cross-section 3.165 (Station 5750 in Figure 2), for the boundary condition EG slope an order of magnitude less than that adopted (BCTest3), the water surface profile diverges from the existing conditions profile and other sensitivity analyses by an absolute value greater than 0.1 foot. At approximately hydraulic model reach station 2.86 (Station 7360 in Figure 2), for the boundary condition EG slope of a similar order of magnitude as the existing conditions, but less than the adopted value (BCTest2), the water surface profile diverges from the existing conditions profile by an absolute value greater than 0.1 foot. At approximately hydraulic model reach station 2.478 (Station 9380 in Figure 4), for the boundary condition EG slope an order of magnitude greater than the existing conditions, the water surface profile diverges from the existing conditions profile by an absolute value greater than 0.1 foot.

Given the friction slope values computed by the Feasibility Study modeling, and the relative agreement of the water surface profiles for the sensitivity analyses with the existing conditions water surface within the reach of analysis, it is believed that the modeled reach extent is adequate to reasonably simulate the proposed Upper Turkey Creek flood risk reduction system.

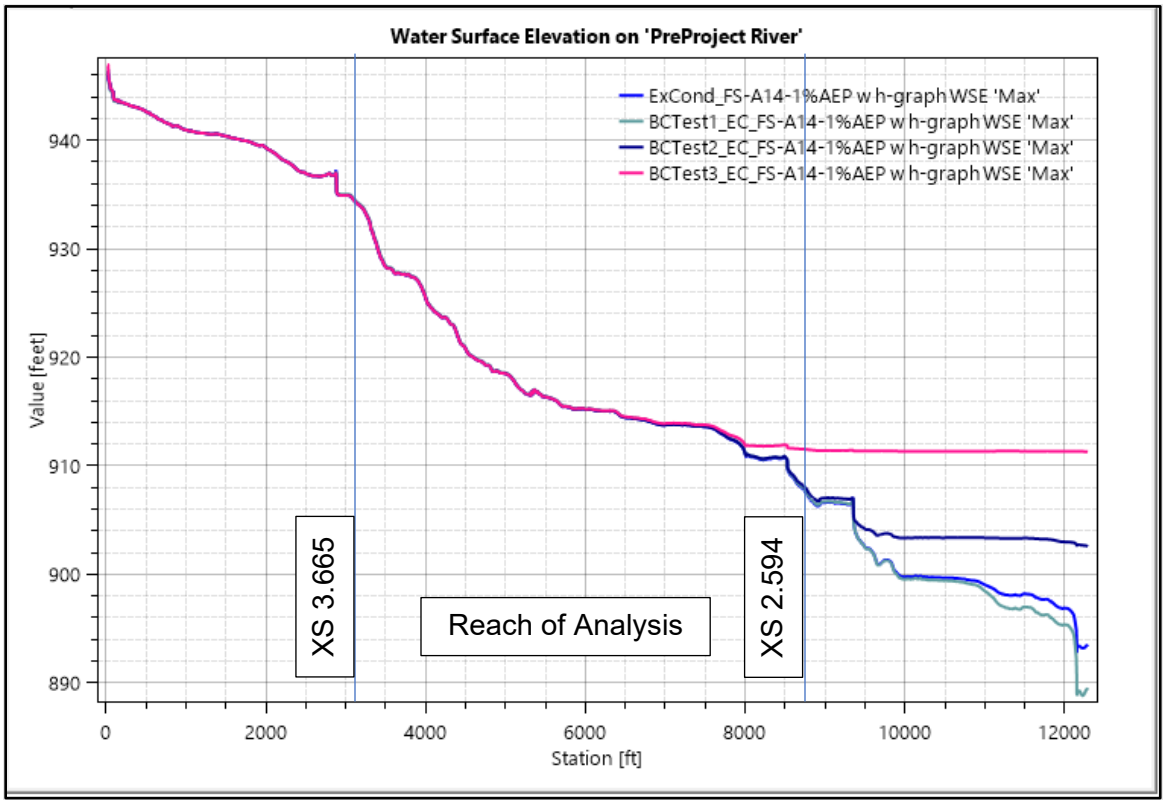


Figure 4. Comparison of boundary condition sensitivity analyses as compared to the existing conditions simulation (ExCond… with upstream EG = 0.002583 and downstream EG = 0.004909) simulations using an upstream and downstream boundary condition 1 order of magnitude greater than adopted (BCTest1 with EG = 0.01); 1 order of magnitude less than adopted (BCTest3 with EG = 0.0001); and within the same order of magnitude, but less than adopted (BCTest2 with EG = 0.001)

E.1.7 Stream Gauging

While no flow gauging is maintained on Turkey Creek, stage gauges are maintained at 67th Street, Johnson Drive, and at the Antioch Road On-Ramp for I-35. The 67th Street gauge is most upstream, while the Antioch On-Ramp gauge is the downstream gauge. Johnson Drive lies approximately halfway in between. These gauges are maintained as part of the Stormwatch gauge network by Johnson County, Kansas. Gauge datums were based upon the surveyed elevations from the current FEMA floodplain mapping effort. A summary of datums and stage adjustments for the purposes of analysis are presented in Table 3, while a map of the gauge locations is in Figure 4.

Table 3. Stormwatch Gauges used for analysis during hydraulic modeling with Stormwatch identifiers, datums and channel bottom stages.

Stormwatch Gauge Identifier	Gauge Location	Gauge Datum (NAVD88)	Channel Bottom Stage (feet)*
TU03	67th Street at Turkey Creek	928.06	2.15
TU04	Johnson Drive at Turkey Creek	895.418	1.65
TU06	Antioch On-Ramp at Turkey Creek	869.46	2.03

* Values are based upon documented measurements at Stormwatch website for the respective gauges.

E.1.8 Two-dimensional Modeling Algorithms and Parameterization

The adopted two-dimensional parameters used for simulation are presented in Table 4. In general, the default values were adopted. Notable exceptions to this include the Equation Set, Turbulence Model, and Matrix Solver parameters.

Due to the location of the project within a highly urbanized area, and some uncertainties with respect to the flow contributions of tributaries to the watershed, the SWE-ELM (original/faster) equation set was selected. The Diffusion Wave method was not anticipated to represent the momentum component of the flow analysis adequately, and the SWE-EM (stricter momentum) was believed to represent too high of a resolution and too lengthy of a computational time given the uncertainties in the precision of roughness values and the inability to reliably calibrate the model to recent flood events. The SWE-LIA (local inertia) equation set is useful for slow rising hydrographs or compound flooding where wind is important and cannot be simulated with the Diffusion Wave Equation. Turkey Creek is a flashy stream, and therefore, SWE-ELM was anticipated to represent the best available equation set option.

The Conservative Turbulence Model was selected to provide greater resolution with respect to the momentum components. Due to numerous features within the floodplain and crossing the channel, such as bridges, curves in the stream channel, in-channel features, and concrete encased pipes, it was anticipated that accounting for the momentum components would be critical to providing a more accurate water surface profile for selecting the appropriate flood risk mitigation measure for a highly developed area.

Longitudinal and Transverse Mixing Coefficient value defaults were adopted. This reach of Turkey Creek generally has gentle meanders and moderate surface irregularities, with some reaches of straight channel and relatively consistent cross-section. The default values are on the boundaries of the respective little- and moderate mixing descriptions in Tables 5.1 and 5.2 of the HEC-RAS 2D User's Manual and seem applicable to this analysis.

The SOR (Iterative) solver was not selected based on unsatisfactory results using the Pardiso solver. The FGMRES-SOR (Iterative) was not evaluated.

Table 4. Summary of adopted 2D simulation parameters for unsteady flow modeling

Parameter	Adopted Value
Theta	1
Theta Warmup	1
Water Surface Tolerance (ft)	0.01
Volume Tolerance (ft)	0.01
Maximum Iterations	20
Equation Set	SWE-ELM (original/faster)
Initial Conditions Time (hrs)	2
Initial Conditions Ramp Up Fraction	0.1
Number of Time Slices	1
Turbulence Model	Conservative
Longitudinal Mixing Coefficient	0.3
Transverse Mixing Coefficient	0.1
Smagorinsky Coefficient	0
Matrix Solver	SOR (Iterative)
Convergence Tolerance	0.00001
Minimum Iterations	3
Maximum Iterations	30
Relaxation Factor	1.3

E.2 Feasibility Study Hydrology – Northeast Johnson County Watershed Study

The Northeast Johnson County Watershed Study was completed in 2005 and provided the hydrology for the Feasibility Study. This analysis was supplemented using the same hydrologic modeling configuration within HEC-1 (and subsequently HEC-HMS) using Atlas 14 annual maximum series 24-hour total precipitation volumes. The hydrologic modeling for the Northeast Johnson County Watershed Study utilized the following parameterizations:

- Precipitation Quantity: KDOT published rainfall depths for 24-hour duration
- Precipitation Distribution: SCS 24-hour, Type II Distribution
- Surface Runoff Volume: SCS Curve Number
- Runoff Hydrograph: SCS Dimensionless Hydrograph
- Flow Routing:
    - Muskingum-Cunge (in streams and enclosed systems where backwater effects are negligible
    - Modified Puls (where significant flood storage is observed; used on selected lakes, detention basins, and upstream of selected bridges and culverts)
- Antecedent Moisture Condition: AMC 2.75 (calculated as AMC II plus 75% of the difference between AMC III and II)
- Subarea Delineation: Outfall locations at primary tributaries, primary crossings (i.e. roads), primary structures (i.e. dams) and the need to satisfy the 160-acre subwatershed drainage area goal for the project

E.3 Validation

A validation effort was undertaken using the 26-27 July 2017, 5-6 August 2017, and 21-22 August 2017 flood events. While Turkey Creek has gauges, none of the tributaries to Turkey Creek are

gauged, so there is uncertainty with respect to the contributions of each of these tributaries as they relate to the stage hydrographs at each site for the validation events. The StormWatch gauges evaluated as part of this validation are presented in Figure 5. In order to assess the gauges for validation, the stage hydrographs for each gauge for each event were overlaid against one another, and are presented in Figure 6, Figure 7, and Figure 8.

An evaluation of watershed geometry and the hydrologic modeling was also undertaken to assess whether the timing of the tributaries was likely to coincide with a peak on Turkey Creek, assuming an identical intensity and duration over the contributing watershed. A comparison of the approximate tributary length to that of the reach upstream of Turkey Creek was evaluated, as well as the flow hydrographs generated by the hydrologic modeling. A graphic of the approximate watershed boundary based upon the downstream end of the modeled reach is presented within the red boundary in Figure 9, while a comparison of a major ungauged tributary to the upstream boundary for the modeled reach is presented in Figure 10.

Based upon a comparison of the approximate watershed areas and the hydrologic response from the hydrologic modeling generated by the "Northeast Johnson County Watershed Study" in 2005 (Watershed Study), it was determined that it would not be possible to assume that the tributaries drained from their respective watersheds prior to the peak of the event on the mainstem of Turkey Creek. While the stage hydrographs from the respective gauges along Turkey Creek (i.e., TU03, TU04, and TU06) approximately coincide with one another in flow response shape and duration, it appears that the tributaries also very closely coincide with drainage response shape and duration relative to the Turkey Creek mainstem. An illustration of this is presented in Figure 11.

Due to the similarity in runoff response of the tributaries with that of mainstem Turkey Creek, the tributary inflow hydrographs could not be differentiated from the mainstem flows in the hydraulic modeling. If a differentiation in tributary versus mainstem hydrographs could have been identified, the identification of the mainstem hydrograph pulse could have been used to justify revising Manning's roughness values in the modeling to match the observed stage hydrographs at Turkey Creek StormWatch gauges at Johnson Drive (TU04) and I-35 (TU06). A rain-on-grid model has not been developed for the entirety of this watershed at this time, so a gridded precipitation simulation has not been performed to estimate the tributary flow contributions to improve the validation and/or calibration of the hydraulic model. Due to the highly developed nature of the entirety of the Turkey Creek watershed, the development of a detailed model would require significant time and cost. This effort is ongoing as part of the FEMA floodplain mapping update, however.

While it is not possible to calibrate the Manning's roughness values to closely approximate the observed stages at the stream gauges, it is possible to validate the results of the modeling with the observed stages. Figure 12, Figure 13, and Figure 14 provide illustrations of the computed water surface profile against the observed peak stage elevations according to the adopted datums. Note that for this analysis, the river centerline adopted for the Feasibility Study was used for the comparison of the computed flow profile to the stage measurements. Due to the inclusion of the turbulence components in the 2D analysis, slight variations in the water surface profile occur laterally within the channel. This variation in water surface elevation was previously separately computed as superelevation in one-dimensional hydraulic analyses according to Equation 2-31 in EM 1110-2-1601 but is now considered within the simulation capabilities of HEC-RAS. A comparison of the computed differences for the three events is presented in Table 5. Note that all simulations used the same mesh, bridge configurations, and roughness parameters.

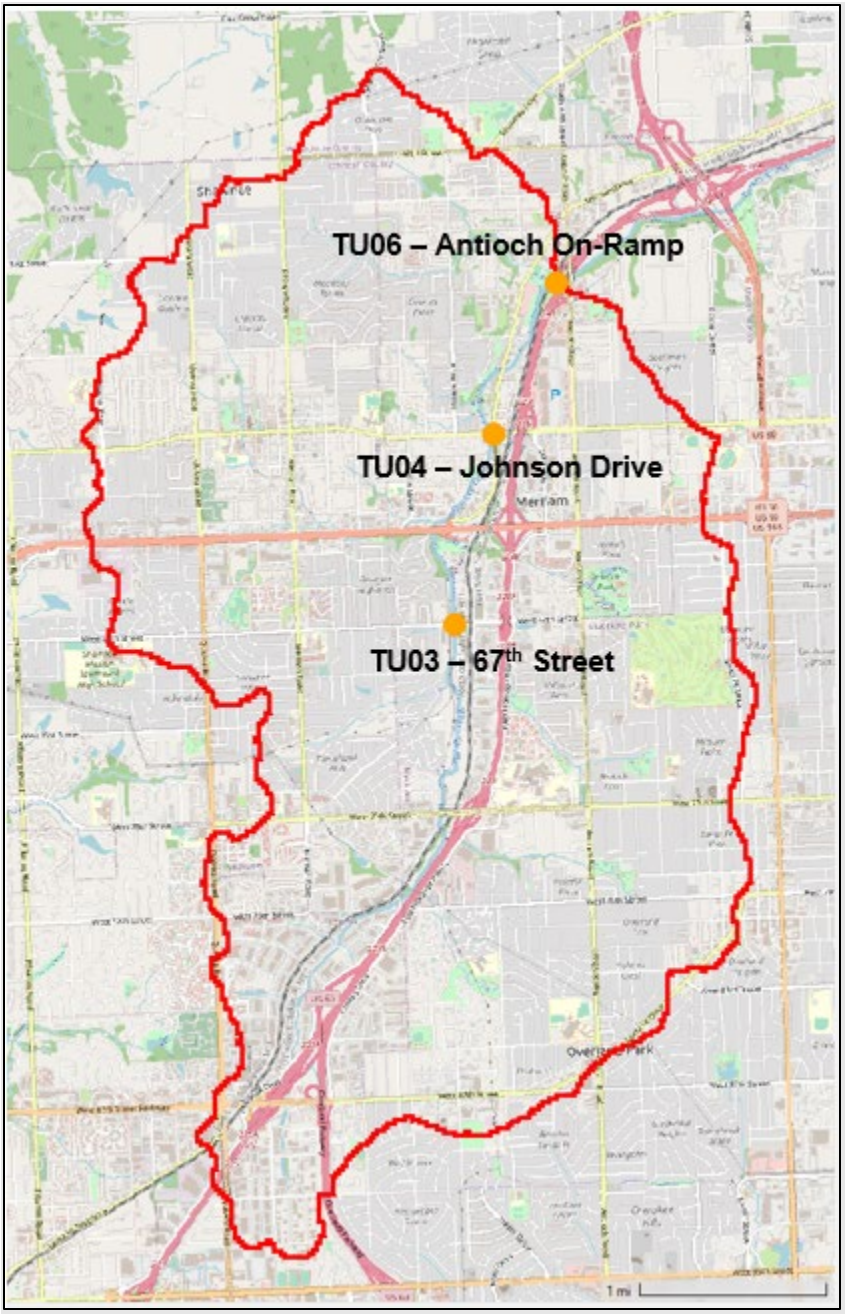


Figure 5. Orange dots indicate the locations of Stormwatch gauges TU03 – 67th Street; TU04 – Johnson Drive; and TU06 – Antioch On-Ramp within the Turkey Creek watershed upstream of I-35. The red perimeter indicates the approximate watershed extents of Turkey Creek upstream of where Turkey Creek passes under I-35 to the north. The modeled reach extends from I-35 at the downstream end to 67th Street at the upstream end. Flow direction is from south to north.

Table 5. Summary of computed differences between hydraulic model water surface and observed stage measurement at the respective 67th Street, Johnson Drive, and Antioch On-Ramp Stormwatch gauge locations.

Stormwatch ID	Location	26-27 July 2017 Elevation Delta	5-6 August 2017 Elevation Delta	21-22 August 2017 Elevation Delta
TU03*	67th Street*	0.00	0.00	0.00
TU04	Johnson Drive	2.69	4.12	2.63
TU06	Antioch On-Ramp	1.75	2.13	3.41

* This gauge was the upstream boundary condition location where a stage hydrograph was used. No difference in stage would be anticipated due to this.

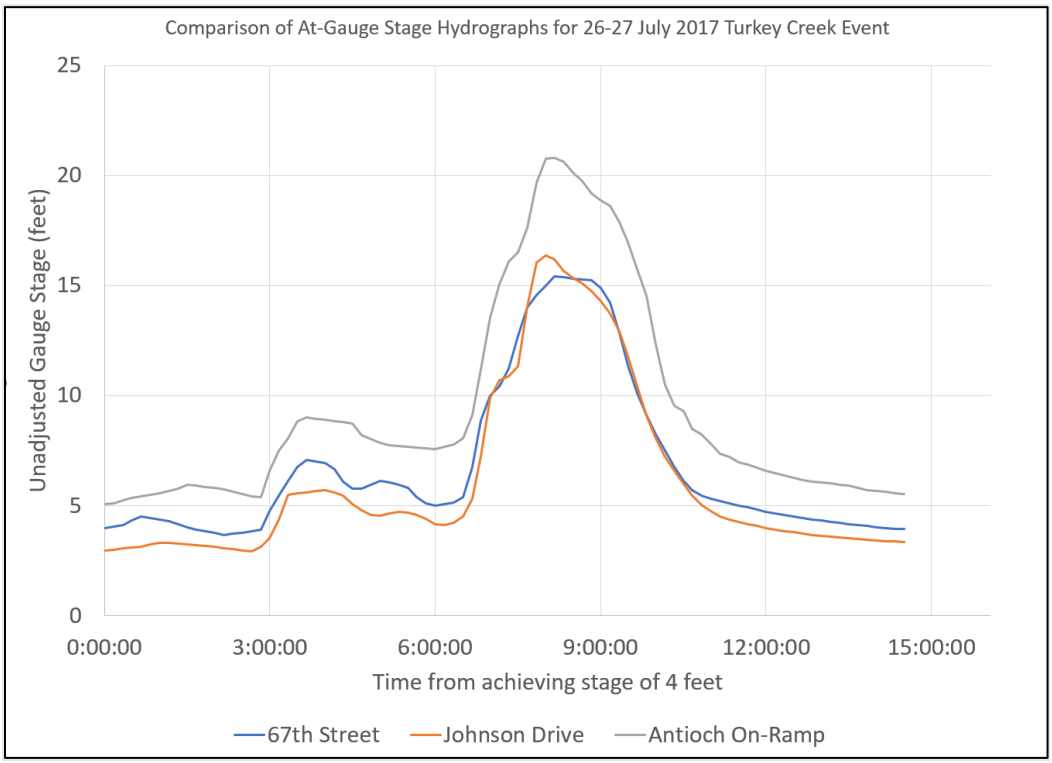


Figure 6. Stage hydrographs at 67th Street, Johnson Drive, and Antioch On-Ramp Stormwatch gauge locations for 26-27 July 2017 flow event.

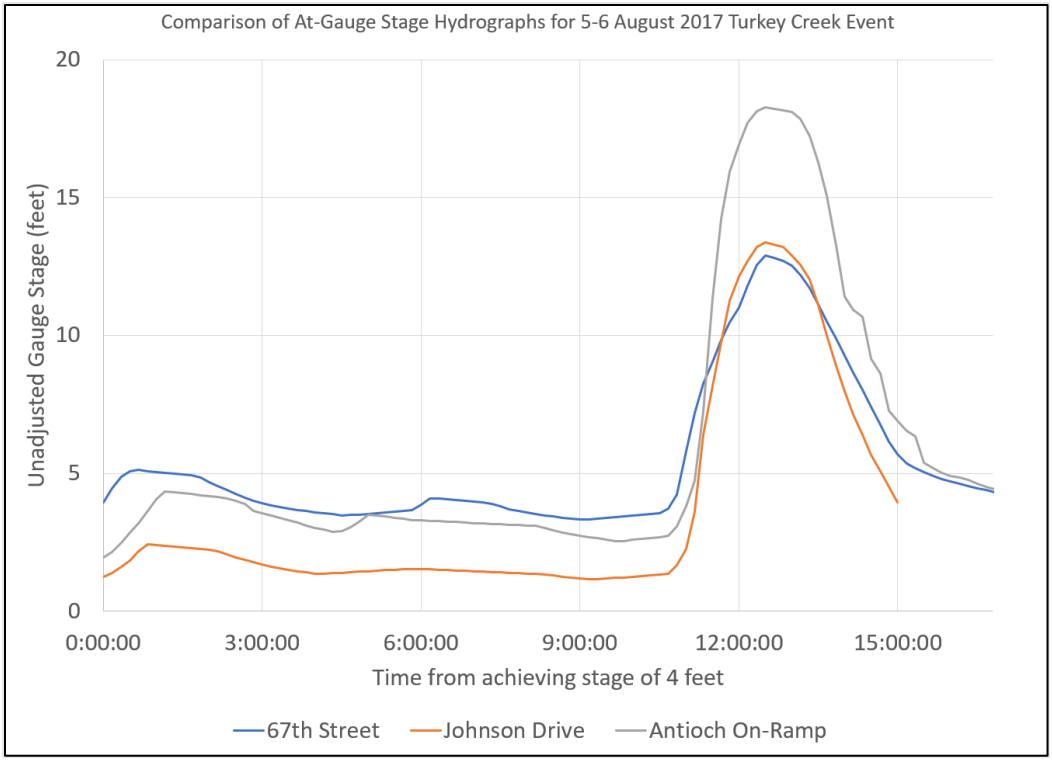


Figure 7. Stage hydrographs at 67th Street, Johnson Drive, and Antioch On-Ramp Stormwatch gauge locations for 5-6 August 2017 flow event.

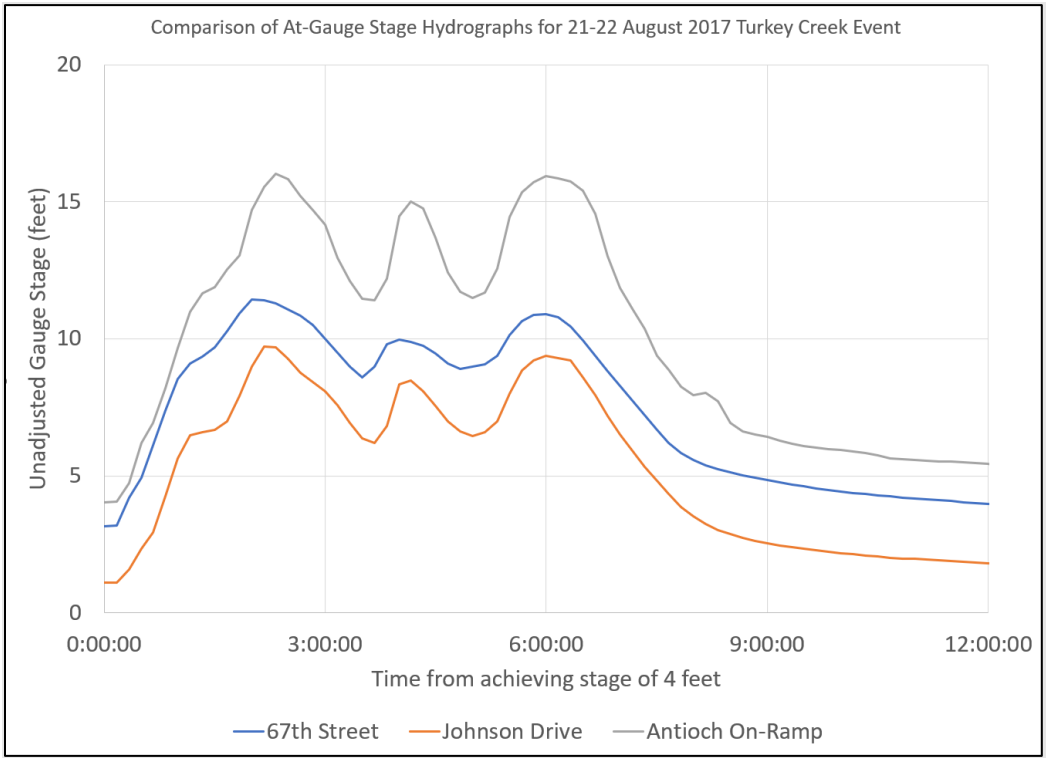


Figure 8. Stage hydrographs at 67th Street, Johnson Drive, and Antioch On-Ramp Stormwatch gauge locations for 21-22 August 2017 flow event.

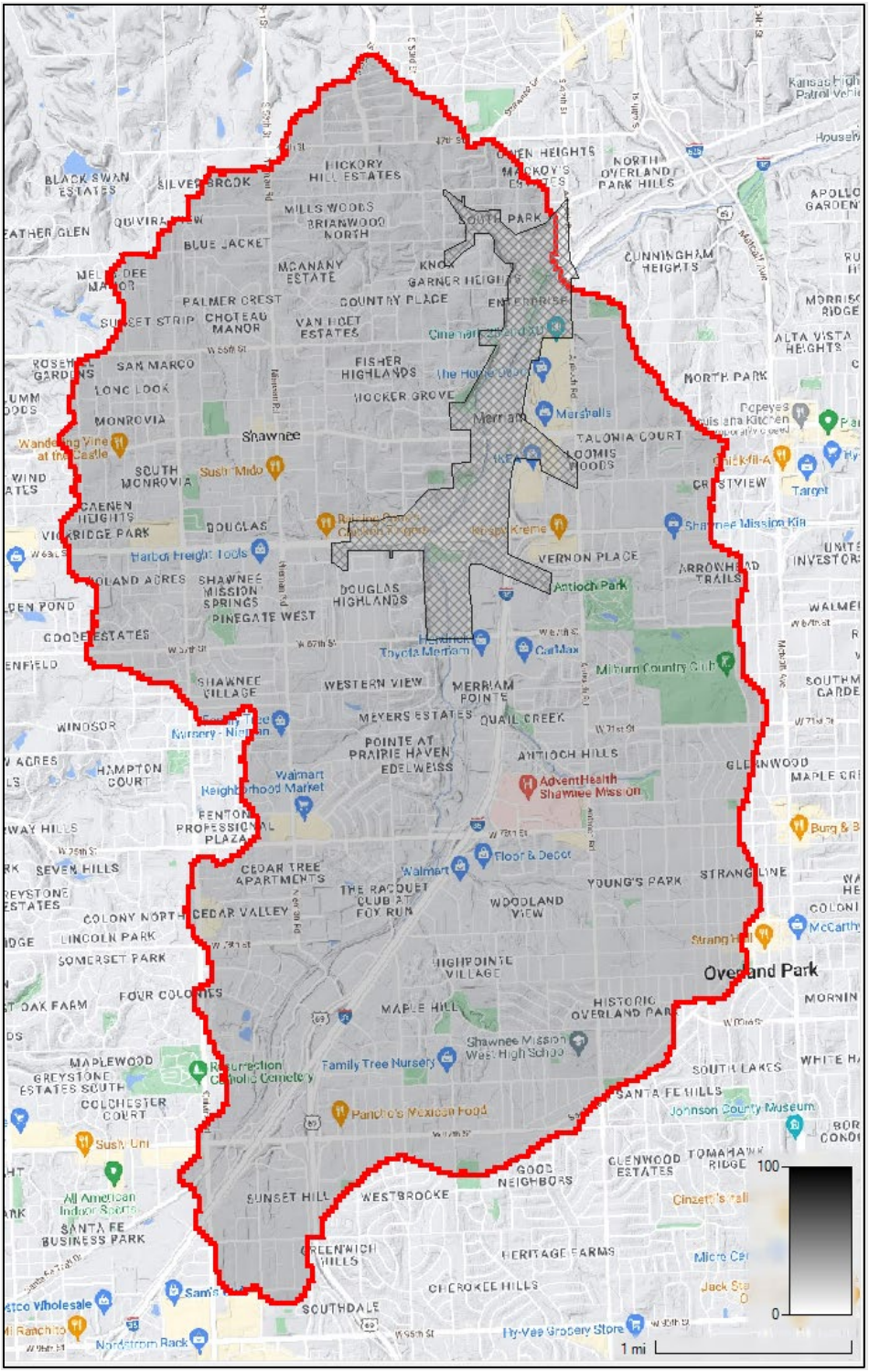


Figure 9. Approximate boundary for Turkey Creek watershed at Stormwatch gauge TU06 – Antioch On-Ramp at Turkey Creek.

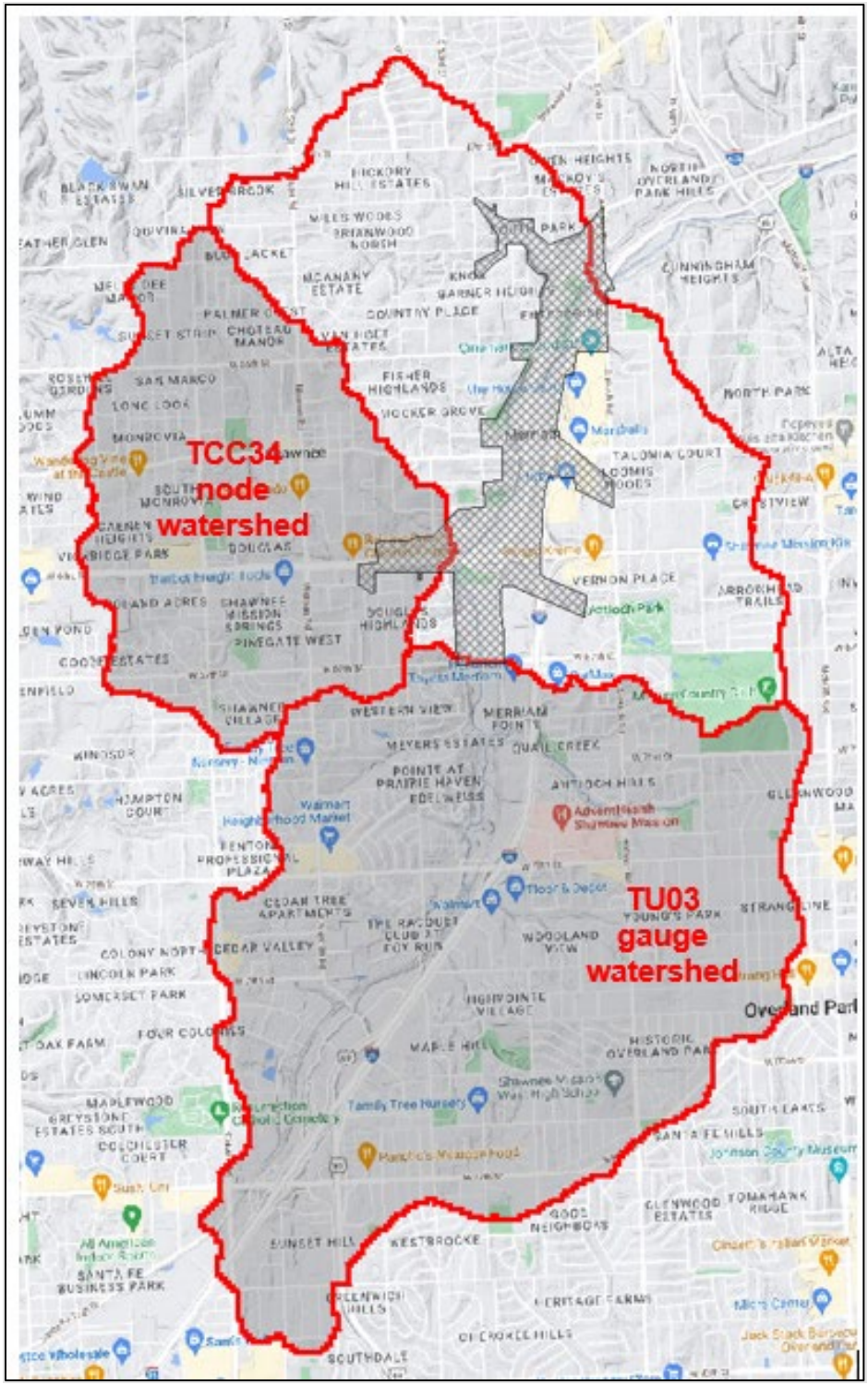


Figure 10. Approximate boundaries for Turkey Creek watershed at Stormwatch gauge TU03 – 67th Street at Turkey Creek and TCC34 node (i.e. the tributary which outlets adjacent to Shawnee Mission Parkway) from the hydrologic modeling. Larger boundary is extent of watershed boundary upstream of Stormwatch gauge TU03 – Antioch On-Ramp at Turkey Creek. Hatched area indicates simulated 2D mesh area.

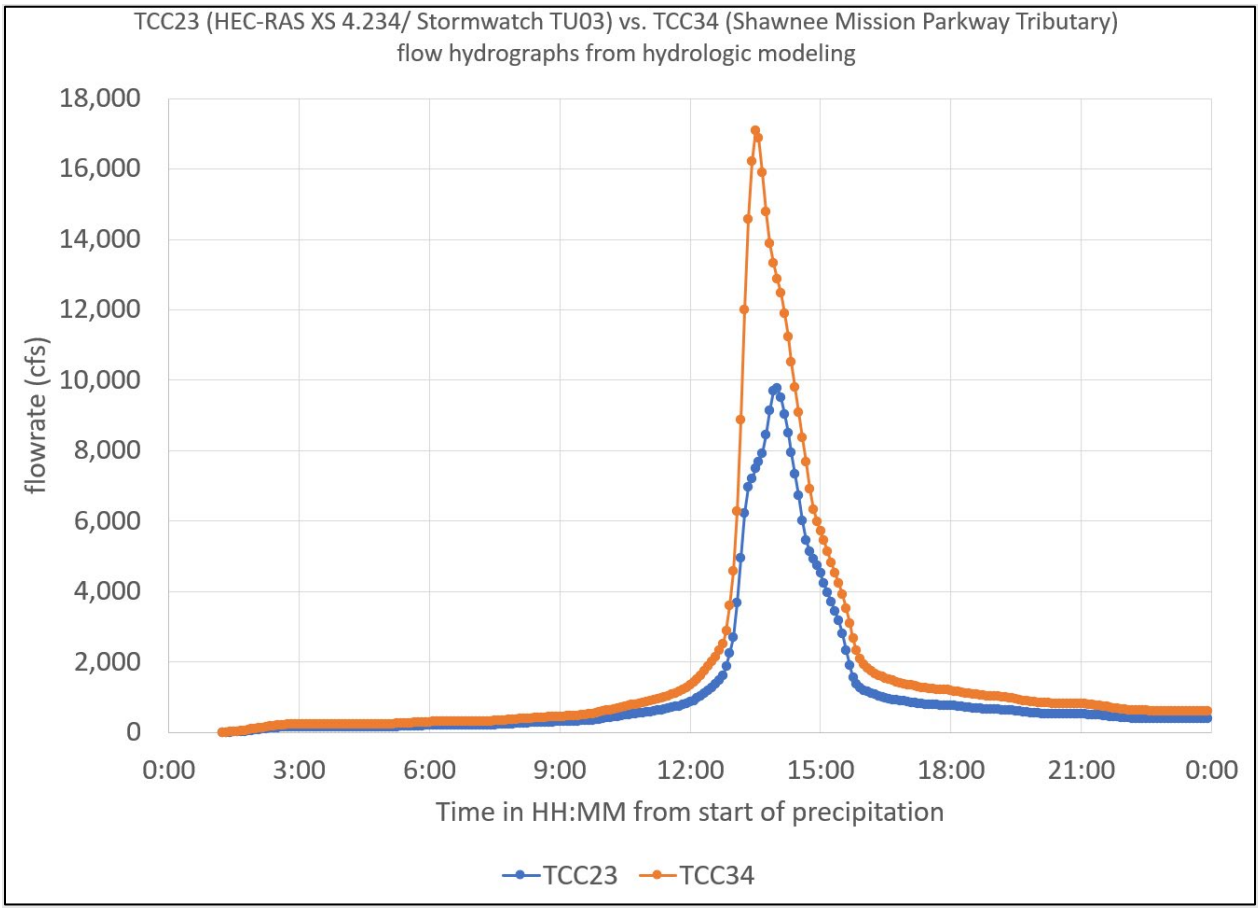


Figure 11. Comparison of flow hydrographs generated for nodes TCC23 and TCC34 from Feasibility Study hydrologic modeling.

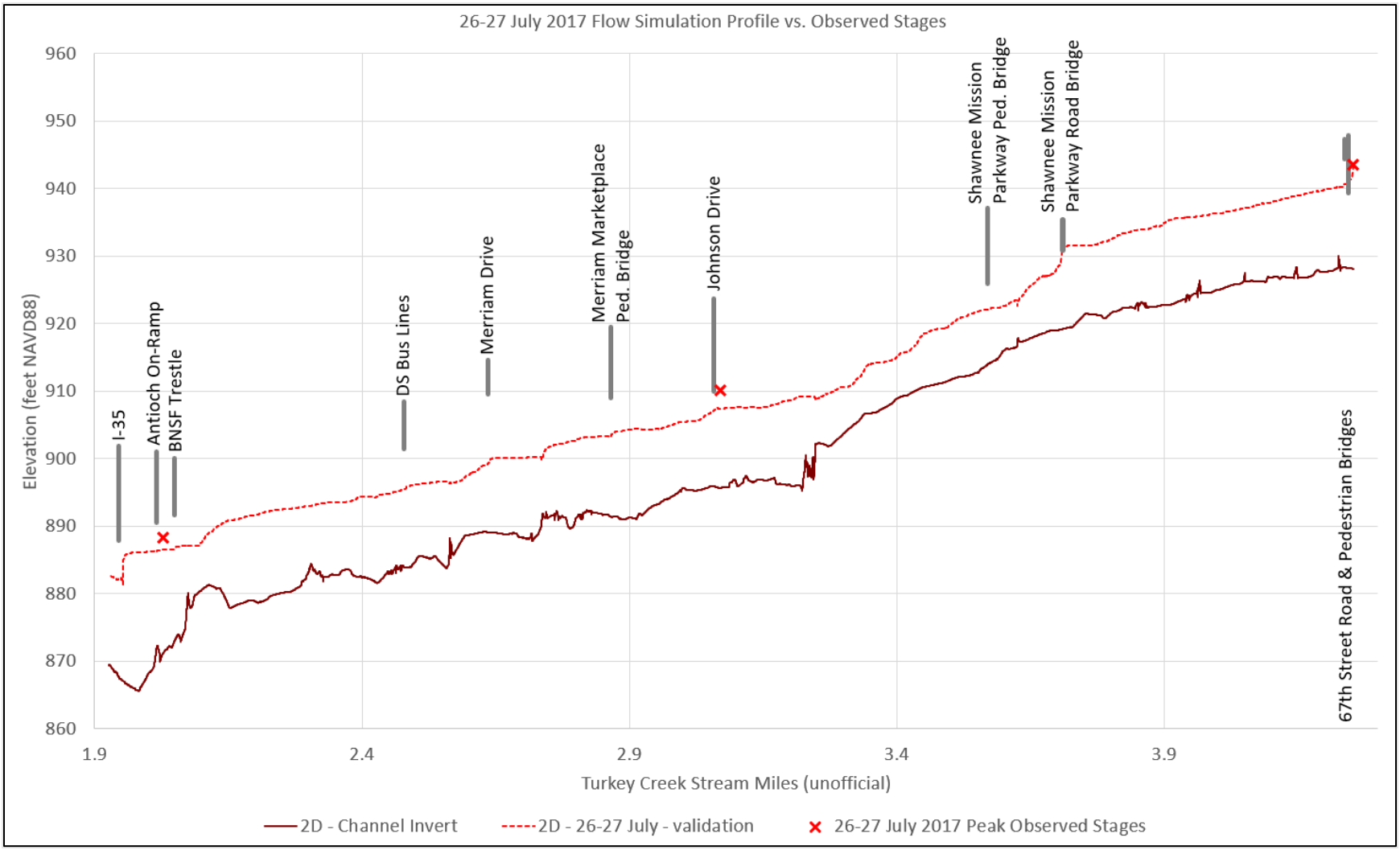


Figure 12. Comparison of hydraulic flow simulation for 26-27 July 2017 event to peak observed stage measurements at gauges TU03 – 67th Street, TU04 – Johnson Drive, and TU06 – Antioch On-Ramp.

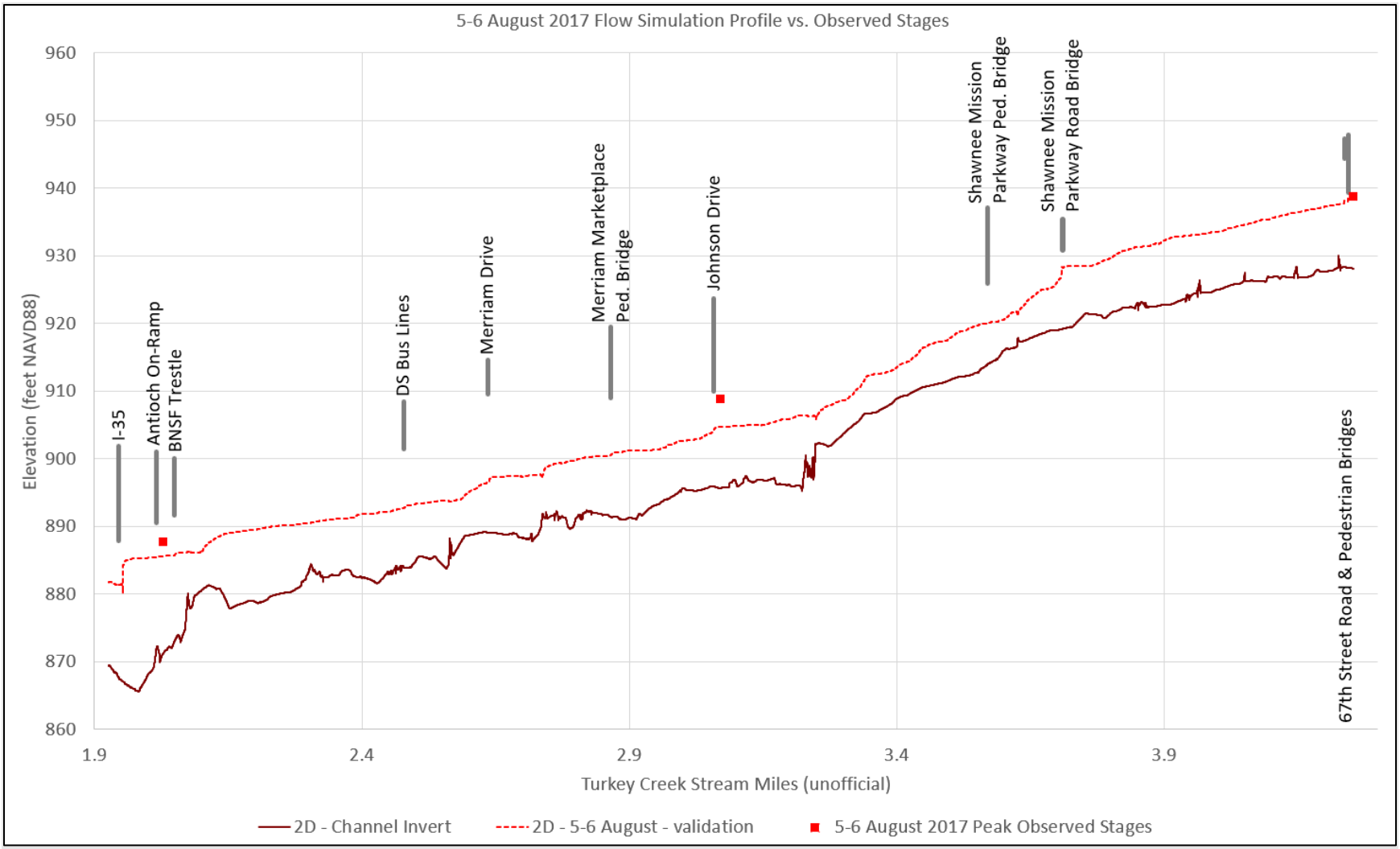


Figure 13. Comparison of hydraulic flow simulation for 5-6 August 2017 event to peak observed stage measurements at gauges TU03 – 67th Street, TU04 – Johnson Drive, and TU06 – Antioch On-Ramp.

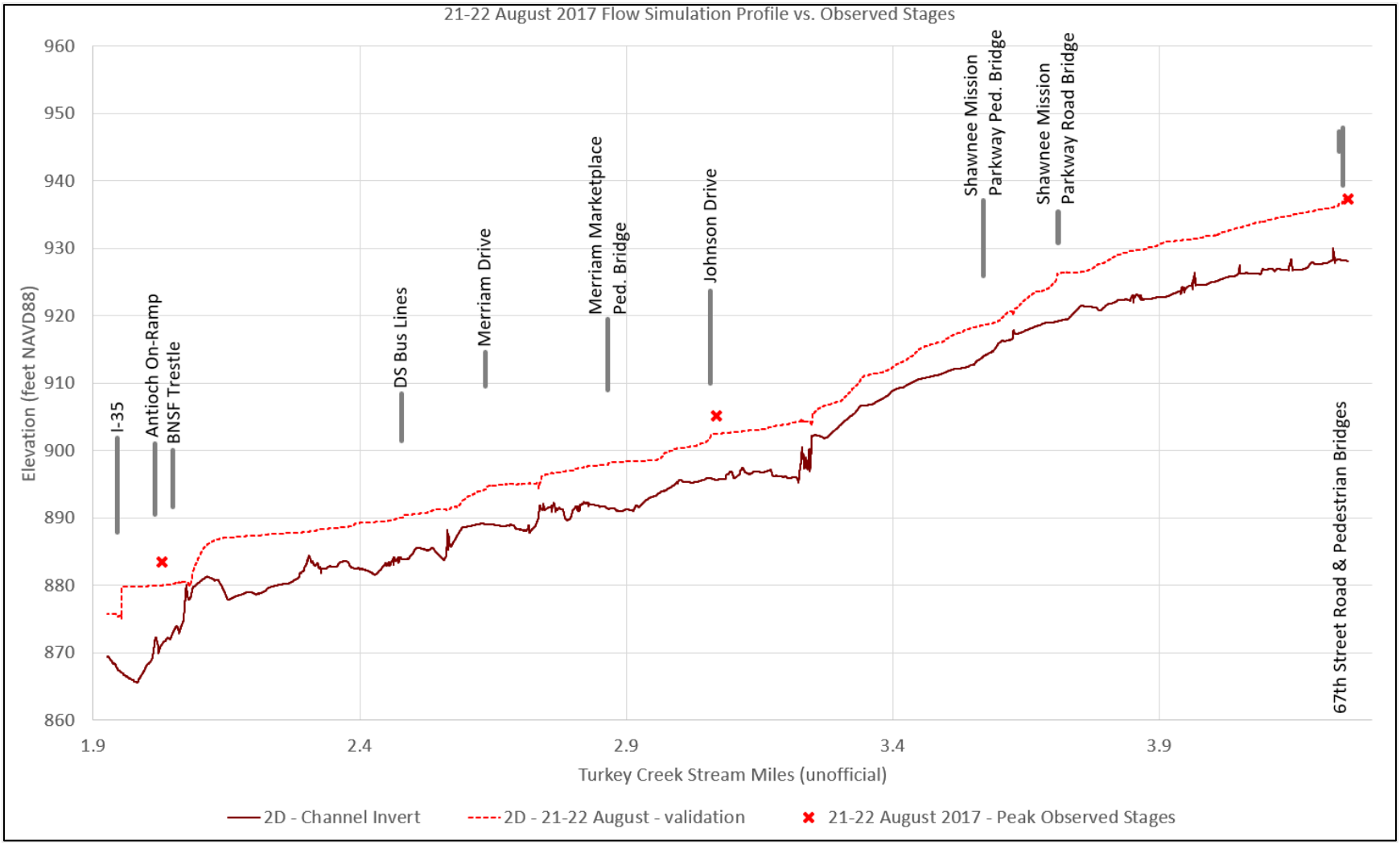


Figure 14. Comparison of hydraulic flow simulation for 21-22 August 2017 event to peak observed stage measurements at gauges TU03 – 67th Street, TU04 – Johnson Drive, and TU06 – Antioch On-Ramp.

E.4 Existing Conditions Comparison

Following the parameterization of the models, multiple methods of evaluation were identified using the more current two-dimensional modeling capabilities. The original one-dimensional Feasibility Study model was evaluated using a steady flow analysis only. The respective TP-40 and Atlas 14 24-hour precipitation accumulation values were evaluated over the entirety of the contributing watershed via the parameterized HEC-1 model, and only the peak discharges from the hydrologic model were entered at flow change locations within the HEC-RAS model as a steady flow model at cross-sections 2.065, 2.396, 2.813, 3.070, 3.665, 3.726, and 4.234.

For the two-dimensional analysis, two approaches were adopted for evaluation of the existing conditions. These were the use of a steady-unsteady simulation and the distribution of unsteady hydrographs within the Turkey Creek hydraulic model according to node location within the Feasibility Study hydrologic model.

E.4.1 Steady-Unsteady Simulation

The steady-unsteady simulation consists of using the same discharges adopted for the Feasibility Study over a 5-hour time interval to approximate a steady-state flow condition, like the steady-state modeling performed for the Feasibility Study. The flow change cross-section locations from the Feasibility Study were duplicated as boundary condition lines over the Turkey Creek channel, and constant value flow hydrographs were entered into the Unsteady Flow Data Editor within HEC-RAS to either add or subtract flow according to the differences computed from the flow change locations within the original steady-state model.

For the steady-unsteady flow simulation, two-dimensional boundary conditions are implemented as additive, in contrast to distinct value changes like they are in one-dimensional steady-flow HEC-RAS (at flow change locations). Therefore, the full upstream flow is entered, and then incremental positive or negative flow differences are entered at the respective downstream locations to add or subtract the necessary amount. A summary of the respective discharge increments, according to boundary condition location, are presented in Table 6.

Table 6. Summary of 1% and 0.5% AEP discharges from Feasibility Study and increments of discharge entered into two-dimensional existing conditions model at respective boundary condition locations.

Flow Change Cross Section (from Feasibility Study) (miles)	1% AEP Atlas 14 steady-state flow at flow change location (cfs)	Flow increment entered at respective 2D boundary condition (cfs)	0.5% AEP Atlas 14 steady-state flow at flow change location (cfs)	Flow increment entered at respective 2D boundary condition (cfs)
4.234 (Upstream)	10,047	10,047	11,248	11,248
3.726	10,381	334	11,629	381
3.665	16,855	6,474	18,610	6,981
3.07	17,691	836	19,657	1,047
2.813	17,348	-343	19,333	-324
2.396	16,613	-735	18,641	-692
2.065 (Downstream)	18,733	2,120	21,068	2,427

E.4.2 Distribution of Unsteady Hydrographs at Hydrologic Model Node Locations

For the second existing conditions flow profile evaluation, flow hydrographs from the hydrologic model were distributed within the hydraulic model at boundary condition lines within the model geometry according to their geographic node location within the hydrologic model used for the Feasibility Study. The computed flow hydrographs from the HEC-HMS model were scaled according to a ratio of the peak HEC-1 discharge divided by the the peak HEC-HMS discharge based upon the 1% and 0.5% AEP Atlas 14 discharges documented in the Feasibility Study.

Variations in the peak discharges were observed between the HEC-1 and HEC-HMS results, even though the HEC-1 modeled flows and geometry were directly imported into HEC-HMS. A spreadsheet adjustment of the HEC-HMS hydrographs was performed to exactly match the peak HEC-1 Atlas 14 discharges hydraulically simulated by the Feasibility Study. The HEC-HMS model was chosen to be carried forward for use for this evaluation due to the age of HEC-1 and the fact that the software is no longer updated or maintained by HEC for use. While the HEC-1 software is available, current software security measures in place within USACE makes its continued use challenging and HEC-HMS the preferred software tool.

For the unsteady flow simulation using the geographically-distributed unsteady flow hydrographs, HEC-HMS computed hydrographs were scaled according to the ratio of the peak discharges from the HEC-1 model to the peak discharge from the HEC-HMS model. Since HEC-HMS provides a timestep-by-timestep accounting of the flow contributions for each node, as well as at each combining node, the ratio of each subordinate node's discharge to the combining node discharge was used to scale the HEC-HMS computed discharges at each timestep according to the ratio of the peak HEC-1 discharge to peak HEC-HMS discharge via a spreadsheet. A sample computation for a single timestep flow adjustment at combining node TCC37 is presented below in Table 7.

An observed consequence of the insertion of unsteady flow hydrographs at the respective geographic node locations from the Feasibility Study hydrologic modeling into the existing conditions 2D hydraulic model is that the 2D mesh and digital raster terrain can intercept or generate storage of flows, or backwater, at the respective hydrologic model node locations.

Storage of flows or backwater may occur due to confinement or constriction of the stream due to a bridge or RCB. Ponding or interception may occur due to features in the terrain surface such as ponds or surface depressions, such as ditches or detention basins, which do not have drainage features, such as culverts or conduits, implemented within the hydraulic model. Due to the generalized parameterization which occurred with the hydrologic modeling of HEC-1 and HEC-HMS, detention and/or interception due to existing infrastructure and/or land surface modifications are not visualized and/or may not be accounted for in the same manner as they are in the hydraulic modeling.

Table 7. Sample adjustment computation for HEC-HMS individual timestep discharge to correspond with HEC-1 discharges evaluated by the Feasibility Study at combining node TCC37 (with subordinate contributing nodes TCR60, TC-T50, TC79, and TC67)

Node:	HEC-HMS Example Timestep Discharge (cfs):	Proportion of contributing node discharge to combining node TCC37 (percentage):	Arithmetic to scale contributing node from HEC-HMS (20,953 cfs) to correspond with Feasibility Study (HEC-1) peak discharge (17,691 cfs):	Scaled HEC-HMS discharge value used at individual timestep (cfs) in HEC-RAS:
TCR60	18.8	52.5%	35.8 x 17,691 / 20,953 x 52.5%	15.9
TC-T50	8.5	23.7%	35.8 x 17,691 / 20,953 x 23.7%	7.2
TC79	4.6	12.8%	35.8 x 17,691 / 20,953 x 12.8%	3.9
TC67	4.0	11.2%	35.8 x 17,691 / 20,953 x 11.2%	3.4
TCC37	35.8	100%	35.8 x 17,691 / 20,953 x 100%	30.2

A summary of the relationship between the Feasibility Study hydraulic model, and the hydrologic node corresponding with the flow change location is presented in Table 8. A description of the approximate node locations is presented in Table 9.

Table 8. Translation of Feasibility Study hydraulic model cross-sections to contributing hydrologic nodes

Feasibility Study Reach	Feasibility Cross-Section	Node – Contributing Nodes
TC-M80	4.234	TCC23 – none (upstream inflow boundary condition location)
TC-M80	3.726	TCC235 – TCR70*, TC94, TC-T71
TC-M80	3.665	TCC34 – TCR65*, TC-T59, TC66
TC-M80	3.07	TCC37 – TCR60*, TC-T50, TC79, TC64
TC-M80	2.813	TCC38 – TCR50*, TC-T40, TC57
TC-M80	2.396	TCC39 – TCR42*, TC48, TC41
TC-M80	2.065	TCC44 – TCC39*, TC-T30, TC19, TC38

* Locations on Turkey Creek included along routing reach. These nodes were not included in modeling as boundary condition locations.

A comparison of computed water surface profiles along the stream centerline adopted for the Feasibility Study is presented in Figure 15. It is noted that this profile comparison only represents the water surface elevations computed along the stream centerline adopted for the Feasibility Study. It does not represent the two-dimensional variability in water surface computed by this hydraulic analysis. The design water surface at the proposed levee/floodwall alignment for this project may be higher or lower than what is presented in this comparison due to two-dimensional effects, such as superelevation. A summary of the computed average, median, maximum, and minimum differences between the Feasibility Study and the current existing conditions models is presented in Table 9.

Table 9. Location descriptions of modeled hydrologic node locations

HEC-1/HEC-HMS Node	Boundary Condition Location Description
TCC23	Upstream boundary condition near XS 4.234. On Turkey Creek and immediately upstream of 67th Street Bridge
TC94	Approximately halfway between 67th Street and Shawnee Mission Parkway on the right bank of Turkey Creek
TC-T71	Right bank tributary to Turkey Creek immediately upstream of Shawnee Mission Parkway
TC-T59	Along tributary immediately east of Shawnee Mission Bridge over Turkey Creek.
TC66	Between Mastin Street and the left bank tributary to Turkey Creek immediately downstream of Shawnee Mission Parkway
TC-T50 & TC64	Along right bank of Turkey Creek approximately 250 feet upstream of Johnson Drive Bridge over Turkey Creek
TC79	Along left bank of Turkey Creek approximately 410 feet upstream of Johnson Drive Bridge over Turkey Creek
TC-T40	Along left bank tributary which drains into Turkey Creek near the Merriam Marketplace Pedestrian Bridge. Approximately 450 feet west of top of left bank.
TC57	Along right bank of Turkey Creek approximately 150 feet downstream of the Merriam Marketplace Pedestrian Bridge.
TC48	Along right bank of Turkey Creek. East of I-35 and approximately straight east of W 55th Street.
TC41	Along left bank of Turkey Creek. Approximately 90 feet upstream of the DS Bus Lines Bridge.
TC-T30	Along left bank of Turkey Creek. Along left bank tributary to Turkey Creek near Waterfall Park. Approximately 190 feet east of Merriam Drive.
TC19	Along left bank of Turkey Creek. Along left bank tributary to Turkey Creek near Waterfall Park. Approximately 70 feet east of Merriam Drive and 100 feet west of TC-T30.
TC38	Along left bank of Turkey Creek. Approximately 500 feet east-southeast of the intersection of W 53rd Street and Merriam Drive

Table 10. Computed statistics for differences between 1% AEP Feasibility Study Pre-Project Profile and respective existing conditions profiles

Scenario	Average Difference (ft)	Median Difference Value (ft)	Maximum Difference (ft)	Minimum Difference (ft)
1% AEP Atlas 14 Steady-Unsteady	0.67	0.82	3.89	-4.42
1% AEP Atlas 14 w/ flow hydrographs	0.41	0.65	5.43	-2.96

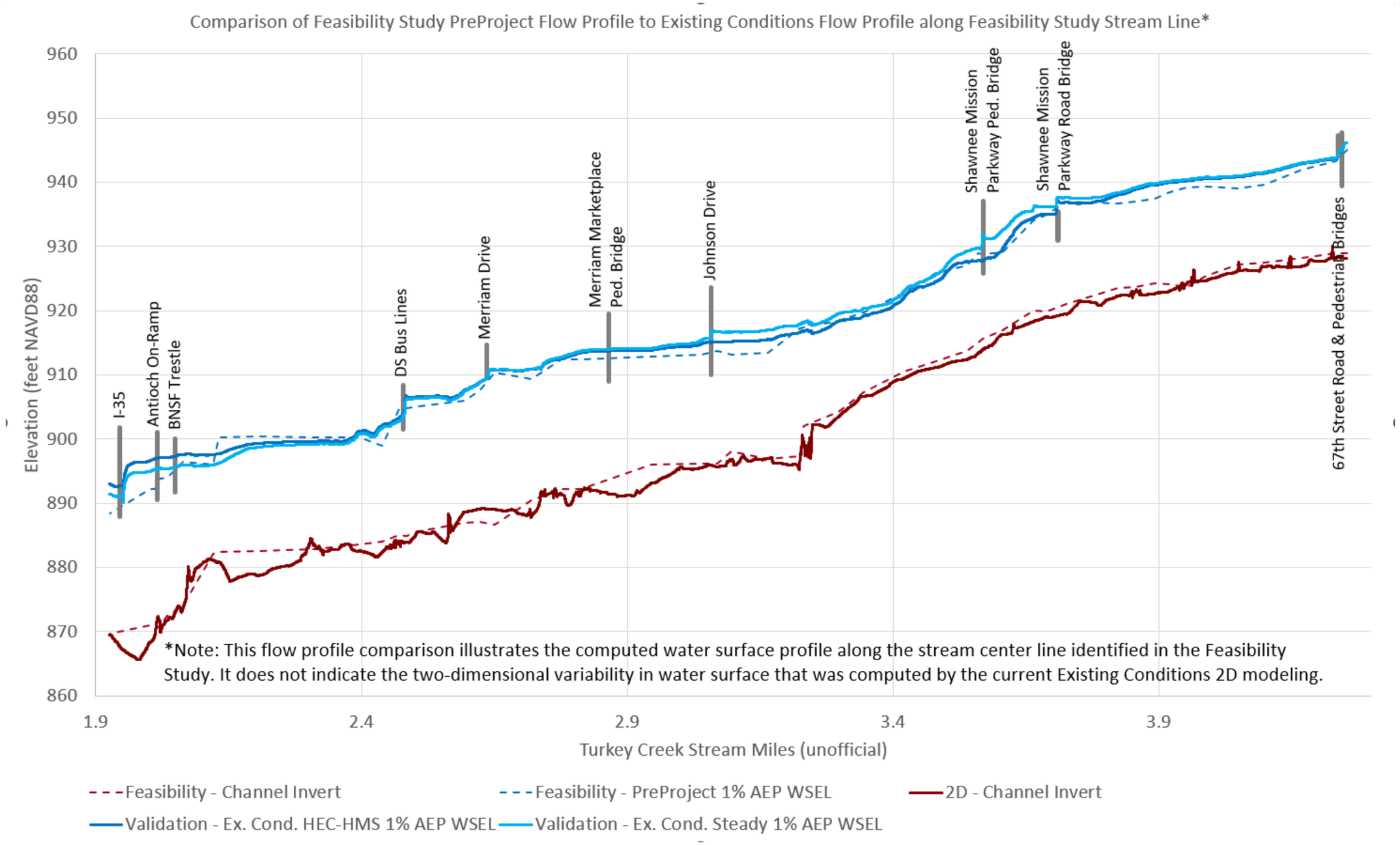


Figure 15. Comparison of pre-project or existing conditions 1% AEP, Atlas 14-based water surface profiles from the Feasibility Study (blue dashed line), and the current analysis (dark blue is the flow hydrograph analysis and light blue is the steady flow hydrograph analysis). The channel invert along the stream centerline is indicated by the brown lines (Feasibility Study is dashed; current analysis is solid).

THIS PAGE IS INTENTIONALLY BLANK.

APPENDIX F
Environmental

Kansas Historical Society

6425 SW 6th Avenue
Topeka KS 66615

phone: 785-272-8681
fax: 785-272-8682
kshs.culturalresources@ks.gov

Laura Kelly, Governor

KSR&C # 22-12-128
February 3, 2023

Gina Powell
Archeologist
US Army Corps of Engineers
Via Email

Re: Preliminary Review, Upper Turkey Creek streambank protection, Merriam – Johnson County

We have reviewed the request for comment on a potential project that would be subject to review in accordance with 36 CFR Part 800. In reviews of this nature, the State Historic Preservation Office (SHPO) determines whether a federally funded, licensed, or permitted project will adversely affect properties that are listed or determined eligible for listing in the National Register of Historic Places. According to the submission, the proposed project will include the acquisition and potential demolition of residential structures of various ages. Without knowing which structures will be demolished, we are unable to comment on the architectural impact. However, we have reviewed the project for archeological potential and have no concerns with this aspect of it.

We expect to receive additional information regarding the structures to be demolished as they become available. Any work that does not include demolition of structures may proceed. Please submit any comments or questions regarding this comment to Lauren Jones at 785-272-8681, ext. 214 or lauren.jones@ks.gov.

Sincerely,

Patrick Zollner
Director, Cultural Resources Division
Deputy State Historic Preservation Officer

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS, KANSAS CITY DISTRICT
601 E. 12TH STREET, 635 FEDERAL BLDG
KANSAS CITY, MO 64106-2824

December 16, 2022

Mr. Patrick Zolner
Acting State Historic Preservation Officer
Kansas Historical Society
6425 SW 6th Avenue
Topeka, KS 6615-1099

Dear Mr. Zolner:

The U.S. Corps of Engineers, Kansas City District (USACE) is planning a feasibility study for flood risk management actions in a short stretch of Upper Turkey Creek in the city of Merriam, Johnson County, Kansas. The project area is a 1.25-mile section of the creek near downtown Merriam in Section 12 T12S R24E (Figure 1). This letter continues consultation for the project under Section 106 of the National Historic Preservation Act that was begun in May 2012, Kansas R&C number 12-OS-134. The letter from your office is attached for your reference in Appendix A. Cultural resources figures and maps are in Appendix B and the project details are in Appendix C.

Project Description

The Recommended Plan is a levee and floodwall along Turkey Creek in the city of Merriam. The features extend from Shawnee Mission Parkway to Merriam Drive, a stretch that includes Merriam's main downtown reach, as well as a commercial and industrial area just south of Johnson Drive. Most of the protected area is on the right bank of Turkey Creek (Figure 2). In the feasibility plan, four properties have been identified for removal (Appendix C).

Background

An archaeological background review of the project area was conducted including a search of the Kansas Archaeological Viewer (online); Kansas Historic Resources Inventory (KHRI, online) the NRCS Web Soil Survey (online); the online National Register of Historic Places (NRHP); LiDAR imagery; historic plat and topographic maps; General Land Office (GLO) records, and in-house USACE records.

As part of the 2012 cultural resources review for the previous feasibility study of the project, USACE archeologist David Cain performed a background review of the available resources and a field reconnaissance survey of the proposed project area. The crossing of the road connecting Ft. Leavenworth and Ft. Scott is depicted crossing Turkey Creek just north of Shawnee Mission Parkway (Figure 3). The trail is also noted in the project area with a marker sign. No LiDAR indications of the trail are present in or near the project area (Figure 4). Field investigations by Cain (2012) showed that the soils in the area are severely disturbed by urban development and historic properties of

an archaeological nature are very unlikely in the channel, a conclusion with which your office concurred in 2012.

No other archeological surveys have been performed since Cain's and no archeological sites have been recorded near the project area since his review. Cain was not able to access the KHRI database. As part of the current study, the KHRI online database was reviewed for architectural resources within or near the project area. Several resources are recorded within a few blocks of the study area (Figure 5), including the RW Hocker Subdivision spec house listed on the National Register of Historic Places (NRHP).

Two architectural resources are recorded directly adjacent to the Turkey Creek project area (Figure 5). The closer, southern building is Merriam School at 5820 Merriam Drive (KHRI 091-2463). The oldest part of the building was built in 1872 and was used as a school until 1911. Since then, it has been modified and used for retail space. The building is in good condition but has not been assessed for NRHP eligibility. It is only 45 m from Turkey Creek but there are no plans to affect it as part of this project. The second building is the subsequent Merriam school, built in 1911, located at 5701 Merriam Drive (KHRI 091-3781). The building served as a school until 1969 and served several other purposes including as the Merriam Community Center from 1990. The building is now gone, having been demolished at some time in 2021, per Google Earth historical images.

Five properties indicated for acquisition and removal in the feasibility plan (Appendix C) are:

- Property JP35000000-0009: a general office building (5568 Merriam Dr.). Built 1963.
- Property JF241212-3005: a warehouse structure (5726 Merriam Dr.). Built 1986.
- Property JF241212-3006: a warehouse/office structure (5816 Merriam Dr #A). Built 1962.
- Property JF84000009-0001A: a shed (5836 Merriam Dr). Possibly built 1986.
- Property JP28000033-0024D: a residence (9675 W. 61st St.). Built 1986.

The first three buildings appear to be concrete block commercial structures of various ages. The shed appears to be corrugated metal. Because it is very likely these properties will change during the design phase of the project, they will be surveyed and uploaded to KHRI at that time to aid their evaluation for NRHP eligibility.

Further consultation with your office will take place during the design phase of this project when specific locations and actions are determined. The project locations, such as the levee and detention basin footprints, will be reviewed again at that time. If your office has information on any cultural resource that may be affected by the proposed project of which we may be unaware, we would take such information into account for our final determination of effect.

Thank you for your consideration in this matter. If you have any questions or have need of further information please contact Gina Powell, USACE Kansas City Archeologist, at Gina.S.Powell@usace.army.mil or at (816) 389-2320.

Sincerely,

Dr. Gina S. Powell, Archeologist

Enclosure

References

Cain, David I.
2012 Archeological Survey of Upper Turkey Creek Basin, Flood Risk Management Project Johnson County, Kansas. On file, USACE Kansas City. B5163.

Appendix A.
2012 SHPO Response Letter

KSR&C No. 12-OS-134

6425 SW 6th Avenue
Topeka, KS 66615

Kansas

phone: 785-272-8681
fax: 785-272-8682
cultural_resources@kshs.org

Kansas Historical Society

Sam Brownback, Governor
Jennie Chinn, Executive Director

May 15, 2012

Timothy Meade
Cultural Resource Manager
Kansas City District, Corps of Engineers
700 Federal Building
Kansas City, Missouri 64106-2896

RE: Flood Risk Management Project
Upper Turkey Creek Basin
Johnson County

Dear Mr. Meade:

In accordance with 36 CFR 800, the Kansas State Historic Preservation Office has reviewed a report entitled *Archeological Survey of Upper Turkey Creek Basin, Flood Risk Management Project Johnson County, Kansas*, by David I. Cain. We find the report to be acceptable and concur with the recommendation that the project as proposed should have no effect on properties listed in the National Register of Historic Places or otherwise identified in our files. This office has no objection to implementation of the flood risk management project.

This information is provided at your request to assist you in identifying historic properties, as specified in 36 CFR 800 for Section 106 consultation procedures. If you have questions or need additional information regarding these comments, please contact Tim Weston at 785-272-8681 (ext. 214) or Kim Gant at 785-272-8681 (ext.225).

Sincerely,

Jennie Chinn, Executive Director and
State Historic Preservation Officer

Timothy Weston for

Patrick Zollner
Deputy SHPO

Appendix B. Figures



Figure 1. Location of project area in Merriam, Johnson County, Kansas.

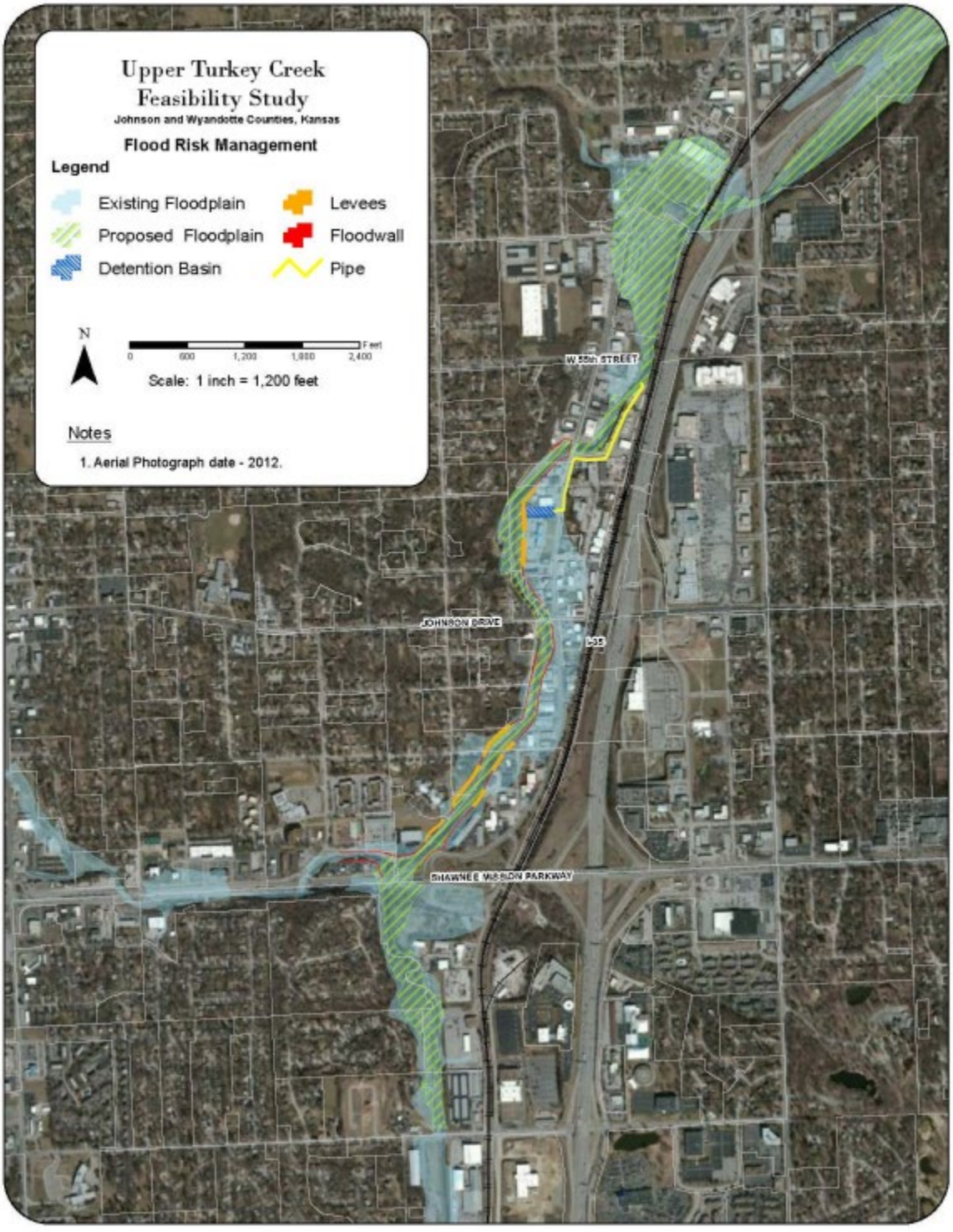


Figure 2. Preferred plan: combination of flood walls and levees.

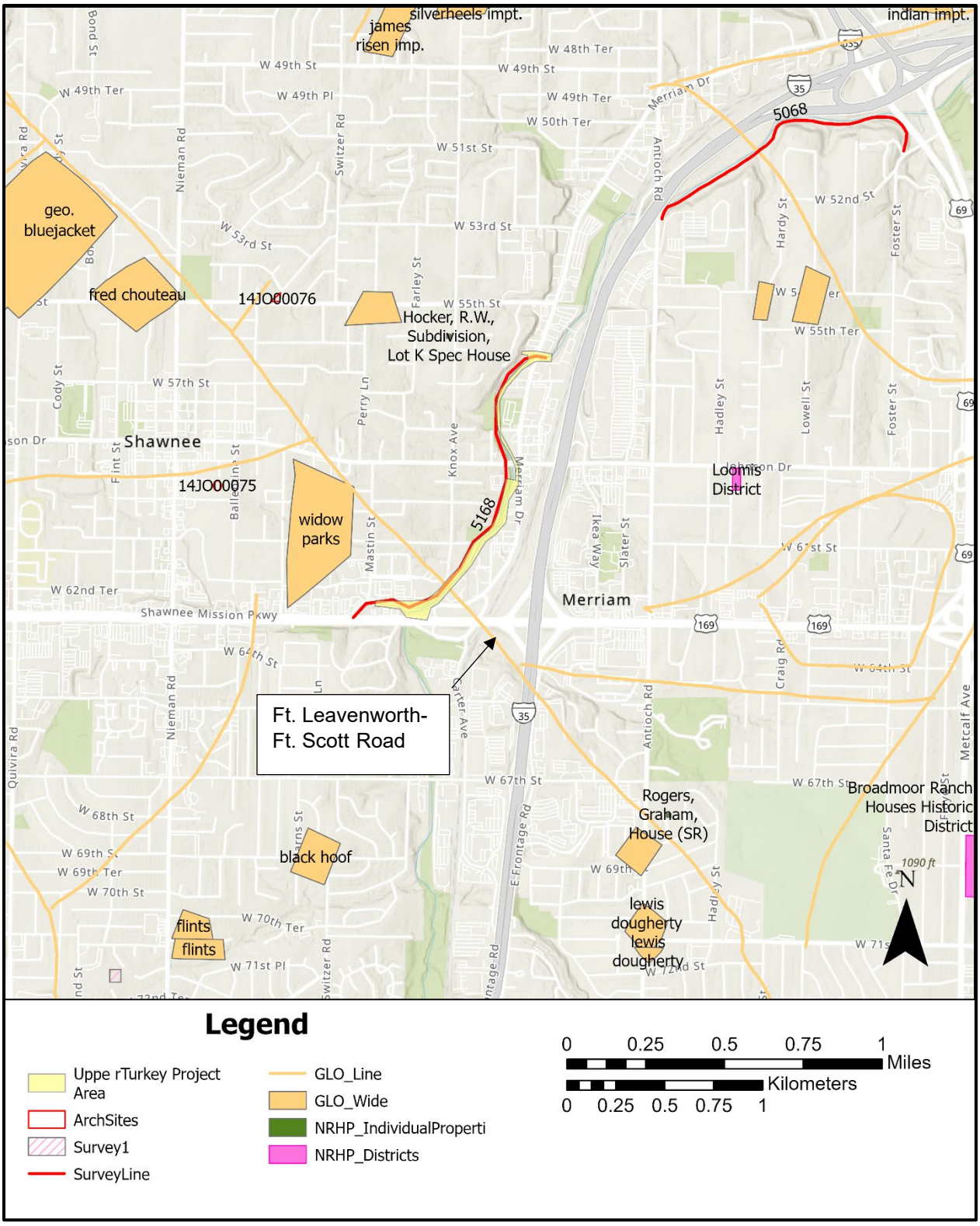


Figure 3. Cultural resources near and crossing the project area.

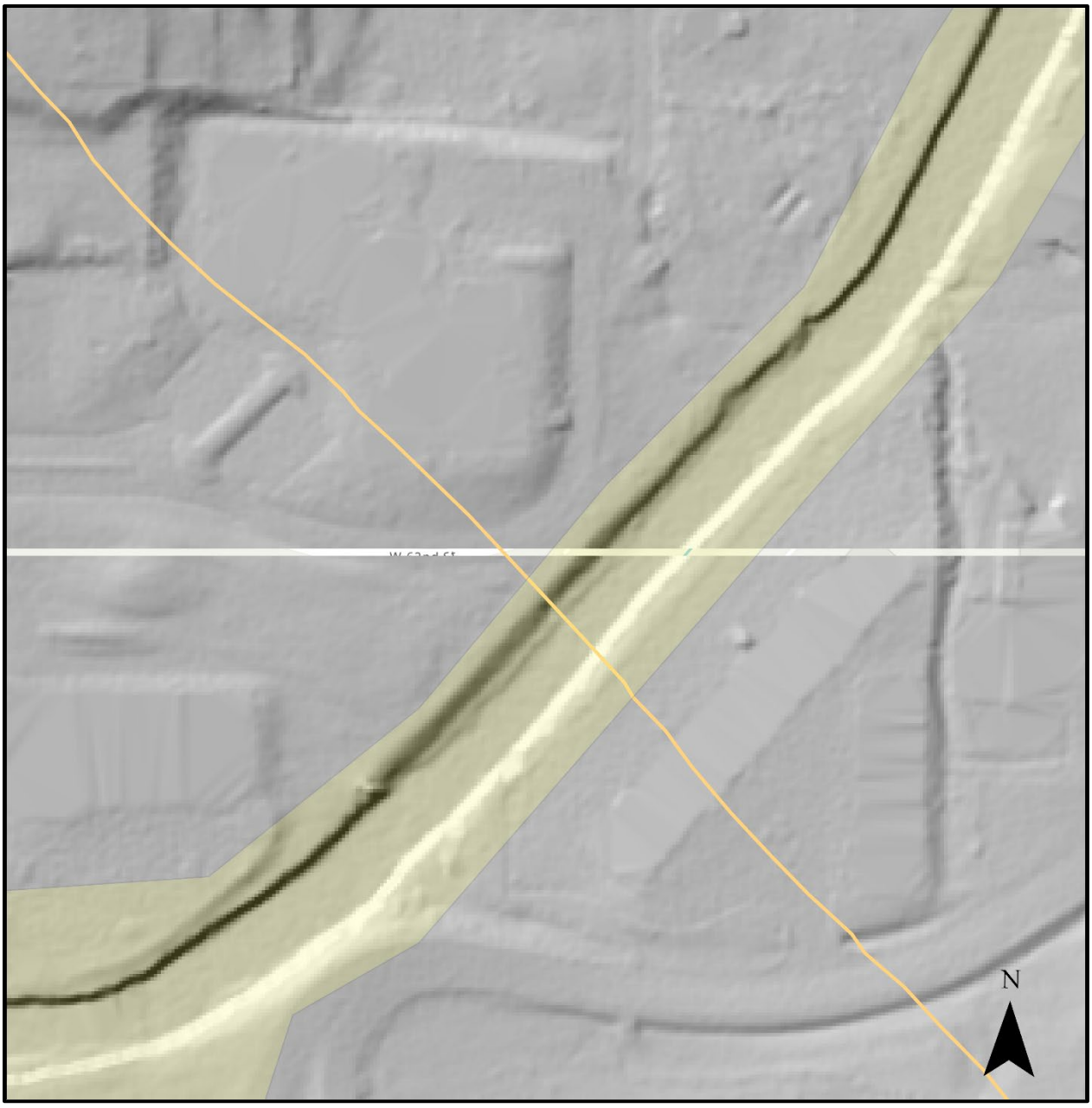


Figure 4. Hillshade image of the military road crossing over the project area.

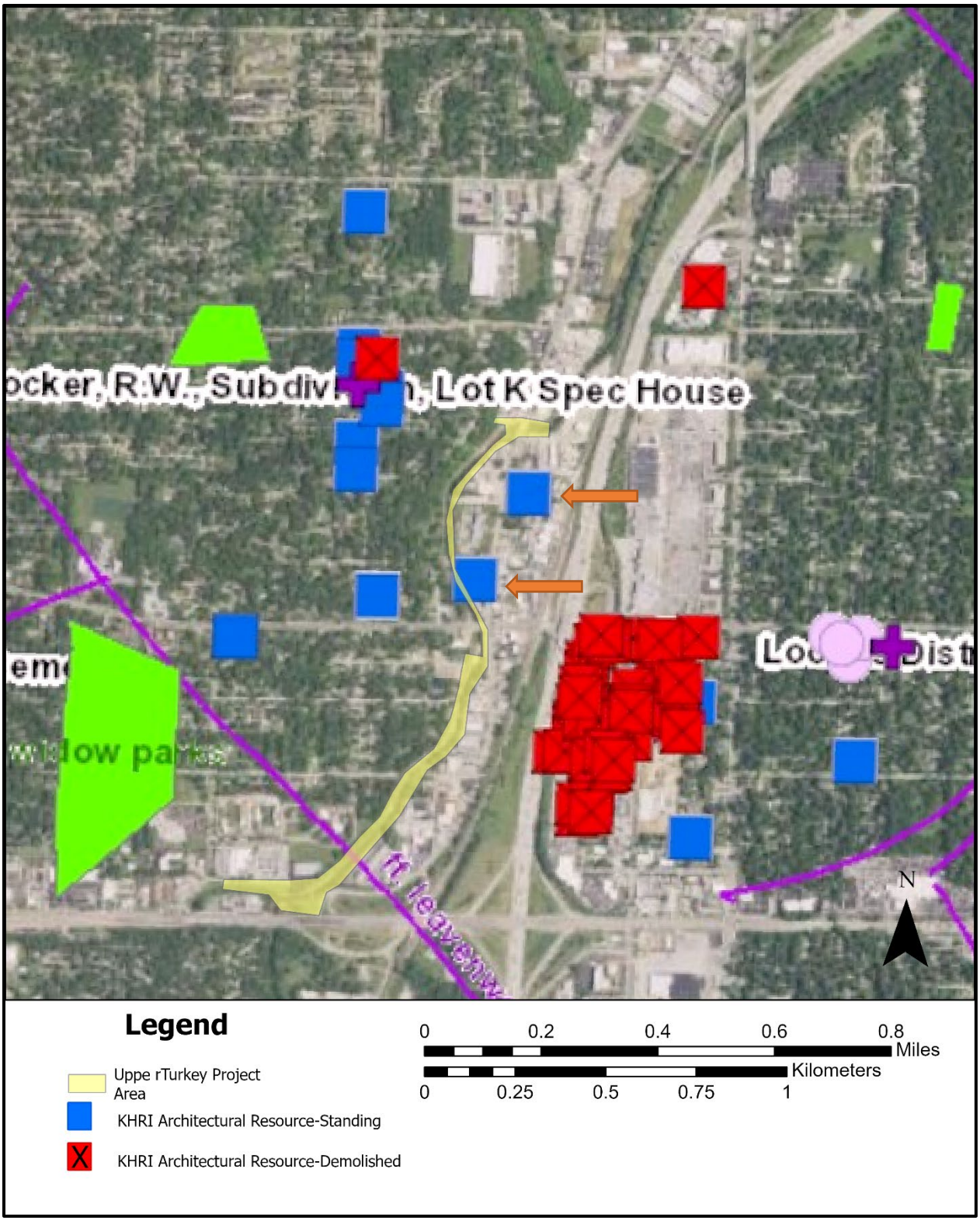
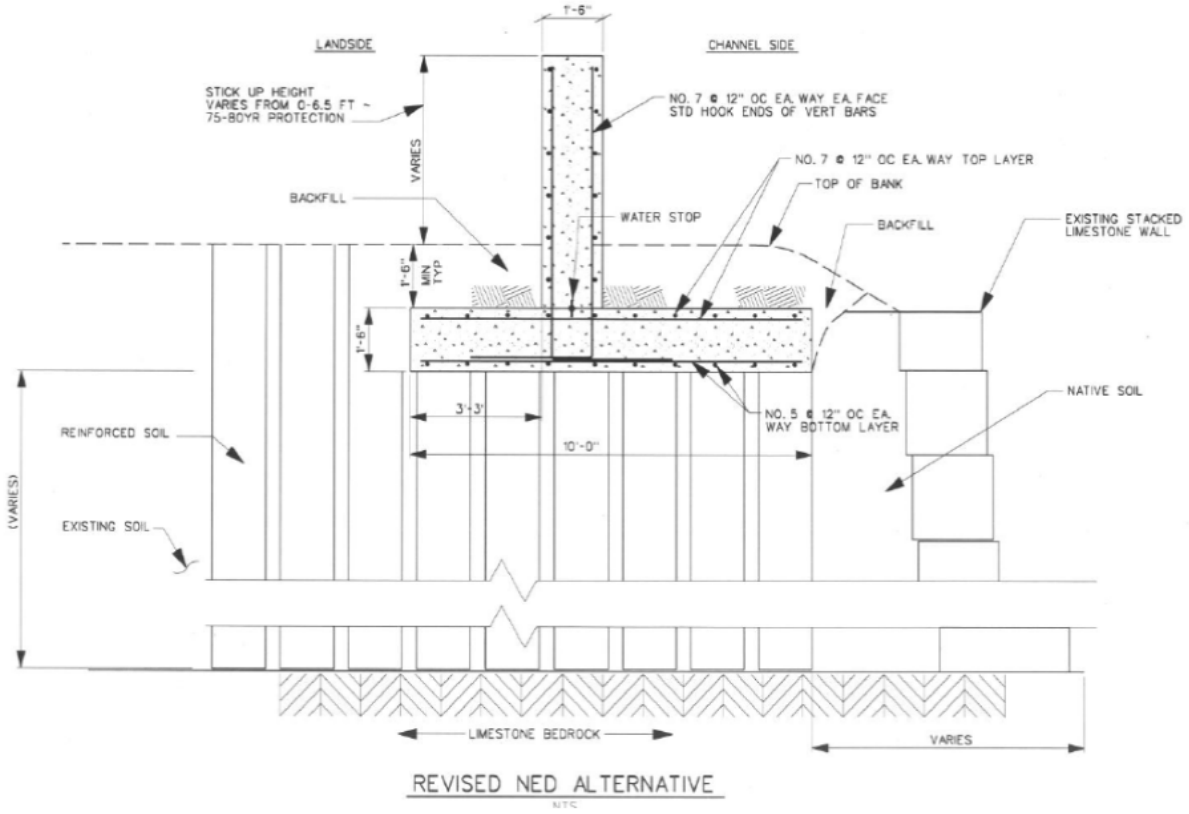


Figure 5. KHRI-listed resources near the project area. Two resources at arrows are discussed in the text.

APPENDIX G
Typical Floodwall



Typical Floodwall Detail (FS)

APPENDIX H
Cost Estimate

APPENDIX H – COST ESTIMATE

COST ESTIMATE

H.1 General

A reprised feasibility level estimate has been developed and is provided under separate cover upon request. The scope of work includes the recommended alternative to construct a flood risk management project in the City of Merriam in Johnson County, Kansas along Turkey Creek. The recommended plan is a levee and floodwall plan in the City of Merriam. The current working estimate is not considered “For Official Use Only.”

H.2 Contract Acquisition

The acquisition method is assumed to be full and open IFB.

H.3 Estimated Project Duration

The estimated duration for this contract is 40 months. It is assumed the construction will begin in February 2027. The midpoint of construction is assumed to be in December 2028.

H.4 Basis of Estimate

The basis of the estimate is the final feasibility study dated December 2014.

H.5 Estimate Preparation

This cost estimate was prepared using the Micro Computer Aided Cost Engineering System (MCACES) Second Generation (MII), version 4.4.2. The following supporting databases were used in the preparation of the cost Estimate:

H.5.1 Cost Book

CB22EN (2022 MII English Cost Book); each unit cost was analyzed and adjusted to reflect current costs.

H.5.2 Labor Rates

This estimate uses a prevailing wage labor rates for Jackson County, MO. The Davis Bacon Wages from decision KS20210026 dated 1 June 202022 were used for

preparing the estimate.

H.5.3 Equipment Rates

This estimate is based on the latest available/supported MCACES MII equipment rate database (EP20R05), which has been updated for the estimate to more accurately represent current prices.

H.5.4 Contractors/Subcontractors

The estimate assumes the majority of the work will be performed by the prime but some subcontracting will be needed by the prime to execute the project.

A surety bond rate was assumed for the Prime Contractor. The bond rate is calculated by the MCACES MII software.

The following markups were applied to the Prime Contractor's work: Job Office Overhead, Home Office Overhead, Profit, Bond, Contingency. Currently a contingency of 44% is being applied until further design detail is provided. Other specific percentages are retained in MII.

The following markups were applied to any subcontracting work: General Overhead and Profit.

An equipment adjustment factor has been applied to the 2020 Equipment prices to update the equipment to current pricing. An equipment adjustment factor has been applied to all equipment items. The adjustment factor has been derived by assuming and escalation factor of 2.0% per year.

A material adjustment factor of 15.03% has been applied to the 2022 Cost Book Material prices to update the materials to current pricing. The adjustment factor has been derived from the Engineering News Record index for construction material.

A sub bid adjustment factor of 4.42% has been applied to the 2022 Cost Book Sub Bid prices to update the sub bids to current pricing. The adjustment factor has been derived from the Engineering News Record index for construction cost.

H.6 Project Owner Markups

The owner also has markups on the project level that are applied after contractor markups. These markups are include: escalation and supervisory and administration. below.

Escalation = 17.5%

Construction Contingency = Included in design contingency.

S&A = 10%

H.7 Cost Drivers and Risks

Updating the cost and schedule risk analysis (CSRA) after each design submittal will help better define cost and schedule risks and provide confidence levels in being able to complete the project at various funding amounts. Below are high level risks.

- Bridge headwall modification and bridge scour not adequately scoped in feasibility
- Flood Wall Lengths
- Ground Improvement - AGP Depth and Width
- Real Estate Limits
- Relocations
- Flood Wall Lengths
- Scope Uncertainty
- Market Conditions and Competition

H.8 Market Conditions and Competition

A large factor in the total project cost will be the number of bids received as open competition has a significant impact on receiving a fair and reasonable price. Historical data shows a small labor pool and the lack of labor could affect cost and schedule.

H.9 Scope Uncertainty

There are many unknowns in the current scope that could impact the total project cost. The estimate will change throughout design as the scope becomes better defined. Below are a few of the unknowns.

- Fill Material sourcing is not final
- Secant Wall or AGP T Wall designs are not final.
- Unknown condition of limestone retaining wall.
- Possible new bridge
- Long term city plans

THIS PAGE IS INTENTIONALLY BLANK.

APPENDIX H – SURVEY REPORT

This page was intentionally left blank for duplex printing.

STOCKWELL

Topographic Survey, GPR and Drafting of Upper Turkey Creek, Merriam, Kansas

Performed by Stockwell Engineers Inc.
for the U.S. Army Corps of Engineers, Kansas City District
Under Contract No. W912DQ-19-D1006
Delivery Order No. W912DQ23F1001

Coordination of the work started prior to crews being on site between Joe Whitlock and Brett Kennedy with Stockwell, Greg Fox and David Kuxhausen of Woolpert and Katrina Marx, John Calcara, Courtney Forsythe and Brian Vanbockern of the USACE, Kansas City District.

This task order involves project setup, checking of existing control, topographic data collection, GPR investigation and drafting of collected survey data for use in upcoming design projects along the Upper Turkey Creek corridor in Merriam, Kansas. The field work for this project was completed from February to April of 2023. Temperatures ranged from 25˚ to 75˚ F.

Stockwell Engineers Survey Crews mobilized to the site in Merriam, Kansas on the 21st of February 2022. Our crews set a base point (CP-1) towards the south end of the site near “Skate City”. We performed two separate OPUS solutions on the point to obtain an accurate position. We then completed checks on NGS points R328 and Shawnee2. During these checks we determined that horizontally the position was accurate but vertically it was about 0.3’ difference. We then adjusted to NGS point R328 to establish the elevation of CP-1.

Once the position was established Stockwell’s crews checked existing control established by BHC for the Merriam Drive project for the City of Merriam in drawing page 4 of 94 in the vicinity of the Upper Turkey Creek survey and determined that it was within 0.1’ horizontally and vertically which we felt was acceptable. Our crews then went through the site and set control points every 800’ to 1000’ to use while surveying with robotic total stations. After the control was set and marked, we completed a level loop across the control using a Trimble DiNI level.

During the first week on site February 20 thru February 24, Bill Welbaum with Woolpert, Inc. was onsite doing GPR work on the 4 sites in question. During that time our surveys crews would meet with Bill a couple of times each day to pick up all marked out areas before they were lost due to traffic or work activities that were going on in the area. *(See attached report from Woolpert describing their findings and showing pictures of field conditions.)*

Upon completion of the Level Loop our crews began performing the topographic survey for the site utilizing both GNSS and Robotic Total Stations using three to four crew members at all times.

STOCKWELL

During this week we completed the topographic survey of the creek bed and the intersection around Mastin and Shawnee Mission Parkway north to the north end of the Skate City property.

Stockwell's crews continued to work on the topographic survey for the next 6 weeks. During that time, our crews continued utilizing both GNSS and Robotic Total Stations along with a Trimble SX10 Terrestrial Scanner to scan the overhead power and communication lines that cross the project.

Along with the ground survey our crews measured the inverts of all manholes, storm inlets and boxes and utility vaults and manholes. Manhole information sheets were created and provided to Woolpert Staff to add all necessary information to the topographic survey drawing.

Throughout this Delivery Order there were only a few issues. One minor one was that the property owner at 5920 Merriam Drive did not receive our letter informing them that there was going to be a survey going on around their property. When our crew was working on site they were told to leave and stay off the property. Upon this interaction our crews left the site and contacted Katrina Marx with the USACE, Kansas City District to let her know what the situation was. Katrina then talked to some people at the City of Merriam, and they stopped over at the property, talked with the property owner and explained what we were doing and they then allowed us back on the property to complete the topographic survey. Stockwell's Crews arrived the next morning and completed the survey on that property.

The biggest issue throughout the topographic survey was getting the underground utility locates completed. Stockwell Engineers, Inc. called in three different locate tickets with little to in most cases no locates being marked in the field. Joe Whitlock called all utility companies that were contacted as part of the Kansas State One Call System and spoke with representatives of the companies trying to get their locating contractor to mark the lines in the field. Through all of this Joe Whitlock spoke to the different utility companies and to USIC, LLC who was the utility locating contractor that wasn't performing their work. After quite a bit of back and forth they assured me that all of this would be located, and they would let Joe Whitlock know when the work was complete. Mr. Whitlock was never contacted by USIC, he traveled to Merriam, KS the first week in June to pick up a couple of building corners and finished floor elevations, while on site he walked the site and didn't find any new markings that were marked as part of the Upper Turkey Creek Topographic Survey.

STOCKWELL

Throughout the duration of the survey of Upper Turkey Creek Stockwell Engineers has been in contact with Katrina Marx and other staff from USACE Kansas City District. We had the kickoff meeting, CADD progress and review meetings and numerous phone calls and emails to discuss the progress and issues that have been noticed.

Stockwell Survey Crews:
The daily field data collection was overseen by Joe Whitlock.

Stockwell Crew Members:
Logan Hanzel
Dan Feucht
Jared Reed
Trenton Olson

Stockwell sub-contracted the GPR and Drafting portion of the work to Woolpert, Inc., which is Stockwell's team member on the IDIQ for this contract.

Equipment utilized by Stockwell Engineers Inc. for this survey included:

- ♦Trimble TSC7 Data Collectors
- ♦Trimble TSC3 Data Collectors
- ♦Trimble R10-2 GNSS Receivers
- ♦Trimble R12i GNSS Receivers
- ♦Trimble R8 Model 3 GNSS Receivers
- ♦Trimble R6 Model 3 GNSS Receiver
- ♦Trimble S6 Robotic Total Station
- ♦Trimble SX10 Scanning Total Station
- ♦Trimble Business Center Software Version 5.2.

The horizontal datum for the project is Kansas State Plane North (1501), NAD83 in U.S. Survey Feet. The vertical datum used was NAVD88, (Geoid 18) also with all elevations in U.S. Survey Feet.

All survey field notes were kept in a field book and transferred to an electronic daily notes' spreadsheet. Photographs were taken when needed to assist in answering questions that come up. Manhole Information Sheets were also used to better describe the interior of the manholes giving depths to the inverts, size and materials of pipes, and the present use. These sheets were filled out in the field and have been since scanned and saved for possible future use.

Respectfully Submitted,

Brett R. Kennedy

Brett Kennedy, PLS
Stockwell Engineers, Inc.



WOOLPERT
ARCHITECTURE | ENGINEERING | GEOSPATIAL

Field Report

Upper Turkey Creek **Kansas City, KS** **Date Written:** ***July 27, 2023***	
Order Number : 10017662	**Field Report Number : 10017662**

Date of Visit	Time of Visit	Weather	Present at Site
February 20, 2023	7:30am	Clear	Bill Welbaum

Introduction:

Ground Penetrating Radar (GPR) was deployed to investigate the potential existence of abandoned underground fuel / storage tanks at four (4) different locations withing the project area.

Objectives:

Identify the potential existence of abandoned underground fuel / storage tanks.

Methodology:

Woolpert deployed a one-man field crew outfitted with the Sensors and Software LMX200 Ground Penetrating Radar using ultra-wideband 250 MHz frequency. The GPR unit was used to assess the possible existence of buried structures. Any identified abnormalities in the soils were painted on the ground as part of that survey. Those marks were then surveyed by Stockwell Engineers. GPR data was reviewed in real time. The LMX200 unit deployed did not store raw data files.

Survey Results:

Any soil inconsistencies or areas that the GPR indicated that soil abnormalities were outlined with paint on the surface, photographed, and reviewed with the survey crew that was on-site.

Data Analysis and Interpretation:

The most common feature seen in GPR data is hyperbolic reflections. These hyperbolas are produced by subsurface objects or interfaces that reflect the radar waves back to the antenna. The shape and position of the hyperbolas provide information about the depth and geometry of subsurface structures. When hyperbolic reflections were seen in the GPR display, paint marks were placed to outline the data findings in real-time.

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030

Appendices:



5568 Merriam Dr 1



5568 Merriam Dr 2



5568 Merriam Dr 3

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



5568 Merriam Dr 4



5568 Merriam Dr 5



5568 Merriam Dr 6

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



5568 Merriam Dr 7



5726 Merriam Dr 1



5726 Merriam Dr 2

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



5726 Merriam Dr 3



5726 Merriam Dr 4



5726 Merriam Dr 5

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



5726 Merriam Dr 6



5726 Merriam Dr 7

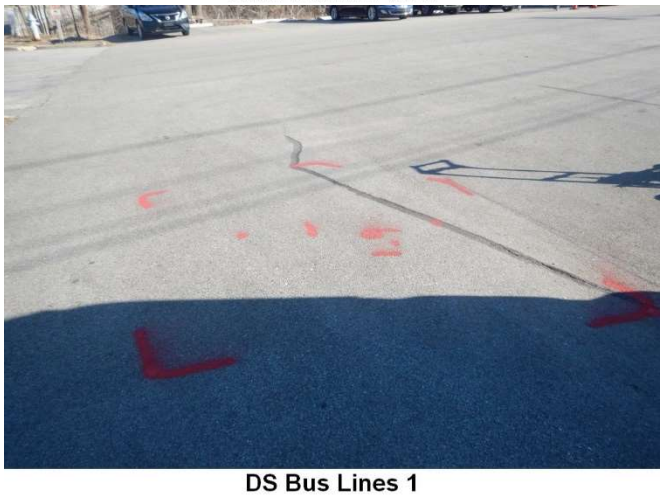


5726 Merriam Dr 8

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



5726 Merriam Dr 9

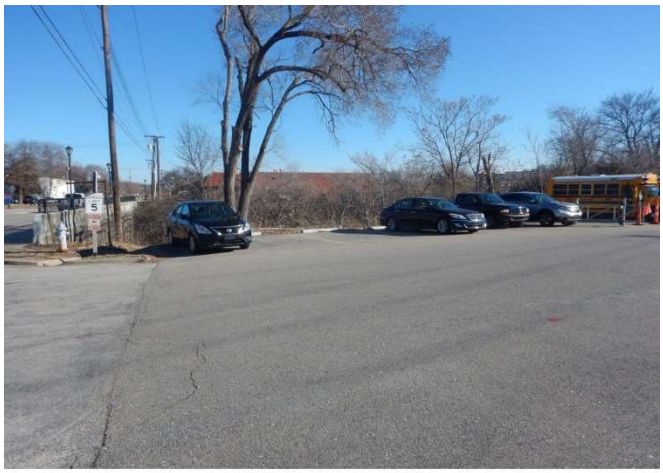


DS Bus Lines 1



DS Bus Lines 2

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



DS Bus Lines 3



DS Bus Lines 4



DS Bus Lines 5

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



DS Bus Lines 6



DS Bus Lines 7



W 61st St 1

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



W 61st St 2



W 61st St 3



W 61st St 4

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



W 61st St 5



W 61st St 6



W 61st St 7

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



W 61st St 8



W 61st St 9



W 61st St 10

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



W 61st St 11

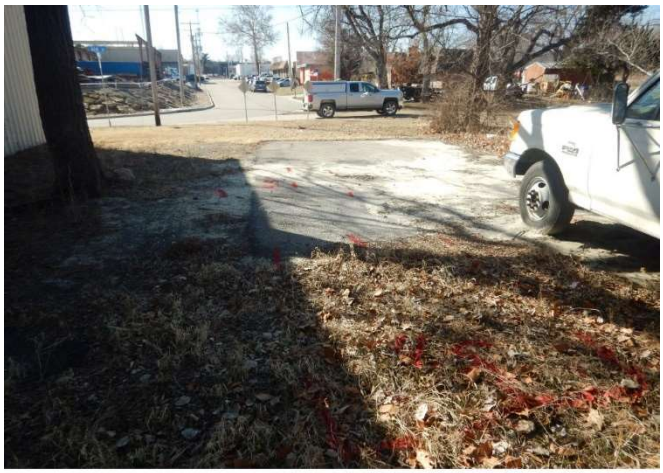


W 61st St 12



W 61st St 13

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



W 61st St 14



W 61st St 15



W 61st St 16

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030



W 61st St 17



W 61st St 18



W 61st St 19

Submitted By:
David Kuxhausen, PLS

Woolpert, Inc.
720 South Colorado Blvd, Suite 1200-S
Glendale, CO 80246
303.524.3030

APPENDIX I – SQRA REPORT

I.1 SQRA Draft Report
I.2 Criteria Waiver

Upper Turkey Creek

Design SQRA



US Army Corps of Engineers®
Dam and Levee Safety Programs

Controlled Unclassified Information (CUI)

Controlled by: Alex Nelson, P.E., USACE Levee Safety Center

CUI Category: General Critical Infrastructure Info

Distribution/Dissemination Control: FEDCON

POC email: alexander.g.nelson@usace.army.mil

US Army Corps of Engineers®
Dam and Levee Safety Program

Upper Turkey Creek

Design SQRA

April 8-10, 2025

Northwest Division

Kansas City District

Upper Turkey Creek FRM

Turkey Creek, KS

Earthen Levee and Floodwall System

Controlled Unclassified Information (CUI)
Controlled by: Alex Nelson, P.E., USACE Levee Safety Center
CUI Category: General Critical Infrastructure Info
Distribution/Dissemination Control: FEDCON
POC email: alexander.g.nelson@usace.army.mil

EXECUTIVE SUMMARY

Project Description

The Upper Turkey Creek Flood Risk Management (FRM) project is located in the City of Merriam, in Johnson County, Kansas. Turkey Creek is a right-bank tributary of the Lower Kansas River. The project consists of levee, floodwall, drainage structure, bridge replacement, and recreation features. The project length is approximately 1.5 miles on both left and right banks of the creek. Levee and floodwall features have a maximum height of approximately 10-14 feet. The non-federal sponsor for this project is the City of Merriam, Kansas. Johnson County, Kansas, contributes to the city's cost share and is involved in the design phase. Part of the Turkey Creek channel in the study area is operated and maintained by the Merriam Drainage District (MDD) under a state charter that provides MDD authority independent of the City of Merriam or Johnson County. MDD may be responsible for operation and maintenance of this project in the future. The project is currently at 35% design, at which point a risk assessment for the purposes of risk-informed design was conducted with the Project Delivery Team, facilitated by the Levee Safety Center.

Risk Characterization

The incremental life safety and economic risk of the Upper Turkey Creek FRM project, for both the left and right bank levee systems, is estimated to be driven by capacity exceedance of the levees and floodwalls resulting in an overtopping breach potential failure mode. The proposed project is in the design phase and the final configuration is still being developed, however, prior-to-overtopping potential failure modes were identified with the current plan and found to be tolerable or sufficiently excluded through the proposed design. Due to the location of the project within an urban area, there is extensive infrastructure along and through the proposed alignment including thousands of feet of sanitary sewer up to 42" in diameter. The team identified a risk associated with the plan to minimize relocations of the sanitary sewer in certain cases and found the risk to be more than one order of magnitude lower than overtopping breach risk. While some cases, such as existing sewer junctions (e.g., "manholes") within the levee footprint and their associated pipes, will need to be relocated, other cases of existing sewer crossing the channel or along the proposed levee toe were found to have low risk and are likely able to remain in place. The risk of internal migration of embankment and foundation material into these pipes is estimated to be low due to the short duration of flood events and the unlikely condition of major defects in the pipes, provided they are inspected prior to approving the pipes to remain in place. The other identified risk of the project is levee global instability if steady-state conditions were to develop, creating high landside seepage pressures, and no mitigating features are included in the design. The team estimated this risk to be low due to the short duration of the flood event making steady-state conditions unlikely and the low likelihood that an instability would progress in a sufficient amount of time and extent to result in a breach of the levee. Intervention for all potential failure modes is likely to be unsuccessful due to the nature of the fast-rising flood event with little warning or preparation time and intense rainfall conditions during the time to take action. Both incremental life loss and incremental economic damages are expected to be low. The short duration of the flood event results in low peak flows and total volumes through potential breach openings. The non-breach risk quickly approaches the total risk of the project for overtopping scenarios due to the small leveed areas filling quickly from overtopping flows. Economic damages are largely driven by damages to the commercial area located within the right bank system. Life safety risk is

primarily present in the residential area in the left bank area at night and in the commercial area on the right bank during the day.

The societal incremental life safety risk estimates with and without intervention are portrayed in Figure ES-1 and summarized in Table ES-1.

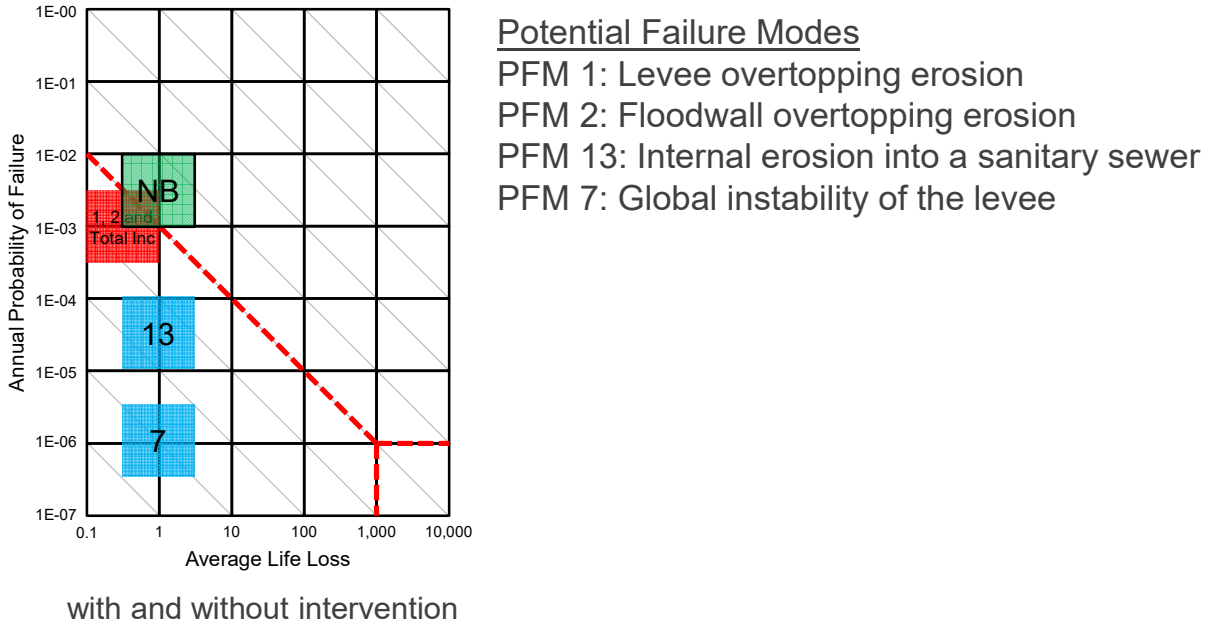
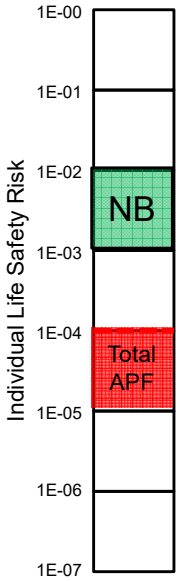


Figure ES-1 Societal Incremental Life Safety Risk Matrix

Table ES-1.
Societal incremental life safety risk summary (with and without intervention).

Potential Failure Mode	Annual Probability of Failure, APF (failures/year)	Average Incremental Life Loss, $\bar{N}$ (lives/failure)	Average Annual Incremental Life Loss, AALL (lives/year)		
PFM 1&2: Overtopping breach	3E-04 to 3E-03	0.1 to 1	3.5E-04		
PFM 13: Internal erosion into a sanitary sewer	1E-05 to 1E-04	0.3 to 3	3E-05		
PFM 7: Global instability of the levee	3E-07 to 3E-06	0.3 to 3	1E-06		
Total			3E-04 to 3E-03	0.1 to 1	3E-04

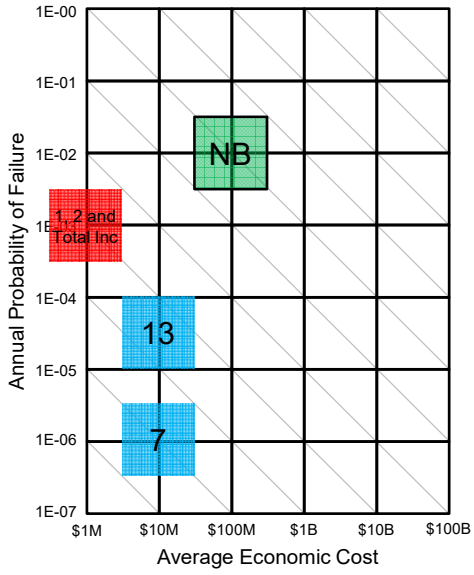
The individual incremental life safety risk estimates with and without intervention are portrayed in Figure ES-2.



With and without intervention

Figure ES-2 Individual Incremental Life Safety Risk Matrices

The incremental economic risk estimates with and without intervention are portrayed in Figure ES-3 and summarized in Table ES-2.



Potential Failure Modes
PFM 1: Levee overtopping erosion
PFM 2: Floodwall overtopping erosion
PFM 13: Internal erosion into a sanitary sewer
PFM 7: Global instability of the levee

With and without intervention

Figure ES-3 Incremental Economic Risk Matrices

Table ES-2.
Incremental economic risk summary (with and without intervention).

Potential Failure Mode	Annual Probability of Failure, APF (failures/year)	Average Incremental Economic Cost, $\bar{\$}$ ($/failure)	Average Annual Incremental Economic Cost, AAEC ($/year)
PFM 1&2: Overtopping breach	3E-04 to 3E-03	$300K to $3M	$1E03
PFM 13: Internal erosion into a sanitary sewer	1E-05 to 1E-04	$3M to $30M	$3E02
PFM 7: Global instability of the levee	3E-07 to 3E-06	$3M to $30M	$1E01
Total	3E-04 to 3E-03	$300K to $3M	$1E03

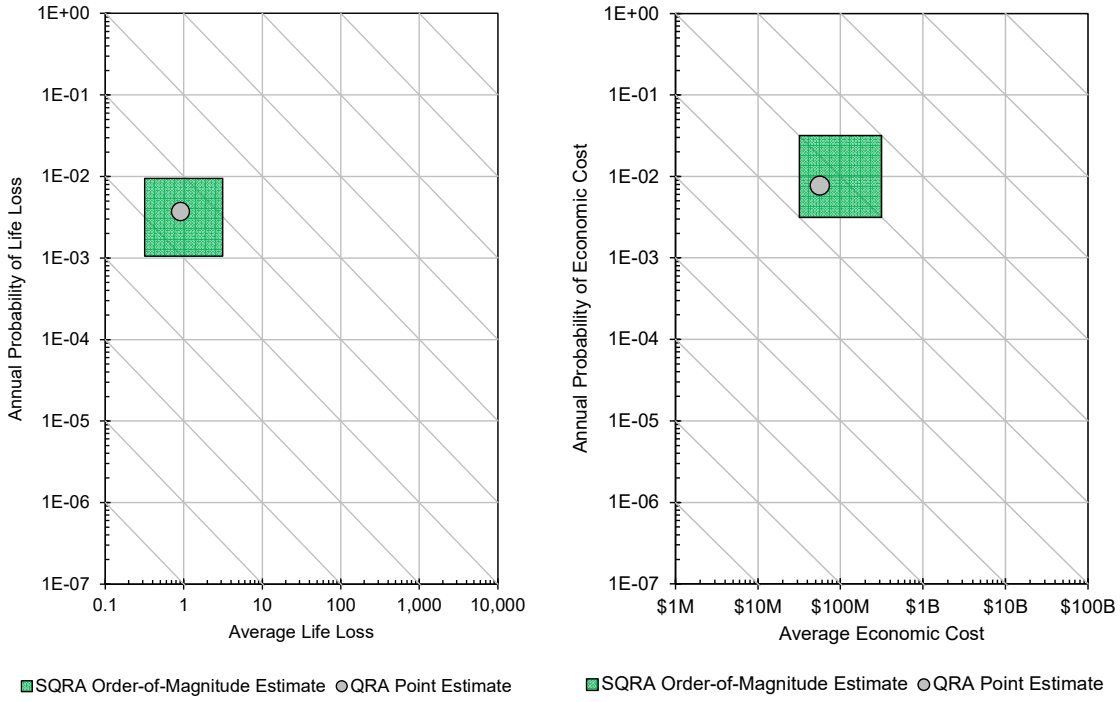


Figure ES-3 Non-Breach Risk Matrices

The annual probability of flooding with non-breach life loss was estimated between 1E-03 and 1e-02 floods per year. The average annual non-breach life loss ($AALL_{NB}$) was estimated to be 3E-03 lives per year. The weighted average non-breach life loss per flood ($\bar{N}_{NB}$) was calculated to be 0.9 lives per flood. Therefore, the estimated average non-breach life loss was estimated between 0.3 and 3 lives per flood.

The average annual non-breach economic cost ($AAEC_{NB}$) was estimated to be $400k per year. The weighted average non-breach economic cost per flood ($\bar{\$}_{NB}$) was calculated to be $56M per flood. Therefore, the average non-breach economic cost was estimated between $30M and $300M per flood.

There is high uncertainty in peak flood levels and durations due to the limited historic data available for the creek. There is uncertainty present in the estimated soil parameters due to the limited data collected to date and the unknown response of the levee fill and foundation material to short duration flood loading. Only minimal information is available about the condition of the sanitary sewer lines, but the uncertainty

should be reduced through televised inspections of the pipes prior to allowing any to remain in place within the zone of influence of the project.

Recommendations

1. Design culvert outfalls with one single passive flap gate closure as the flashy nature of flood events make active closures unlikely and the additional risk reduction of a secondary passive gate is minimal. (PFM 6; TRG 1, USACE).

2. Consider targeted sanitary sewer relocations to only those that pose the highest risk. These include pipes with manholes located in the construction footprint and associated pipes and any pipes that are in a deteriorating condition. (PFM 13; TRG 1&4, USACE).

3. Perform additional stability analysis for the detention basin, including appropriate water levels. (Stability PFMs; TRG 1, USACE).

4. Specify levee fill to use CL or ML and remove other soil classifications that are not fine grained. (All levee PFMs; TRG 4, USACE).

5. Ensure that, if I-walls are included, they have concrete or asphalt surface or armoring with riprap on the landside. (PFM 2; TRG 4, USACE).

6. Ensure all transitions from I-walls to levees have paved surface or armoring to mitigate overtopping erosion at these more critical locations. (PFM 1&2; TRG 4, USACE).

7. Inspect flap gates multiple times a year during routine inspections and perform post flood inspections for every high water event above a threshold to be determined by the PDT. (PFM 6; TRG 3, Sponsor).

8. Perform stability check on a T-wall with landside soil eroded to the base of the footing for top of wall loading. (PFMs 9, 11, 29; TRG 1, USACE).

9. Ensure pipes under the channel and under the levee are televised and show no defects before adopting continued use of the pipes. Ensure inspections are performed every five years and pipes are slip lined upon any sign of defect. (PFM 13; TRG 3, USACE and sponsor).

10. Ensure bridge replacement construction is conducted in a way that does not significantly raise life safety and economic risk to the community. The current plan is that bridge replacements will occur prior to levee and floodwall construction, so the construction specifications should ensure the bridges maintain appropriate conveyance and have emergency flood plans in place. (Existing risk; TRG 4, USACE and contractor).

TABLE OF CONTENTS

1. FINDINGS AND RECOMMENDATIONS 1
1.1. Societal Incremental Life Safety Risk 1
1.2. Individual Incremental Life Safety Risk 3
1.3. Incremental Economic Risk 5
1.4. Non-Breach Life Safety Risk 6
1.5. Non-Breach Economic Risk 8
1.6. Major Findings and Understandings from the Risk Assessment 10
1.7. Recommendations 11
2. BACKGROUND 13
2.1. Location and Purpose 13
2.2. Project Description and Pertinent Data 14
2.3. Levee Embankments 15
2.4. Floodwalls 16
2.5. Relocations 17
2.6. Gravity Drainage 18
2.7. Limestone Wall 19
2.8. Bridge Replacements 20
2.9. Non-Project Features 21
2.10. Site Characterization 22
3. PERIODIC INSPECTION 24
4. HYDROLOGIC HAZARDS 25
5. SEISMIC HAZARDS 73
5.1. Background 73
5.2. Historical Seismicity 75
5.3. Induced Seismicity 77
5.4. Site Classification 79
5.5. Seismic Hazard Curves 81
5.6. Disaggregation 82
6. CONSEQUENCES 84
6.1. Background 84
6.2. Inundation Scenarios 84

6.3. Breach Assumptions 86
6.4. Description of Impacted Area 88
6.5. Life Loss Methodology 91
6.6. Population at Risk 92
6.7. Life Loss Model Inputs and Parameters 94
6.8. Direct Life Loss 96
6.9. Indirect Life Loss 108
6.10. Economic Losses 109
7. RISK ASSESSMENT 111
7.1. Potential Failure Mode Analysis 111
7.2. Risk Assessment 113
7.3. PFM 01: Overtopping breach of the levee 114
7.4. PFM 7: Global instability of the levee 124
7.5. PFM 13: Internal migration into sanitary sewer 136
8. REFERENCES 147

LIST OF FIGURES

Figure 1. Societal incremental life safety risk matrices. 2
Figure 2. Structure used to estimate individual risk. 4
Figure 3. Individual incremental life safety risk matrix. 5
Figure 4. Incremental economic risk matrices. 6
Figure 5. Non-breach life safety risk matrix. 8
Figure 6. Non-breach economic risk matrix. 10
Figure 7. Project location. 14
Figure 8. Project overview. 15
Figure 9. Levee Embankment Typical Section. 16
Figure 10. Floodwall Typical Section. 17
Figure 11. Existing CMP Crossing Limestone Block Wall. 18
Figure 12. Typical Drainage Crossing Through Levee Embankment. 19
Figure 13. Existing Limestone Block Wall. 20
Figure 14. Merriam Drive Bridge. 20
Figure 15. Johnson Drive Bridge. 21
Figure 16. Shawnee Mission Parkway roadway embankment. 22
Figure 17. Proposed leveed area. 28

Figure 18. Turkey Creek StormWatch gages. 29
Figure 19. Example of irregular stage observation intervals at Johnson Dr. at Turkey Creek gage (SW TU04). 30
Figure 20. 10 July 1993 Rainfall hyetograph for Overland Park, Kansas City hall rain gage. 34
Figure 21. Hyetograph of 4 October 1998 Storm Event 35
Figure 22. Turkey Creek Basin Average Hyetograph for 26–27 July 2017. 37
Figure 23. Turkey Creek Basin Average Hyetograph for 5–6 August 2017. 38
Figure 24. Turkey Creek basin average hyetograph for 21–22 August 2017. 40
Figure 25. USGS discharge rating curve (zero gage height = 837.0 ft-NAVD88). 41
Figure 26. Comparison of USGS rating curve and HEC-RAS computed rating curve 42
Figure 27. Bulletin 17C analysis: Peak annual flow record. 44
Figure 28. Bulletin 17C analysis: Peak discharge-frequency curves. 45
Figure 29. Stage-duration curve at Johnson Drive at Turkey Creek gage (StormWatch TU04). 47
Figure 30. Stage-duration curve at Lamar Avenue at Turkey Creek gage (StormWatch TU08). 47
Figure 31. Synthetic flood hydrographs with July 2017 flood hydrograph. 48
Figure 32. Hydraulic model schematic. 49
Figure 33. Flow hydrograph boundary condition locations. 50
Figure 34. Location of incipient overtopping 52
Figure 35. Elevation-discharge relationship and uncertainty at LCL (Channel STA 65+40). 53
Figure 36. Elevation-frequency curve at LCL (Channel STA 65+40). 54
Figure 37. Left-bank flood protection alignment. 57
Figure 38. Right-bank flood protection alignment. 58
Figure 39. Left Bank comprehensive system profile with relevant hydraulic, observed, and system profile data points. 59
Figure 40. Right Bank comprehensive system profile with relevant hydraulic, observed, and system profile data points. 60
Figure 41. Expected overtopping durations at LCL based on routing the scaled synthetic hydrographs in the model. 61
Figure 42. Outflanking of left bank floodwall at end of project. 62
Figure 43. Peak velocity mapping for top of levee loading scenario. 63
Figure 44. Velocity profile plot for loading scenarios. 64
Figure 45. Breach locations. 65
Figure 46. Proposed drainage structure locations. 67
Figure 47. Interior flooding with drainage structure ST-15 flap gate stuck fully open. 68
Figure 48. Interior flooding with drainage structure ST-20 flap gate stuck fully open. 69
Figure 49. Bridge locations. 70

Figure 50. May 2010 Aerial of tributary sediment deposition. 71
Figure 51. June 2023 aerial of tributary sediment deposition. 72
Figure 52. 2018 NSHM declustered seismicity catalogs for $M_w \geq 2.7$ (Petersen et al. 2020) 73
Figure 53. 2018 Generalized seismotectonic zonation of the CEUS (CEUS-SSCn 2012) 74
Figure 54. Quaternary faults (USGS 2025) 75
Figure 55. Historical seismicity near the Upper Turkey Creek Project Area since 1960 (USGS 2025) 76
Figure 56. Injection Wells within 20 KM of the Upper Turkey Creek Project Area (MODNR 2025) 78
Figure 57. Injection Wells within 20 KM of the Upper Turkey Creek Project Area (KGS 2025) 79
Figure 58. USGS Compilation Values of v_{s30} in the United States (McPhillips, D.F., Herrick, J.A., Ahdi, S., Yong, A.K., and Haefner, S., 2020, Updated Compilation of VS30 Data for the United States: U.S. Geological Survey data release, https://doi.org/10.5066/P9H5QEAC). 80
Figure 59. Seismic hazard curve for PGA and Site Class D (USGS 2023) 81
Figure 60. Probabilistic seismic hazard disaggregation for PGA and Site Class D with 1/ 145 AEP (USGS 2023) 82
Figure 61. Probabilistic seismic hazard disaggregation for PGA and Site Class D with 1/ 2475 AEP (USGS 2023) 83
Figure 62. Breach locations. 85
Figure 63. Flood loading scenarios. 86
Figure 64. Sta. RB 56+63 LCL 2-ft OT flood inundation extent at Merriam, Kansas. 89
Figure 65. Sta. RB 56+63 LCL 2-ft OT breach and non-breach inundation at Merriam, Kansas. 90
Figure 66. Sta. RB 56+63 LCL 2-ft OT breach flood depths at Merriam, Kansas. 91
Figure 67. Warning and response timeline. 92
Figure 68. PAI curve for preparedness unknown, perception unknown. 96
Figure 69. Example overtopping location of a 10-foot levee section. 115
Figure 70. Overtopping at a 10-foot-tall levee section. 116
Figure 71. Estimated overtopping velocities for 0.5-ft OT depth at 10-ft levee section. 117
Figure 72. Estimated overtopping velocities for 1-ft OT depth at 10-ft levee section. 118
Figure 73. Estimated overtopping velocities for 2-ft OT depth at 10-ft levee section. 118
Figure 74. System response curve for PFM 01 (with and without intervention). 122
Figure 75. Event tree for PFM 07. 125
Figure 76. Levee design reaches. 126
Figure 77. Typical levee section of Reach 2 LB: 40+80 showing sand drain and stability berm. 127
Figure 78. Hydrograph used for transient seepage in Reach 2 LB Sta 40+80 130
Figure 79. Slope stability analysis at top of levee in Reach 2 LB Sta 40+80 131
Figure 80. Drained strength function with no cohesion. 133
Figure 81. Curvilinear drained strength function with no cohesion 133

Figure 82. Event tree for PFM 13. 137
Figure 83. Sanitary Sewer As-built (1959). 139
Figure 84. Example location of 42” Sanitary Sewer crossing. 140
Figure 85. Failure Mode of PFM 13. 141
Figure 86. Tailwater development at floodwall with 2-foot overtopping scenario. 152
Figure 87. Interior flooding with drainage structure ST-15a flap gate stuck fully open. 155
Figure 88. Interior flooding with drainage structure ST-20a flap gate stuck fully open. 156
Figure 89. Location of boring ADU-208 where a levee will be used. 159
Figure 90. Boring log of ADU-208. 160
Figure 91. Typical levee section with inspection trench. 161
Figure 92. LB Sta 30+30 velocity estimates. 164
Figure 93. RB Sta 11+20 velocity estimates. 165
Figure 94. Photo of a pedestrian bridge in Minot, North Dakota during a flood in 2011 (https://nara.getarchive.net/media/a-pedestrian-bridge-that-fell-into-the-souris-river-c1e0cb?zoom=true) 166
Figure 95. LB Sta 51+00 and RB Sta 10+00 167
Figure 96. Vegetation along the Upper Turkey Creek. 168
Figure 97. Non-project feature flood protection of Shawnee Mission Parkway roadway embankment. 171

LIST OF TABLES

Table 1. Life safety risk-driving potential failure modes. 1
Table 2. Societal incremental life safety risk summary (with and without intervention). 2
Table 3. Incremental economic risk summary (with and without intervention). 6
Table 4. 19
Table 5. Pertinent information for the levee system. 26
Table 6. Historical flood events. 27
Table 7. Pertinent information for neighboring levee systems. 28
Table 8. Systematic record of gage data for the levee system. 30
Table 9. Historical data prior to systematic record. 31
Table 10. Comparison of Rainfall Totals for Major Turkey Creek Floods. 33
Table 11. Precipitation totals for 26–27 July 2017. 36
Table 12. Precipitation totals for 5–6 August 2017. 38
Table 13. Precipitation totals for 21–22 August 2017. 39
Table 14. Hydrology used for design. 43
Table 15. Discharge-frequency statistics. 44

Table 16. Comparison of design hydrology to Bulletin 17C analysis. 46
Table 17. Flow factoring for loading scenarios. 51
Table 18. Summary of standard deviation in stage uncertainty. 53
Table 19. Change in flows between Johnson Drive and LCL location. 54
Table 20. Computed Stage-Frequency Results at LCL. 55
Table 21. Alignment stationing crosswalk. 56
Table 22. Overtopping depth and duration summary at LCL. 61
Table 23. Peak velocity and location for loading scenarios. 64
Table 24. Average and peak change in velocity from top of levee loading for overtopping scenarios. 64
Table 25. Overtopping depths and normal depth velocities at 10-foot levee section. 65
Table 26. Overtopping velocity estimation parameters 66
Table 27. Quaternary faults (USGS). 75
Table 28. Historical seismicity since 15 APR 1960 (USGS). 76
Table 29. Selected historical earthquakes with M4.5+ since 15 APR 1960 (USGS). 77
Table 30. Peak horizontal ground acceleration summary for Site Class D (USGS 2023). 82
Table 31. Breach locations. 84
Table 32. Hydrologic loading event parameters. 86
Table 33. Simplified physical breach parameters for Sta. LB 77+43 BL2. 87
Table 34. Simplified physical breach parameters for Sta. RB 47+08 BL3. 87
Table 35. Final levee breach dimensions. 88
Table 36. Floodwall breach parameters. 88
Table 37. Estimated number of inundated structures. 92
Table 38. Estimated mean population at risk. 93
Table 39. Relative hazard identification times. 95
Table 40. Estimated direct life loss for breach at Sta. RB 56+63 LCL 2-ft OT with minimal warning. 96
Table 41. Estimated direct life loss for breach Sta. RB 56+63 LCL 2-ft OT with ample warning. 97
Table 42. Estimated direct life loss for breach at Sta. RB 56+63 LCL 1-ft OT with minimal warning. 97
Table 43. Estimated direct life loss for breach Sta. RB 56+63 LCL 1-ft OT with ample warning. 97
Table 44. Estimated direct life loss for breach at Sta. RB 56+63 LCL 0.5-ft OT with minimal warning. 98
Table 45. Estimated direct life loss for breach Sta. RB 56+63 LCL 0.5-ft OT with ample warning. 98
Table 46. Estimated direct life loss for breach at Sta. RB 56+63 LCL TOL with minimal warning. 98
Table 47. Estimated direct life loss for breach Sta. RB 56+63 LCL TOL with ample warning. 99
Table 48. Estimated direct life loss for breach at Sta. RB 56+63 LCL 75p with minimal warning. 99
Table 49. Estimated direct life loss for breach Sta. RB 56+63 LCL 75p with ample warning. 99
Table 50. Estimated direct life loss for breach at Sta. RB 56+63 LCL 50p with minimal warning. 100

Table 51. Estimated direct life loss for breach Sta. RB 56+63 LCL 50p with ample warning..... 100
Table 52. Estimated direct life loss for breach at Sta. LB 77+43 BL2 TOL with minimal warning. 100
Table 53. Estimated direct life loss for breach Sta. LB 77+43 BL2 TOL with ample warning..... 101
Table 54. Estimated direct life loss for breach at Sta. LB 77+43 BL2 75p with minimal warning. 101
Table 55. Estimated direct life loss for breach Sta. LB 77+43 BL2 75p with ample warning. 101
Table 56. Estimated direct life loss for breach at Sta. LB 77+43 BL2 50p with minimal warning. 102
Table 57. Estimated direct life loss for breach Sta. LB 77+43 BL2 50p with ample warning. 102
Table 58. Estimated direct life loss for breach at Sta. RB 47+08 BL3 TOL with minimal warning. 102
Table 59. Estimated direct life loss for breach Sta. RB 47+08 BL3 TOL with ample warning. 103
Table 60. Estimated direct life loss for breach at Sta. RB 47+08 BL3 75p with minimal warning..... 103
Table 61. Estimated direct life loss for breach Sta. RB 47+08 BL3 75p with ample warning. 103
Table 62. Estimated direct life loss for breach at Sta. RB 47+08 BL3 50p with minimal warning..... 104
Table 63. Estimated direct life loss for breach Sta. RB 47+08 BL3 50p with ample warning. 104
Table 64. Estimated direct life loss for breach at Sta. RB 25+32 BL4 TOL with minimal warning. 104
Table 65. Estimated direct life loss for breach Sta. RB 25+32 BL4 TOL with ample warning. 105
Table 66. Estimated direct life loss for breach at Sta. RB 25+32 BL4 75p with minimal warning..... 105
Table 67. Estimated direct life loss for breach Sta. RB 25+32 BL4 75p with ample warning. 105
Table 68. Estimated direct life loss for breach at Sta. RB 25+32 BL4 50p with minimal warning..... 106
Table 69. Estimated direct life loss for breach Sta. RB 25+32 BL4 50p with ample warning. 106
Table 70. Estimated direct life loss for breach at Sta. LB 25+20 BL5 TOL with minimal warning. 106
Table 71. Estimated direct life loss for breach Sta. LB 25+20 BL5 TOL with ample warning..... 107
Table 72. Estimated direct life loss for breach at Sta. LB 25+20 BL5 75p with minimal warning. 107
Table 73. Estimated direct life loss for breach Sta. LB 25+20 BL5 75p with ample warning. 107
Table 74. Estimated direct life loss for breach at Sta. LB 25+20 BL5 50p with minimal warning. 108
Table 75. Estimated direct life loss for breach Sta. LB 25+20 BL5 50p with ample warning. 108
Table 76. 109
Table 77. Estimated mean direct property damage. 110
Table 78. PFMA team. 111
Table 79. Potential failure modes. 112
Table 80. Risk-driving potential failure modes..... 113
Table 81. Overtopping velocities and durations at the 10-foot levee section. 116
Table 82. Stage-Frequency at LCL location (Channel STA 65+40) 117
Table 83. Likelihood factors for PFM 1. 119
Table 84. Overtopping system response curve for PFM 01. 121
Table 85. Levee design reaches with stationing..... 126

Table 86. Material properties used in slope stability analyses. 127
Table 87. Slope stability factors of safety for end of construction and steady state seepage conditions. 128
Table 88. Material properties used for transient seepage analyses. 129
Table 89. Results of slope stability analyses with transient seepage. 132
Table 90. Revised steady state stability results. 134
Table 91. Likelihood factors for PFM 07. 134
Table 92: Utility Relocation/Abandonment/Replacement Decision Matrix. 141
Table 93. Likelihood factors for PFM 13. 143

APPENDIXES

APPENDIX A: EXCLUDED POTENTIAL FAILURE MODES 151

APPENDIX B: PERTINENT PLATES AND DRAWINGS 170

APPENDIX C: CALCULATIONS 173

APPENDIX D: REVIEW DOCUMENTATION 174

1. Findings and Recommendations

Kansas City District has completed a Semi-Quantitative Risk Assessment (SQRA) for Upper Turkey Creek FRM Levee design in general accordance with Engineer Circular (EC) 1165-2-218 *USACE Levee Safety Program* (USACE 2021) and Dam and Levee Safety (DLS) Standard Operation Procedure (SOP) DLS-SOP-2023-03 *PFMA and SQRA Methodology* (USACE 2023c), and Appendix A of DLS-SOP-2023-04 *Periodic Assessments* (USACE 2023d). The SQRA consisted of a facilitated Potential Failure Mode Analysis (PFMA), a site visit, and a risk assessment of potential failure modes judged to be risk drivers.

1.1. Societal Incremental Life Safety Risk

A total of 37 potential failure modes (PFM) were identified by the SQRA team for consideration. Table 1 summarizes the risk-driving PFMs that were fully evaluated and are discussed in more detail in Chapter 7. The remaining 33 PFMs were not developed in detail because they were not considered to be risk drivers for the project. They are discussed in Appendix A.

Table 1.
Life safety risk-driving potential failure modes.

Potential Failure Mode	Description
PFM 1	Overtopping breach of the levee
PFM 7	Global instability of the levee
PFM 13	Internal erosion into a sanitary sewer

A risk matrix has been established to portray the incremental life safety risk (due to failure) with annual probability of failure (APF) on the vertical axis and the associated incremental life loss on the horizontal axis. Cell divisions correspond to order-of-magnitude ranges of APF and incremental life loss. The risk matrix is similar to the f-$\overline{N}$ diagram for portraying societal incremental life safety risk for quantitative risk analysis.

An approximate numerical estimate of APF and average incremental life loss was obtained for each PFM using the centroid (geometric mean) of the box (order-of-magnitude estimate). The total APF was calculated by summing the APFs for all primary risk-driving PFMs assuming they are mutually exclusive. The total average annual incremental life loss (AALL) was calculated by summing the product of the APF and average incremental life loss for all primary risk-driving PFMs. The weighted average incremental life loss ($\overline{N}$) was calculated by dividing by the total AALL by the total APF. Calculations are provided in Appendix C.

The societal incremental life safety risk estimates with and without intervention are portrayed i**n Figure 1 and summarized in Table 2.**Figure 1

Figure 1. Societal incremental life safety risk matrices.

Table 2.
Societal incremental life safety risk summary (with and without intervention).

Potential Failure Mode	Annual Probability of Failure, APF (failures/year)	Average Incremental Life Loss, $\bar{N}$ (lives/failure)	Average Annual Incremental Life Loss, AALL (lives/year)
PFM 1&2: Overtopping breach	3E-04 to 3E-03	0.1 to 1	3.5E-04
PFM 13: Internal migration into a sanitary sewer	1E-05 to 1E-04	0.3 to 3	3E-05
PFM 7: Global instability of the levee	3E-07 to 3E-06	0.3 to 3	1E-06
Total	3E-04 to 3E-03	0.1 to 1	3E-04

Additional details about the risk-driving PFMs and associated incremental life loss are provided in the following sections.

1.1.1. PFM 1 & 2: Overtopping breach of the levee and/or floodwall

PFM 1, Overtopping breach of the levee, was estimated to be the highest risk-driver of the system. The project is authorized to convey flood flows up to a 1% annual exceedance probability with 82.9% assurance of not overtopping. The current estimate of the overtopping frequency of the proposed design is a 1/130 AEP, however, additional hydraulic uncertainties are being incorporated into the design which may result in a higher design height and a lower overtopping AEP. The annual probability of failure from overtopping of the levee is estimated to be approximately one order of magnitude lower than the incipient overtopping probability due to the well-vegetated, lean clay, wide embankment that would be exposed to short durations of overtopping flows for extreme events. The flashy nature of the small, steep watershed results in sharp flood peaks and overtopping durations lasting an expected two hours or less for overtopping depths up to two feet. Additionally, the volume and extent of the leveed area is small, which would likely result in tailwater building quickly, which could limit the progression to a full breach and limit incremental consequences of a breach. The probability of unsuccessful intervention, however, is very likely due to the limited time to forecast a flood event and limited time to respond to quickly rising water levels.

PFM 2, Overtopping breach of the floodwall is included in this risk description as the team did not fully estimate the risk, as it was assumed the risk would be equal to or less than the risk of a levee breaching. T-wall floodwall types are expected to remain stable with erosion of the soil material above the footing and it is unlikely that erosion would reach this extent given the short duration. I-wall floodwall types would be more susceptible to instabilities from overtopping erosion, however, the current plan and one of the risk team's recommendations is to provide armoring or paving on the landside of I-wall features to mitigate overtopping erosion. Similarly to levees, unsuccessful intervention is very likely for the same reasons that the probability of failure without intervention is low – that the duration of flood events is extremely short due to the flashy nature of floods in this watershed.

Incremental life loss is expected to be low associated with overtopping breach of a levee or floodwall. Breach modeling indicates generally low peak breach flows and low breach volumes of inundation with no expected life loss associated with a failure up to top of levee. For overtopping depths of 1-2 feet, breach and non-breach conditions could result in life loss, with an estimate of 1 incremental life loss in both cases.

1.1.2. PFM 13: Internal migration into a sanitary sewer

PFM 13, Internal migration into a sanitary sewer, consists of areas where existing sanitary line that is not relocated beneath the zone of influence of the levee could be a location where a defect in the pipe could allow enough material to move through and eventually collapse the embankment. The team assessed different locations of sanitary infrastructure including manholes in and near the alignment and pipes through and parallel to the alignment. The team concurred that any manholes directly in the alignment of the levee or floodwall will need to be relocated, along with the incoming and outgoing pipes associated with the relocated manholes. However, existing pipes that are under the levee or wall or parallel to the levee or wall that are found to be in good condition and will continue to be inspected in the future could likely remain in place. Many pipes are 50 years old, but the pipe materials range from ductile iron pipe (DIP), clay, concrete, and PVC with design life ranging from 75-150 years so they may still be in good condition. Some pipes are larger (up to 42”) which could move a significant amount of material away over time, however, the flashy nature of storms makes it unlikely that enough material could be moved to cause failure in a single event. A good inspection program could ensure that these pipes continue to be in good service and any signs of defects would need to be replaced or relocated in the future. The team judged for any prior to overtopping failure scenarios, the incremental life loss could be around 1, so the team estimated life loss at 0.3 to 3 lives per failure.

1.1.3. PFM 7: Global instability of the levee

PFM 7,

1.2. Individual Incremental Life Safety Risk

Individual risk (IR) is the annual probability of life loss for the most exposed person or group. An estimate of the likelihood of the most exposed person or group losing their life was obtained from LifeSim software output (USACE 2023a). LifeSim computes a mean fatality rate at each structure over all LifeSim iterations, which includes those people that started in each structure and lost their life while evacuating. The structure inventory was reviewed for all flood scenarios, and the maximum of the mean fatality rates of .057 was multiplied by the estimated mean APF to obtain the mean individual risk estimate of 0.00002907 (2.91E-05). Therefore, the annual probability of life loss for the person or group most at risk was estimated between 1E-05 and 1E-04. This estimate is associated with LCL 2ft OT breach at Sta. RB 56+63 and the structure shown in Figure 2. The individual incremental life safety risk estimates with and without intervention are portrayed in Figure 3.



Figure 2. Structure used to estimate individual risk.

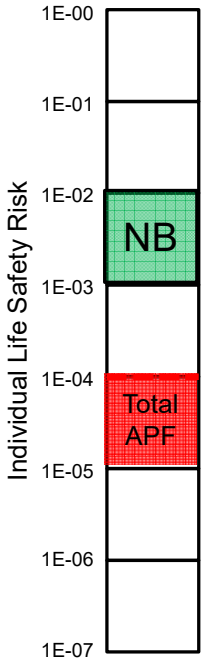


Figure 3. Individual incremental life safety risk matrix.

1.3. Incremental Economic Risk

A risk matrix has been established to portray the incremental economic risk (due to failure) with annual probability of failure (APF) on the vertical axis and the associated incremental economic cost on the horizontal axis. Cell divisions correspond to order-of-magnitude ranges of APF and incremental economic cost. The risk matrix is similar to the f-$\bar{\$}$ diagram for portraying incremental economic risk for quantitative risk analysis.

An approximate numerical estimate of APF and average incremental economic cost was obtained for each PFM using the centroid (geometric mean) of the box (order-of-magnitude estimate). The total APF was calculated by summing the APFs for all primary risk-driving PFMs assuming they are mutually exclusive. The total average annual incremental economic cost (AAEC) was calculated by summing the product of the APF and average incremental economic cost for all primary risk-driving PFMs. The weighted average incremental economic cost ($\bar{\$}$) was calculated by dividing by the total AAEC by the total APF. Calculations are provided in Appendix C.

The incremental economic risk estimates with and without intervention are portrayed in Figure 4 and summarized in Table 3.

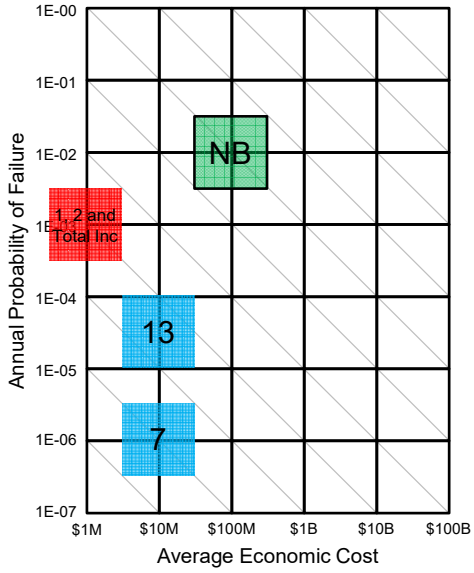


Figure 4. Incremental economic risk matrices.

Table 3.
Incremental economic risk summary (with and without intervention).

Potential Failure Mode	Annual Probability of Failure, APF (failures/year)	Average Incremental Economic Cost, $\bar{\$}$ ($/failure)	Average Annual Incremental Economic Cost, AAEC ($/year)
PFM 1&2: Overtopping breach	3E-04 to 3E-03	$300K to $3M	$1E03
PFM 13: Internal erosion into a sanitary sewer	1E-05 to 1E-04	$3M to $30M	$3E02
PFM 7: Global instability of the levee	3E-07 to 3E-06	$3M to $30M	$1E01
Total	3E-04 to 3E-03	$300K to $3M	$1E03

Additional details about the risk-driving PFMs and their associated incremental economic cost are provided in the following sections.

1.4. Non-Breach Life Safety Risk

Non-breach risk occurs when the flood capacity of the levee is exceeded. At this point, the levee transitions from managing the flood to passing the flood. For levees, the transition occurs at the overtopping stage. This elevation corresponds to the annual probability of non-breach inundation but may not result in life loss. The annual exceedance probability (AEP) when life loss begins to occur ($AEP_{N>0}$) is estimated based on available consequence information and judgment. The annual probability of flooding with non-breach life loss was estimated to be 1/250 AEP based on one foot of overtopping above top of levee (TOL) elevation (AEP_{TOL}). Therefore, the annual probability of flooding with non-breach life loss was estimated between 1E-03 and 1E-02 floods per year.

The non-breach life safety risk was estimated from a quantitative risk analysis (QRA). The stage-frequency curve in Chapter 4 defines a continuous loading relationship between peak water surface elevation and AEP. Z-variate interpolation was used to calculate the AEP for intermediate peak water surface elevations. The overtopping without breach life loss scenarios in Chapter 6 define a continuous relationship between peak water surface elevation and non-breach life loss. Linear interpolation was used to calculate non-breach life loss for intermediate peak water surface elevations. Average non-breach life loss estimates were weighted 0.45 for daytime and 0.55 for nighttime.

The average annual non-breach life loss ($AALL_{NB}$) was estimated to be 0.3 lives per year. The weighted average non-breach life loss per flood ($\overline{N}_{NB}$) was calculated by dividing the $AALL_{NB}$ by the annual probability of flooding with non-breach life loss resulting in 0.9 lives per flood. Therefore, the average non-breach life loss was estimated between 0.3 and 3 lives per flood. Calculations are provided in Appendix C. The results are plotted on a separate non-breach life safety risk matrix, similar to the societal incremental life safety risk matrix previously described. However, the vertical axis is labeled “annual probability of life loss,” and no tolerable risk limit lines are shown since they are not applicable. The non-breach life safety risk is portrayed in Figure 5.

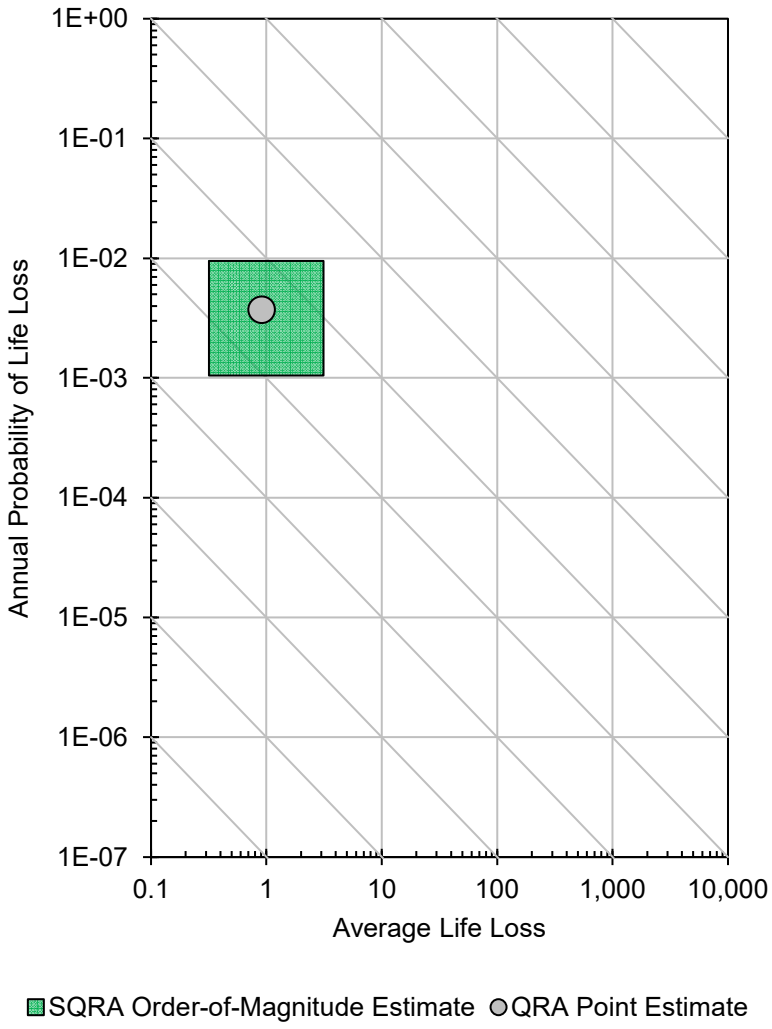


Figure 5. Non-breach life safety risk matrix.

The levee system overtops at an estimated 1/130 AEP. Due to the short duration of overtopping flows, non-breach life loss does not occur until one foot of overtopping depth, approximately 1/250 AEP. Non-breach life loss is estimated to occur for depths of overtopping greater than 1 foot on the right bank during the day in the highly commercial area or on the left bank at night where the leveed area has more residential structures.

1.5. Non-Breach Economic Risk

Non-breach risk occurs when the flood capacity of the levee is exceeded. At this point, the levee transitions from managing the flood to passing the flood. For levees, the transition occurs at the overtopping stage. This elevation corresponds to the annual probability of non-breach inundation but may not result in flood damages. The annual exceedance probability (AEP) when flood damages begin to occur ($AEP_{\$>0}$) is estimated based on available consequence information and judgment. The annual probability of flooding with non-breach economic cost was estimated to be 1/130 AEP based on the top of

levee (TOL) elevation (AEP_{TOL}). The annual probability of flooding with non-breach economic cost was estimated between 3E-03 and 3E-02 floods per year.

The non-breach economic risk was estimated from a quantitative risk analysis (QRA). The stage-frequency curve in Chapter 4 defines a continuous loading relationship between peak water surface elevation and AEP. Z-variate interpolation was used to calculate the AEP for intermediate peak water surface elevations. The overtopping without breach economic cost scenarios in Chapter 6 define a continuous relationship between peak water surface elevation and non-breach economic cost. Linear interpolation was used to calculate non-breach economic cost for intermediate peak water surface elevations

The average annual non-breach economic cost ($AAEC_{NB}$) was estimated to be $400k per year. The weighted average non-breach economic cost per flood ($\bar{\$}_{NB}$) was calculated by dividing the $AAEC_{NB}$ by the annual probability of flooding with non-breach economic cost resulting in $56M per flood. Therefore, the average non-breach economic cost was estimated between $30M and $300M per flood. Calculations are provided in Appendix C. The results are plotted on a separate non-breach economic risk matrix, but the vertical axis is labeled “annual probability of economic cost.” The non-breach economic risk is portrayed in Figure 6.

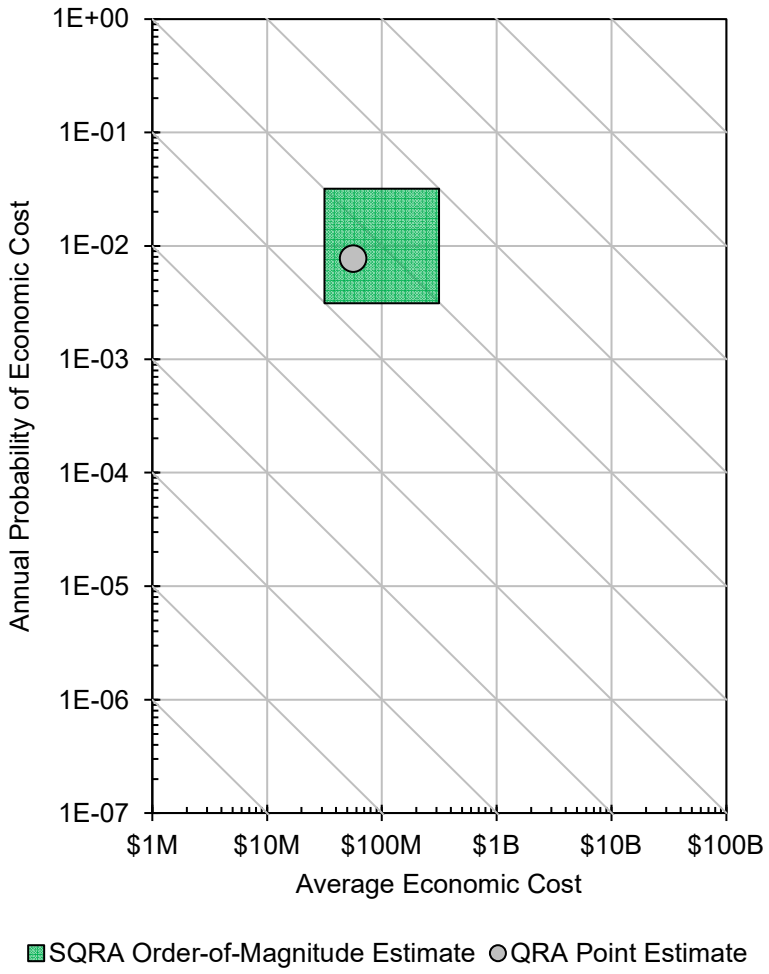


Figure 6. Non-breach economic risk matrix.

Non-breach economic risk is limited by the limited leveed area extent and volume associated with breaching. The primary impacted area is commercial development along the right bank levee.

1.6. Major Findings and Understandings from the Risk Assessment

The following major findings and understandings were developed by the SQRA team:

- The proposed sheet pile for the T-wall does not mitigate intended potential failure mode
- As part of this effort, the design team indicated they will move toward including armoring of transitions to mitigate vulnerabilities to overtopping flows.
- Culverts through the levee of less than 36” diameter are proposed to have a single passive closure, which meets guidance in EM 1110-2-2902.

- The current plan for two passive flap gates for culverts larger than 36” meets the intention of EM 1110-2-2902 to have two means of closure, however, it does not meet the criteria that one of the closures is an active closure. The PDT and risk team concurred that the flashy nature of flooding would not allow the eight culvert locations that are greater than 36” to be actively closed ahead of a flood event.

- There is extensive existing sanitary sewer infrastructure that falls along and near the proposed alignment.

- The flashy nature of flood loading is not expected to approach steady state seepage conditions for all possible flood conditions.

- The levee lies within a small watershed and any creek flooding is associated with heavy rainfall over the entire watershed including the leveed area

- As a new system, it is expected that the entire alignment of both levee systems is expected to overtop at approximately the same frequency.

- Bedrock is not deep – approximately the same elevation as the channel bed for the entire alignment

- Pedestrian bridges remain in place and will likely have flood inundation that exceeds the low chord.

- Life safety risk associated with a breach or overtopping is low based on the current population and structures at risk. If the PAR, development, or zoning changes, this life safety risk could change.

- The channel banks are stabilized with a limestone wall that regularly experiences loss of some of the limestone blocks. The Merriam Drainage District regularly repairs these blocks and works to keep rodents from causing instabilities. There are instances of more permanent repair measures along the wall including concrete placement to stabilize toe scour.

- Hydraulic modeling indicates up to 2 feet of super elevation, which needs to be verified.

1.7. Recommendations

Tolerable Risk Guidelines (TRG) are used for risk-informed decision making within the USACE Dam and Levee Safety Programs. These guidelines are the criteria USACE uses to determine if the risks associated with dams and levee systems are “tolerable.” The four TRG are provided below.

- TRG 1 (Understanding the Risk): Society is willing to live with the risk associated with the dam or levee to secure the benefits provided by the dam or living and working downstream or in the leveed area.

- TRG 2 (Building Risk Awareness): There is a continuation of recognition and communication of the dam or levee risk.

- TRG 3 (Fulfilling Daily Responsibilities): The risks associated with the dam or levee system are being properly monitored and managed by those responsible for managing the risk.

- TRG 4 (Actions to Reduce Risk): There are no cost-effective, socially acceptable, or environmentally acceptable ways to reduce risks.

Recommendations were formulated based on these four TRG to manage and reduce risk. Additional details about the hazard, performance, and consequences can be found elsewhere in this report. The following recommendations were developed by the SQRA team.

11. Design culvert outfalls with one single passive flap gate closure as the flashy nature of flood events make active closures unlikely and the additional risk reduction of a secondary passive gate is minimal. (PFM 6; TRG 1, USACE).

12. Consider targeted sanitary sewer relocations to only those that pose the highest risk. These include pipes with manholes located in the construction footprint and associated pipes and any pipes that are in a deteriorating condition. (PFM 13; TRG 1&4, USACE).

13. Perform additional stability analysis for the detention basin, including appropriate water levels. (Stability PFMs; TRG 1, USACE).

14. Specify levee fill to use CL or ML and remove other soil classifications that are not fine grained. (All levee PFMs; TRG 4, USACE).

15. Ensure that, if I-walls are included, they have concrete or asphalt surface or armoring with riprap on the landside. (PFM 2; TRG 4, USACE).

16. Ensure all transitions from I-walls to levees have paved surface or armoring to mitigate overtopping erosion at these more critical locations. (PFM 1&2; TRG 4, USACE).

17. Inspect flap gates multiple times a year during routine inspections and perform post flood inspections for every high water event above a threshold to be determined by the PDT. (PFM 6; TRG 3, Sponsor).

18. Perform stability check on a T-wall with landside soil eroded to the base of the footing for top of wall loading. (PFMs 9, 11, 29; TRG 1, USACE).

19. Ensure pipes under the channel and under the levee are televised and show no defects before adopting continued use of the pipes. Ensure inspections are performed every five years and pipes are slip lined upon any sign of defect. (PFM 13; TRG 3, USACE and sponsor).

20. Ensure bridge replacement construction is conducted in a way that does not significantly raise life safety and economic risk to the community. The current plan is that bridge replacements will occur prior to levee and floodwall construction, so the construction specifications should ensure the bridges maintain appropriate conveyance and have emergency flood plans in place. (Existing risk; TRG 4, USACE and contractor).

2. Background

2.1. Location and Purpose

The Upper Turkey Creek FRM project is located in the City of Merriam, in Johnson County, Kansas. Turkey Creek is a right-bank tributary of the Lower Kansas River located in Johnson and Wyandotte counties, Kansas, between approximate river miles (RMs) 2.7 to 3.7. Figure 7 shows the location of the Upper Turkey Creek FRM project.

The project was originally authorized for a feasibility study by the Resolution of the Committee on Transportation and Infrastructure, U.S. House of Representatives, Docket 2616, adopted 16 February 2000. Authority for the design and construction of this project was provided in the Water Infrastructure Improvements for the Nation Act (WIIN Act) of 2016, Public Law (PL) 114-322, as recommended in the report of the Chief of Engineers dated 22 December 2015, based on the 15 March 2015 Feasibility Report with Integrated Environmental Assessment. Funding was provided in the amount of $500,000 under the fiscal year (FY) 2022 Omnibus funding-Community Project Funding/Congressionally Directed spending and in the amount of $23.9 million under the Infrastructure Investment and Jobs Act, PL 117-58.

The non-federal sponsor for this project is the City of Merriam, Kansas. Johnson County, Kansas, contributes to the city's cost share and is involved in the design phase. Part of the Turkey Creek channel in the study area is operated and maintained by the Merriam Drainage District (MDD) under a state charter that provides MDD authority independent of the City of Merriam or Johnson County. MDD may be responsible for operation and maintenance of this project in the future.

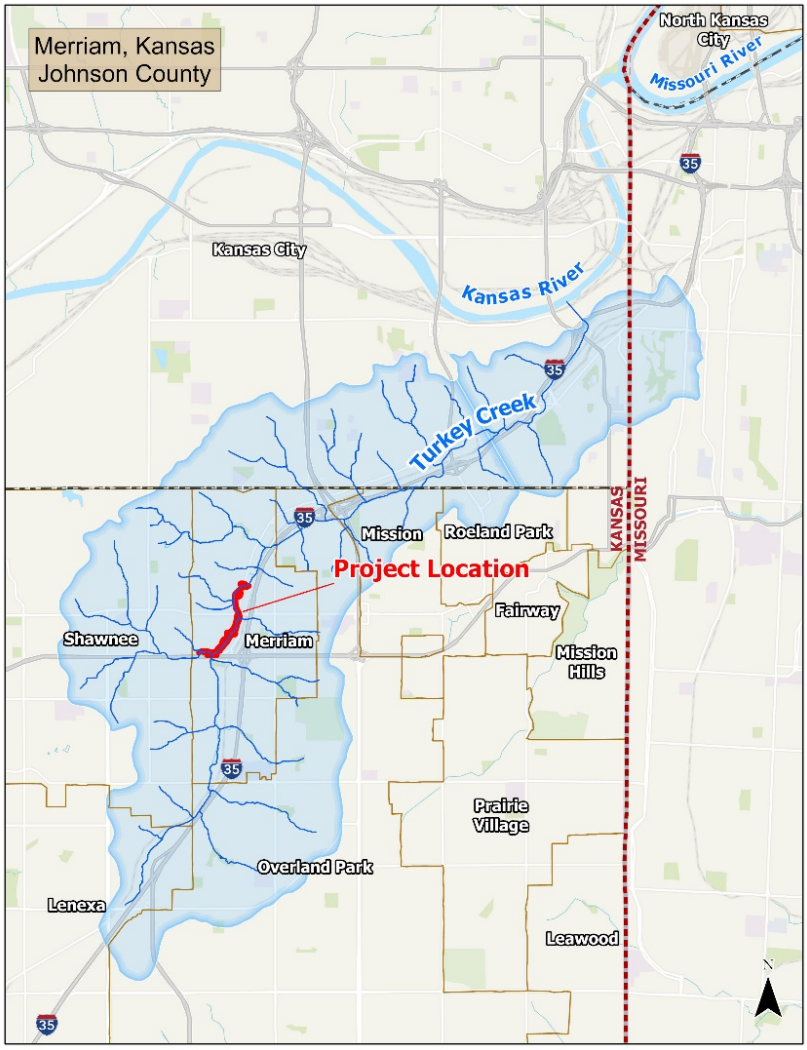


Figure 7. Project location.

2.2. Project Description and Pertinent Data

The main components of the project are shown in Figure 8 and include earthen levee, floodwalls, bridge replacements, detention basin, and drainage structures. The overall project is approximately 1.5 miles long. A majority of the existing limestone block wall will remain, but the floodwall/levee will be set back so the projected toe will not be impacted by the existing wall. The maximum levee height is approximately 10 feet, and the floodwall height ranges from 6 to 14 feet. The levee embankment consists of compacted impervious material.

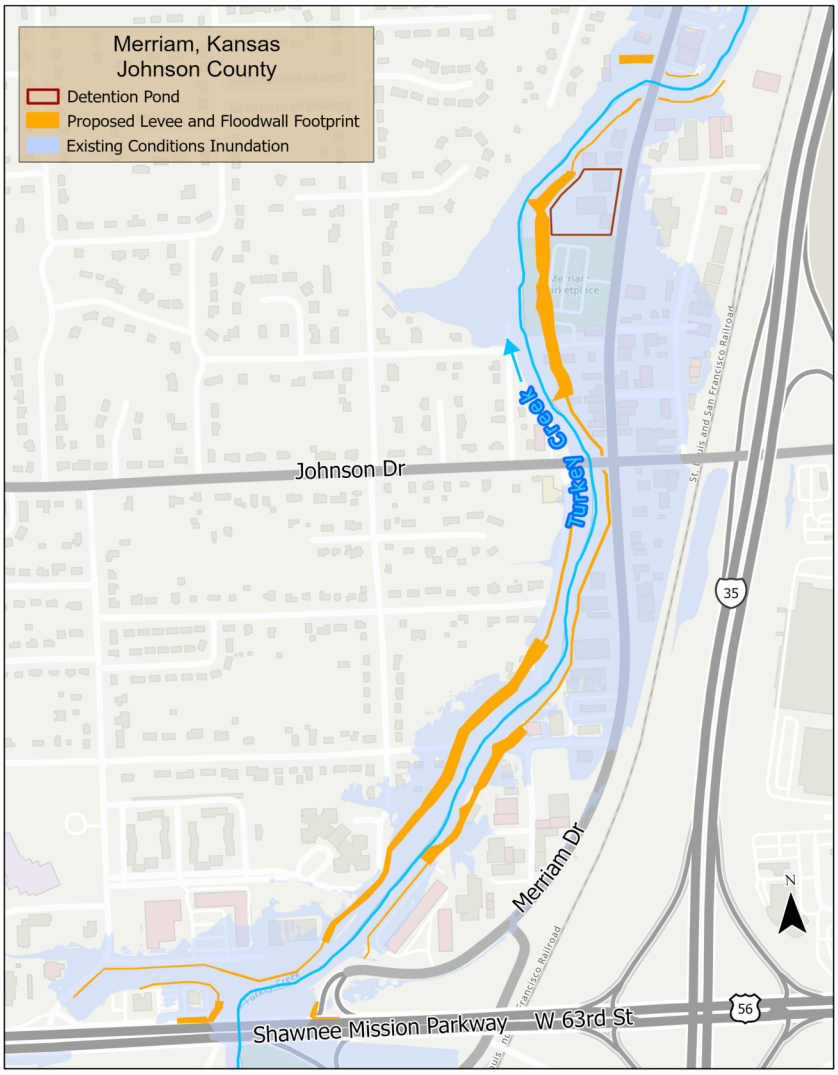


Figure 8. Project overview.

2.3. Levee Embankments

There is roughly 4,000 linear feet of levees with a maximum levee height of approximately 10ft along the left and right banks of this approximately 1.5-mile-long project. The levee embankment consists of compacted impervious material with a 10ft wide crest and 3:1 side slopes. A majority of the existing limestone block wall will remain, but the levee will be set back so the projected toe will not be impacted by the existing wall.

The levee stability design is controlled by drained strength. To meet the required 1.4 factor of safety for steady state stability, some of the levee embankment requires sand drains or stability berms and sand drains. See Figure 9 for a typical levee section showing the stability berm and sand drain. During the SQRA elicitation, the team investigated the risk of not designing to steady state criteria. That discussion can be found in Chapter 7.

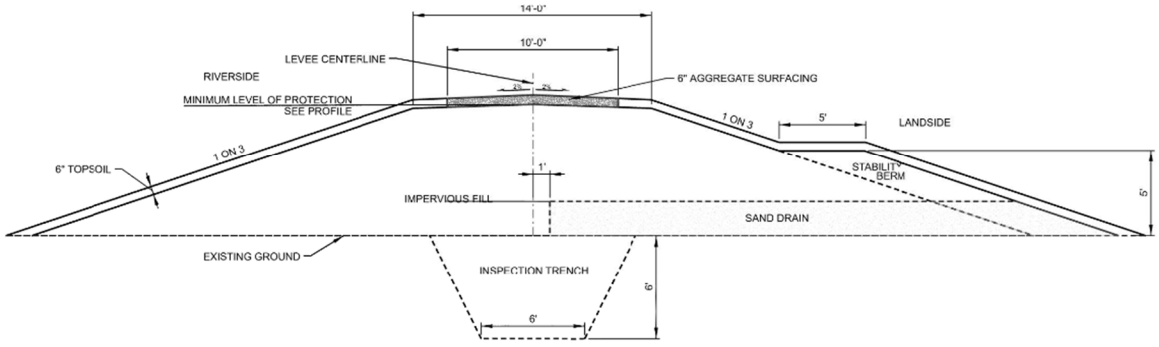


Figure 9. Levee Embankment Typical Section.

2.4. Floodwalls

There is roughly 6,000 linear feet of spread footing floodwalls ranging from 6 feet to 14 feet in height along the left and right banks of this approximately 1.5 mile long project. A majority of the existing limestone block wall will remain, but the levee will be set back so the projected toe will not be impacted by the existing wall.

In areas where floodwalls are required, the original design intent was to use I-walls where feasible, based on their smaller footprint. Based on Table 9.4 in EM 1110-2-2502 (12/2022), for the Unusual load category, using soft clay foundation type, I-walls used as floodwalls should be limited to a maximum wall height of 7 feet.

During initial I-wall analysis, the required embedment depth for rotational stability is greater than depth to bedrock for all configurations, resulting in selection of T-wall instead of I-wall for all floodwall areas. The I-wall design is controlled by drained strength. During the SQRA elicitation, the team investigated the risk of not designing to steady state criteria. That discussion can be found in Appendix A. See Figure 10 for Floodwall Typical Section.

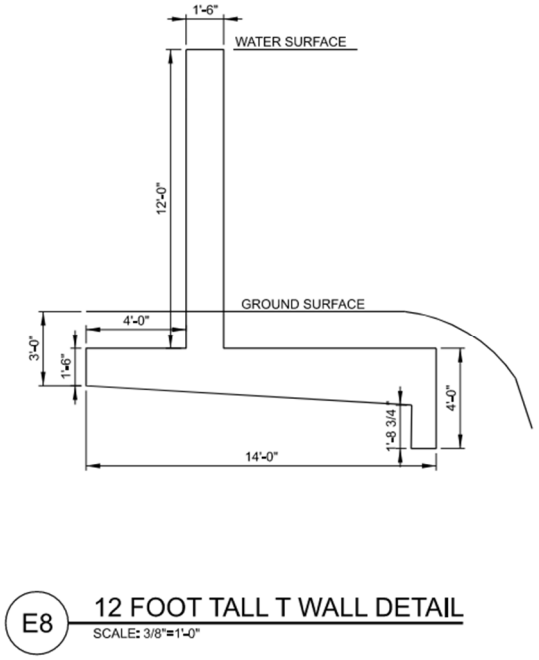


Figure 10. Floodwall Typical Section.

2.5. Relocations

Due to the urban and developed nature of this project location, there are several existing utilities within the project footprint. Existing pipelines crossing the levee that do not meet current USACE criteria will be replaced with pipelines that are compliant. If feasible, all utilities should be located outside of the influence zone. The influence zone at levees is identified as the 15-foot distance from the centerline of the levee on both the riverside and landside. The influence zone at floodwalls is identified as the 15-ft distance from the outside stem wall face or a minimum of 8-ft distance from the footer of the floodwall base, whichever is greater. Existing pipe strengths, unless otherwise known, will be assumed to be that required by USACE criteria at the time of their installation. Pipe condition shall be determined by field assessment.

Gravity lines should be relocated under the crest of the levee raise if it cannot be located outside of the influence zone.

Pressure lines should be relocated over the crest of the levee raise if it cannot be located outside of the influence zone. Pipes crossing over the levee shall have enough cover for local frost protection, one foot minimum. 10:1 slopes on either side of the raised utility to allow for vehicular traffic. Pressure pipes need to be designed and constructed to prevent and control seepage and erosion along the pipe.

Before consideration is given to allowing a pressure pipe to remain through or beneath the levee, the pipe owner will provide an engineering study to support their request. The owner, regardless of the type of pipe, must show adequate capability as to the condition of the existing pipe. An agreement will then be formed between the owner and the levee district to the effect that should repairs to a pipe in the levee become necessary, the pipe will be abandoned, grouted, and relocated over the levee.

Pipes that have been abandoned and do not meet criteria will be removed or grouted accordingly. Pipelines that are currently active but are to be abandoned as part of this project will be removed or grouted full in accordance with criteria.

Utility structures should be located outside of the influence zone if feasible. Structures that are placed inside of the influence zone must be evaluated to mitigate any potential breach. Existing and proposed storm pipes should have a gate box on the riverside. Sanitary pipes should have a manhole located on the landside when feasible. During SQRA elicitation, the team investigated the risk of only removing pipe sections that impact construction and grout-fill remaining lines to abandon within construction limits. That discussion can be found in Chapter 7 and Appendix A.

2.6. Gravity Drainage

There are approximately 27 gravity drainage crossings under the levee/floodwall. The portion of the existing drainage pipe crossing the levee or floodwall will be replaced with reinforced concrete pipe (RCP) and any existing pipes smaller than 24-inch will be upsized to 24-inch with this project. See Figure 11 for an example of an existing drainage crossing.



Figure 11. Existing CMP Crossing Limestone Block Wall.

EM 1110-2-2902, *Conduits Pipes and Culverts Associated with Dams and Levee Systems* lists the closure type requirements for different flood rise conditions. See Figure 12 for a typical drainage crossing. Table 4 summarizes the closure requirements.

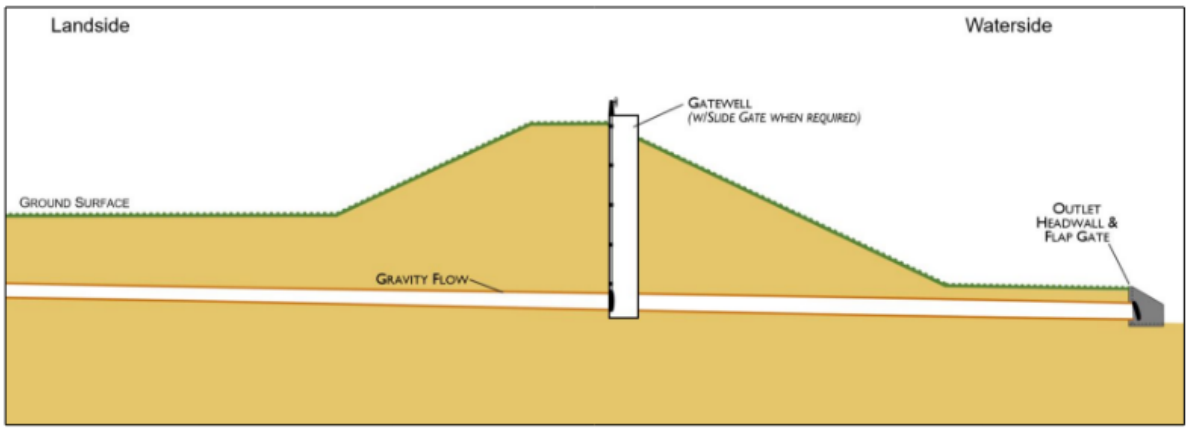


Figure 12. Typical Drainage Crossing Through Levee Embankment.

Table 4.
Closure Type Requirements[1]

Flood Rise[2]	Pipe Diameter (inches)	Closure Configuration
Fast	<36	One Passive
Fast	≥36	One Passive AND One Active
Slow	<36	One Passive OR One Active
Slow	≥36	One Passive AND One Active OR Two Active

[1]Table 9-1 of EM 1110-2-2902
[2]Fast flood rise refers to floods capable of rising to a flood stage with less than 12 hours prediction time. Slow flood rise refers to floods capable of rising to a flood stage with greater than 12 hours prediction time

The flood rise condition for our project is considered fast since there is less than 12 hours of prediction time. The Project Delivery Team (PDT) recommends a deviation from Table 9-1 of EM 1110-2-2902 to use one passive closure at the outlet for all pipes including those greater than or equal to 36 inches. This recommendation affects 7 of the 27 drainage crossings and is based on the short duration of the design storm. There is also a concern that it is not feasible for the Sponsor to manually operate more than 7 sluice gates throughout the project with less than 12 hours prediction time.

2.7. Limestone Wall

The existing limestone block retaining wall is a gravity block wall with no reinforcements tying the blocks together. The individual blocks are approximately 6 feet long, 2 feet deep, and 14 to 18 inches tall. The face of each block in the stack is offset approximately 2 inches back from the face of the block below it. There is no drain behind the wall. The wall was backfilled with local soil which appears to be mainly

clay. In some recent replacement sections of wall, filter fabric and aggregate backfill were placed behind the wall. This limestone block wall is considered a non-engineered structure and was not considered to provide support to the proposed levees or floodwall.



Figure 13. Existing Limestone Block Wall.

2.8. Bridge Replacements

There are two roadway bridges within the project limits that must be removed and replaced. Due to increased water elevations, the existing bridges at Johnson Drive and Merriam Drive do not meet the required uplift or overturning criteria to remain. The new bridge for Merriam Drive is planned tobe a Kansas Department of Transportation (KDOT) slab bridge. The new bridge at Johnson Drive will require further analysis and a new bridge will be selected at the 65 percent phase.



Figure 14. Merriam Drive Bridge.



Figure 15. Johnson Drive Bridge.

2.9. Non-Project Features

The Shawnee Mission Parkway immediately east of Turkey Creek acts as a non-project portion of the flood protection system which prevents inundation that builds up in Chatlain Park from flowing into the proposed levee interior. An upstream view of the embankment is shown below in Figure 16



Figure 16. Shawnee Mission Parkway roadway embankment.

2.10. Site Characterization

2.10.1. Geologic Setting

The Turkey Creek Flood Control Project is in the Osage Plains region of the central lowland physiographic region. The generally rolling to level upland has been eroded by Turkey Creek and associated drainageways to create vertical relief of more than 200 feet between the highlands and lowlands of the Turkey Creek channel. Bedrock is exposed in numerous places along the valley walls and in the creek bottom.

2.10.2. Bedrock Geology

Bedrock is exposed in numerous places along the valley walls and in the creek bottom. Bedrock consists of sedimentary bedrock of the Pennsylvanian Age. The rock units along the creek bottom and the valley walls are alternating shales and limestones of the Bronson and Linn subgroups of the Kansas City Group. The high hilltops outside the study area are part of the Zarah subgroup.

The valley walls are eroded into the formations of the Linn subgroup, which are predominantly shales with several limestones. From the top down, the rock units are the Iola Limestone Formation, the Chanute Shale Formation, the Drum Limestone Formation, and the Cherryvale Formation. The Cherryvale Formation consists of the following members in descending order: Quivira Shale, Westerville Limestone,

Wea Shale, Block Limestone, and Fontana Shale. Of particular note is the Westerville Limestone, which in the upper portion is highly variable in lithology, usually consisting of cross-bedded oolitic limestone but may contain varying amounts of shale. The lower 3 to 4 feet is consistently hard, dense limestone. Vertical faces of the Westerville Limestone in this area may not be stable over the long term and the unit may be water bearing.

The valley floor is cut into the formations of the Bronson Subgroup, which are predominately limestones with several shales.In descending order, the rock units are the Dennis Limestone Formation, the Galesburg Shale Formation, and the Swope Limestone Formation. The Dennis Formation consists of the Winterset Limestone Member and the Stark Shale Member. The Swope Limestone Formation consists of the Bethany Falls Limestone Member, the Hushpuckney Shale Member, and the Middle Creek Limestone Member. The upper several feet of the Bethany Falls Limestone consist of a nodular limestone in a shale matrix and is referred to as the peanut zone.

The intervening Galesburg Shale Formation consists of Canville Limestone (usually absent) and the Galesburg Shale. Differentiating the Galesburg Shale from the overlying Stark Shale is sometimes difficult. The two shales are frequently grouped together as the Stark-Galesburg Shale even though they are from different formations. Review of KDOT profiles for I-35 in the project vicinity indicates a noticeable thickening of this unit to the north, with the appearance of the Canville Limestone.

The present Turkey Creek alignment generally flows on or near the top of the bedrock surface. The exposed bedrock in the channel is either shale or Winterset Limestone in the upper reaches while Bethany Falls Limestone is present in the lower reaches near the tunnel entrance.

2.10.3. Unconsolidated Deposits

The soils in the Turkey Creek flood plain area are mostly alluvial deposits overlying eroded bedrock surface. The surficial soils generally are comprised of unclassified fill overlying lean clay with layers of clayey gravel found at depth near the bedrock surface. The native alluvial silts and clays belong to Onawa-Haynie Association. The hillside soils in the valley are derived from loess and residual soils of the Knox-Ladoga Association.

The native lean clay deposits (CL) are described as medium to stiff and very stiff consistency materials with liquid limits less than 50 percent and plasticity indices varying from about 13 to 29 percent. Natural moisture contents reported in the exploration data vary from 21 to 29 percent.

There are occasional layers of fat clay deposits within these natural deposits. The fat clays have liquid limits on the order of 52 percent.

Groundwater is generally encountered a few feet above the bedrock surface and fluctuates with ground surface elevation. A relatively thin layer (approximately 4 feet) of clayey gravel immediately above the bedrock is a water bearing layer.

Some fills from the channelization of Turkey Creek were randomly placed in the project area. Fills from I-35 rock cuts were placed along the left bank and the construction of a low-level levee and railroad yard on the right bank resulted in fills of unclassified materials. The fills have been in place for many years and appear to have consolidated under their own weight.

Some evidence of dumped debris along the right bank and within the flood plain area exists. This dumped debris is generally unsuitable material for use in engineered fills.

3. Periodic Inspection

The semi-quantitative risk assessment did not include a periodic inspection. Therefore, this chapter is left blank intentionally.

4. Hydrologic Hazards

4.1. Introduction

USACE uses a risk-informed approach where risk assessments and deterministic design standards provide more information for decision makers. The primary objectives of the hydrologic hazard analysis for levees to support a semi-quantitative risk assessment (SQRA) are:

- Characterize the dominant flood hazard(s) for the watershed and system and document/describe the history of past flood events.
- Estimate the discharge-frequency relationship and maximum flood events, considering effects of any upstream regulation.
- Characterize the typical duration of flood events and the annualized water surface elevation-duration relationship.
- Estimate the water surface elevation-discharge relationships and uncertainties across the entire levee system through hydraulic modeling, including the effects of coincident flooding sources.
- Provide water surface elevation-frequency relationship at numerous index locations throughout the system and through development of system water surface profiles along the levee.
- Summarize risks related to overtopping flows and outflanking of the levee system and provide preliminary expected annual exceedance probability of overtopping inundation.
- Document other known hydrologic and hydraulic concerns and uncertainties for the levee system such as channel or bank erosion, sedimentation, stage impacts, wind-wave or wake impacts, ice and debris, known interior drainage issues, or water management operational changes.

4.2. Background Data

Table 5 provides pertinent information for the current effective design of the levee system. The SQRA is taking place during the design of Upper Turkey Creek levee. Authorization is for the design 0.01 AEP (1/100-yr) event with an assurance of 82.9%. The levee height has been adjusted to meet the 85% assurance to meet USACE certification standards given negligible height increases are needed. The updated levee design passes the 0.01 AEP design flow with an assurance of 85%, surpassing the authorized assurance level. There is desire to modify the levee profile to have a designed overtopping location that would be both hydraulically advantageous and of low consequences. This is one of the risk informed design decisions that are to be discussed during the risk assessment.

The flood of record on Turkey Creek occurred on October 4th, 1998. At this time, there were no active flow recording gages. Peak flow was estimate using a hydraulic model based on surveyed high-water marks as part of the USACE 2003 General Reevaluation Report (GRR).

Table 5.
Pertinent information for the levee system.

Description	Value
Authorization	Water Infrastructure Improvements for the Nation Act (WIIN Act) of 2016, Public Law 114-322
Watercourse(s)	Turkey Creek (RM 7.3 to 6.1)[1]
Watershed	Turkey Creek Basin
Drainage Area[3]	12.3 square miles
Design Top of Levee Elevation(s)	940.1 to 910.1 ft-NAVD88
Minimum Controlling Elevation (Levee Control Location)	910.1 ft-NAVD88
Flood of Record [2]	792.04 ft-NAVD88 at Turkey Creek at Kansas City, KS gage (USGS 06892940), 23,800 cfs, (04 October 1998)
Design Basis	1/100 AEP at 85% confidence limit

[1]River miles above Turkey Creek tunnel based on FEMA FIS Reports.
[2]Flood of record was estimated using hydraulic modeling and highwater marks. Not observed gage flow
[3]Upstream Drainage area from downstream extent of project

The Turkey Creek at Kansas City, KS (USGS 06892940) gage conversion between the North American Vertical Datum of 1988 (NAVD88) and National Geodetic Vertical Datum of 1929 (NGVD29) is as follows:

$$\text{NAVD88} = \text{NGVD29} + 0.32 \text{ ft} \quad (1)$$

Table 6 summarizes historical flood events at Turkey Creek at Kansas City, KS (Southwest Blvd). This gage is downstream of the project extents. This gage was used as a common reference point to report historic Turkey Creek flooding. Documentation reported estimated flood magnitudes at various locations along Turkey Creek, so all flows were scaled to Turkey Creek at Kansas City, KS using a basin area ratio.

Table 6. Historical flood events.

Date	Peak Discharge (cfs)	Peak Water Surface Elevation (ft-NAVD88) at Turkey Creek at Kansas City, KS Gage (USGS 06892940)	Peak Gage Height (ft)[1] at Turkey Creek at Kansas City, KS Gage (USGS 06892940)	Source Location (Basin Area Ratio)
04 October 1998	23,800 cfs[3]	792.36[2]	29.94	Southwest Blvd.
21-22 April 1944	15,900 cfs[3]	-	-	Johnson Dr. (1.41)
10 July 1993	14,270 cfs[3]	788.42[2]	26.0	Southwest Blvd.
01 June 1935	14,000 cfs[3]	-	-	Southwest Blvd.
10 October 1986	13,770 cfs	788.32[2]	25.9	Southwest Blvd.
05-06 August 2017	13,400 cfs	780.19[2]	17.77	Southwest Blvd.
01 May 1983	12,500 cfs	788.22	25.80	Southwest Blvd.
12 September 1977	11,700 cfs	787.62	25.20	Southwest Blvd.

NOTE: [1] Zero gage height = 762.42 ft-NAVD88
[2]Surveyed high water mark
[3]Estimated flow

The Upper Turkey Creek levee project is located on Turkey Creek from approximately river mile 7.3 to 6.1 in Johnson County, Kansas. The levee is located along Merriam, Kansas approximately 8 miles southwest of the Kansas City district office in downtown Kansas City, Missouri. Figure 7 provides an overview of the project location, and Figure 17 provides an overview of the leveed area. The levee protection project is currently in design as of April 2025. The levee is being designed by USACE. It is not yet determined who the sponsor for the levee will be, but it will likely be the city of Merriam or the Merriam Drainage District. The drainage area above the levee system is approximately 12.3 square miles and is characterized as heavily urbanized. USGS StreamStats (https://streamstats.usgs.gov/ss/) characterizes the upstream drainage area as 100% developed and 43.2% impervious according to the 2011 NLCD layer. Flood events are influenced by intense local storm events that lead to large volumes of surface runoff mainly during the spring and summer seasons but can occur in any month with sufficient rainfall. Turkey Creek flooding is not significantly driven by antecedent conditions and can occur from one single storm event with no prior rainfall.

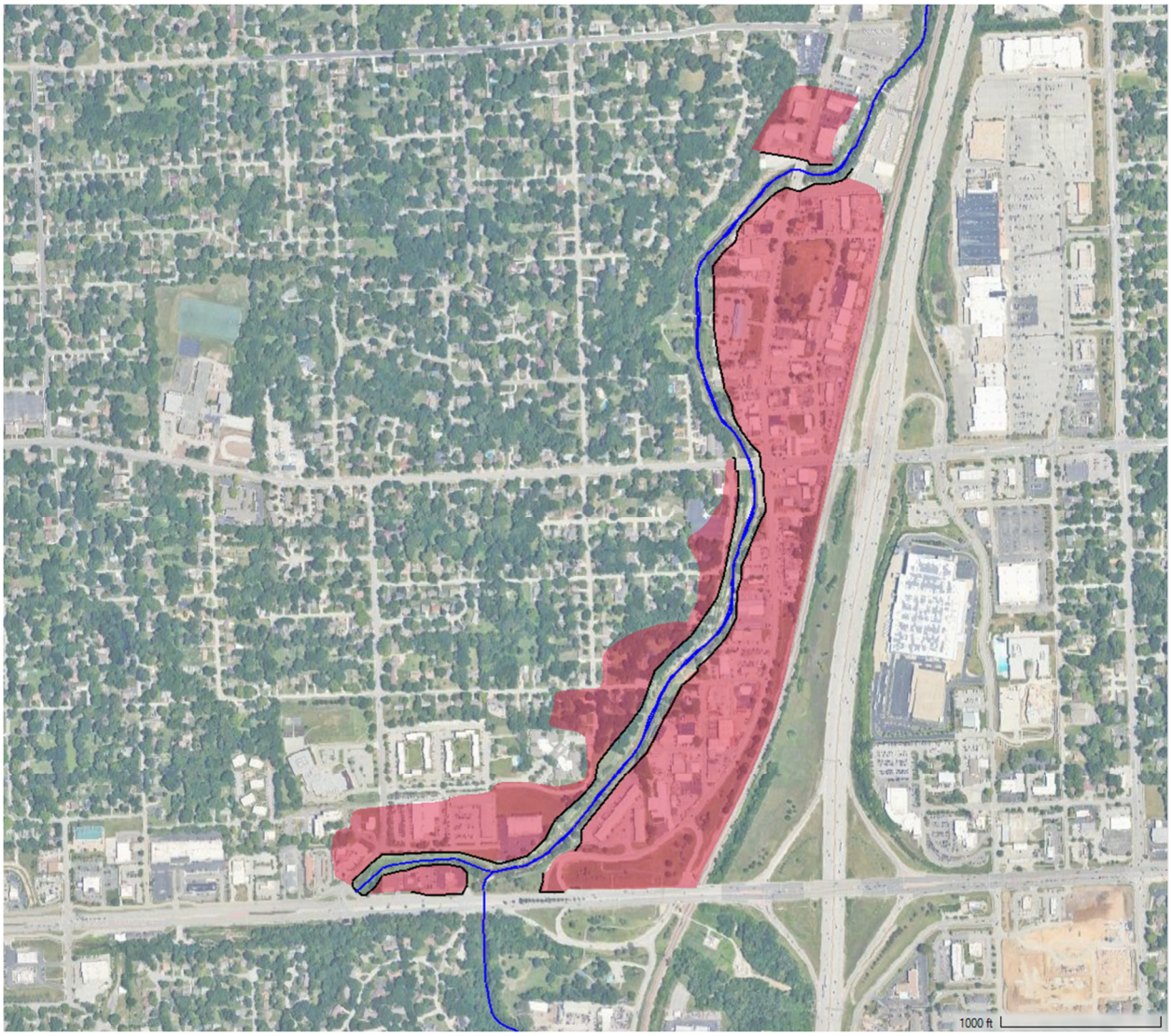


Figure 17. Proposed leveed area.

Table 7 provides pertinent information for the two Lower Turkey Creek levee projects which were approximately 2-3 miles downstream of the Upper Turkey Creek project area. There are no other levee systems along the Turkey Creek watershed.

Table 7. Pertinent information for neighboring levee systems.

Description	Value	
Levee System Name	Turkey Creek Left Bank	Turkey Creek Right Bank
Owner/Sponsor	Unified Government of Wyandotte County and Kansas City, KS	Unified Government of Wyandotte County and Kansas City, KS
River Mile (RM)*	RM 1.8 to 1.3	RM 0.8 to 0.3

Description	Value	
Annual Probability of Inundation	0.01 AEP	0.01 AEP
Authorization Date	April 1999	April 1999
Construction Completion Date	March 2016	November 2011

*Note: River miles above Turkey Creek tunnel based on FEMA FIS Reports

4.2.1. Data Sources

There is a notable lack of continuous and consistent period of record data for gages within the Turkey Creek watershed. Majority of gages along Turkey Creek are operated by StormWatch (SW) (https://www.stormwatch.com), a county wide flood warning system managed mostly by Johnson County, KS but extends to the larger Kansas City metro as well. Stormwatch has one gage located within the project extents, Johnson Drive at Turkey Creek (SW TU04). This gage along with the rest of the storm gages only record stage. Figure 18 provides locations of the StormWatch gages along Turkey Creek. All gages record both river stage and rainfall, with the exception of TU09, which records rainfall only.

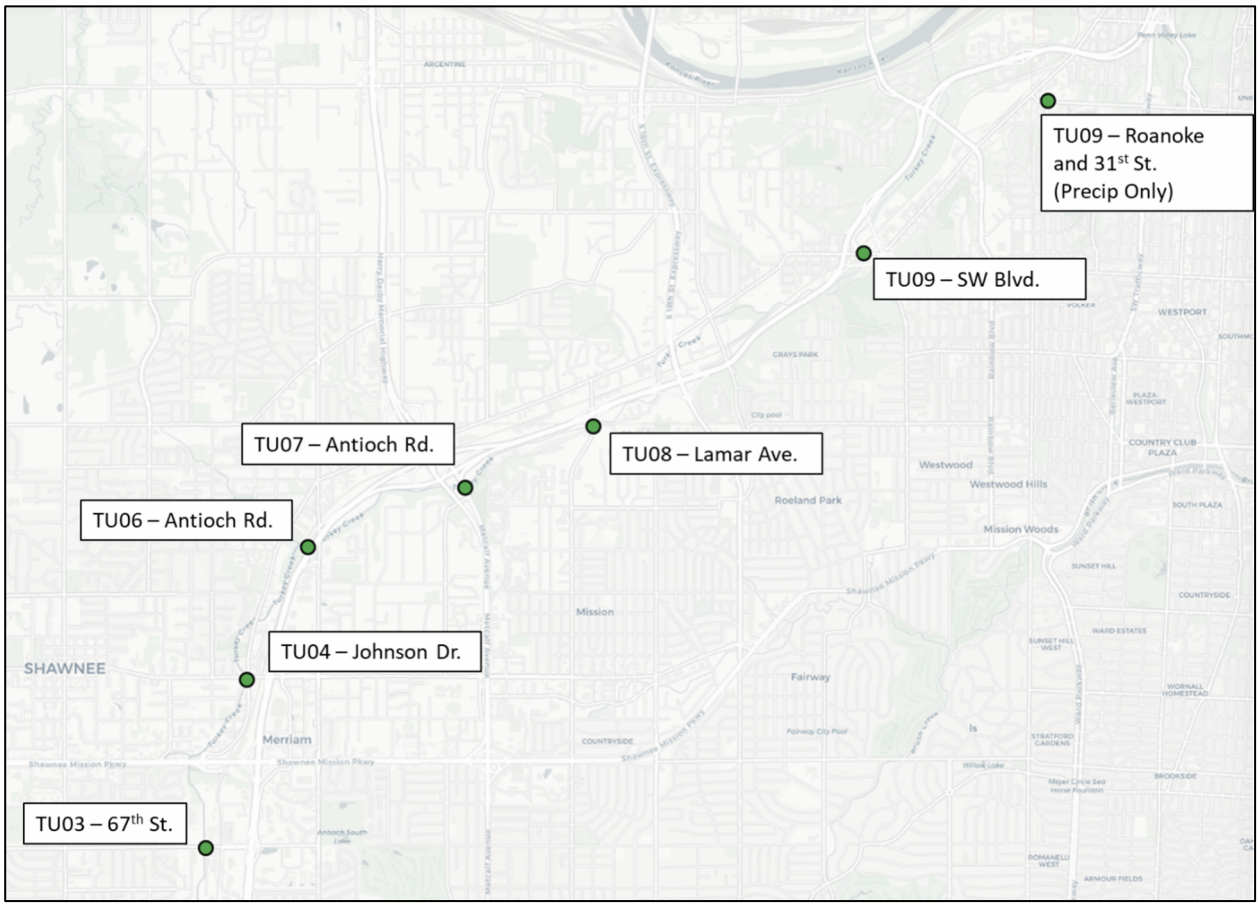


Figure 18. Turkey Creek StormWatch gages.

Johnson Drive at Turkey Creek (SW TU04) has a stage period of record from 1998 to current day. The StormWatch gages seemingly have the capability to record instantaneous data, even sub-15-minute increments, but appear to only do so during times of flow increases. An example of this is shown below in Figure 19. During times of typical stages, the gage records stage at 12-hour increments, but during increases in stage, even well below the 14-feet monitor stage (yellow line), observation frequency picks up to as low as 3-minutes in this example. This leads to fairly irregular observation increments and irregular observation times. The 12-hour readings prior to the rise in stage occurred at 10:36:18 am/pm and then 10:37:18 am/pm after the peak. This varies significantly over the period of record.

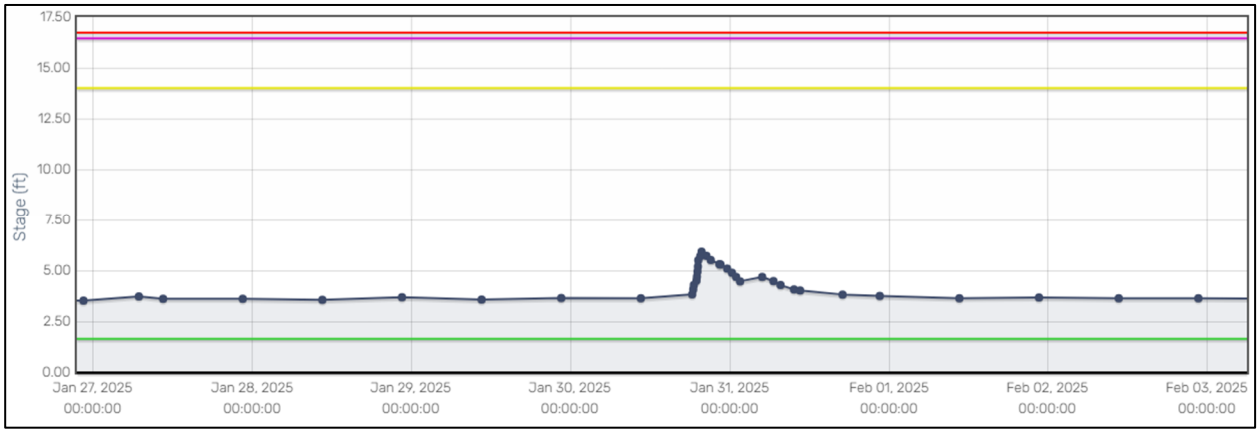


Figure 19. Example of irregular stage observation intervals at Johnson Dr. at Turkey Creek gage (SW TU04).

Period of record data previously analyzed and used for this SQRA included peak annual stage and flow data at the, now inactive, Turkey Creek at Kansas City, KS gage (USGS 06892940) from 1974 to 1987. Additionally, field measurements were taken from 1987 to 1989 at this gage. This gage is located approximately 4.7 river miles downstream of the project area.

Period of record data is also available at the Turkey Creek at Merriam, KS gage (USGS 06892800), including annual peak data from 1974 to 1986 and 2006 to current. There are also field measurements from 2003 to 2018. This gage location does not record daily or instantaneous data.

Period of record data is also available from the Turkey Creek at Mission, KS gage (USGS 06892900). This gage records stage only and only began instantaneous observations in August of 2022. Annual peak data is not published at this gage as of now. This gage is located about 2.5 river miles downstream of the project and is within the model extents.

Table 8. Systematic record of gage data for the levee system.

Gage	Daily Water Surface Elevation (ft-NAVD88)	Daily Discharge (cfs)	Peak Annual Elevation (ft-NAVD88) and Flow (cfs)	Field Measurements Elevation (ft-NAVD88) and Flow (cfs)	Instantaneous (15-min, hourly) Elevation (ft-NAVD88) and Flow (cfs)
Johnson Dr at Turkey Creek gage (Stormwatch TU04)[1]	N/A	N/A	N/A	N/A	1998-Current[2]

Gage	Daily Water Surface Elevation (ft-NAVD88)	Daily Discharge (cfs)	Peak Annual Elevation (ft-NAVD88) and Flow (cfs)	Field Measurements Elevation (ft-NAVD88) and Flow (cfs)	Instantaneous (15-min, hourly) Elevation (ft-NAVD88) and Flow (cfs)
Lamar Ave at Turkey Creek gage (Stormwatch TU04)[1]	N/A	N/A	N/A	N/A	1998-Current[2]
Turkey Creek at Merriam, KS (USGS 06892800)	N/A	N/A	1974-1986, 2006-Current	2003-2018	N/A
Turkey Creek at Mission, KS (USGS 06892900)[1]	N/A	N/A	N/A	Aug2022-Current	Aug2022-Current
Turkey Creek at Kansas City, KS (USGS 06892940)	N/A	N/A	1974-1987	1987-1989	N/A

[1]Stage gage only. No flow data is recorded
[2] Instantaneous data is irregular and inconsistent over the period of record

Record stages on Turkey Creek were produced prior to most StormWatch gages coming online and outside of the period of record of Turkey Creek USGS gages. It should be noted that the below list of historic data was used for developing a flow frequency curve as a check to the hydrology being used for design. Additionally, many of these flows are rough estimates from historic documentation or from RAS modeling. Some have had a basin area ratio applied to shift all historic flows to the Southwest Blvd gage location for the purposes of a semi-comprehensive input for Bulletin 17C. Floods that were described in documentation as having *minimal* ponding at State Line were given a best estimate flow of 9,000 cfs based on HEC-RAS rating curves developed during a previous Lower Turkey Creek study. Floods with *some* ponding at State Line were given a best estimate flow of 10,000 cfs.

Table 9. Historical data prior to systematic record.

Location	Type	Year(s)	Magnitude (cfs)	Citation
Turkey Creek at Kansas City, KS (USGS 06892940)	Perception Threshold	1903-1935	14,000[1]	USACE 1962
Turkey Creek at Kansas City, KS (USGS 06892940)	Perception Threshold	1935-1973	8,000	Estimated Channel Overflow
Turkey Creek at Kansas City, KS (USGS 06892940)	Perception Threshold	1988-1997	8,000	Estimated Channel Overflow
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1935	14,000[1]	USACE 1962
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1944	15,900[1]	USACE 1962 – Economics Analysis

Location	Type	Year(s)	Magnitude (cfs)	Citation
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1958	10,000[2]	USACE 1962 – Economics Analysis
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1961	10,000[2]	USACE 1962 – Economics Analysis
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1968	9,000[2]	USACE 1998 – LTC Feasibility Study
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1993	14,270	USACE 1998 – LTC Feasibility Study
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1995	9,000[2]	USACE 1998 – LTC Feasibility Study
Turkey Creek at Kansas City, KS (USGS 06892940)	Historical Range	1996	9,000[2]	USACE 1998 – LTC Feasibility Study

[1]Basin area ratio applied to shift flow estimate to Southwest Blvd

[2]Flow estimated based on description of event.

4.2.2. Historic Floods

Major floods on Turkey Creek are usually caused by short-duration, high-intensity storms following an extended period of general rainfall over the basin. The rainfall preceding the heavy rains saturates the soil, preventing further infiltration and producing runoff into the Turkey Creek channel, creating higher than average flow. Since the Turkey Creek channel is so narrow and urbanized, it is usually subject to flash flood events, which are difficult to forecast and prepare for. Earlier flood events on Turkey Creek are known to have happened in 1844, 1935, 1944, 1958, 1961, and 1968. Little information was documented for these specific events. Recent flood events that have available records and documentation include 1977, 1993, 1995, 1996, 1998, and 2017. However, the focus will be on the four of the largest flood events, which were 1977, 1993, 1998, and 2017. The 1977, 1993, and 1998 floods' rainfall data were recorded by the same rain gage at Shawnee, Kansas. Precipitation for the 2017 events were determined using both Johnson County StormWatch rain gage data and National Weather Service (NWS) radar observations. A basin average is provided for 21–22 August 2017. Table 10 summarizes the observed precipitation events. Subsections 4.2.2.1 through 4.2.2.8 discuss details pertaining to historic flood events.

Table 10. Comparison of Rainfall Totals for Major Turkey Creek Floods

Flood Event Date	Peak Hour Rainfall (inches)	Total Rainfall (inches)	Recorded Dates
1977	1.60	10.90	12–13 September 1977
1993	1.50	6.00	10 July 1993
1998	3.51	5.61	04 October 1998
2017	1.05	5.71	21–22 August 2017

4.2.2.1. June 1935

The greatest known flood at Merriam, KS at the time occurred in June 1935 with an estimated Turkey Creek discharge of 13,650 cfs. It should be noted that a specific location of estimated flow was not given in the historic documentation, but it is assumed that flow was estimated at Johnson Dr. in Merriam, KS. This flood inundated the city area to depths of 7 feet above what was considered flood stage at the time. Development in the area was not extensive in 1935 and no estimated damage caused by this flood is available.

4.2.2.2. April 1944

The April 1944 flood at Merriam, KS had an estimated Turkey Creek discharge of 11,300 cfs. It inundated the city to depths of about 6 feet above what was considered flood stage at the time. 58 residences and businesses were affected, and the damage was estimated at $118,700 in 1944 price levels.

4.2.2.3. July 1958

In July 1958, a flood with an estimated discharge of 4,400 cfs inundated a total of 69 residences and business establishments and resulted in damages estimated at $155,000 in 1958 price levels.

4.2.2.4. September 1961

The September 1961 flood at Merriam had an estimated discharge of 4,700 cfs and inundated a total of 82 residences and business establishments. An exhaustive damage survey, conducted shortly after the flood, established a value of $257,000 (1961 price levels) in damages in the city, of which $240,000 occurred along the main stem of Turkey Creek. This survey included all flood damages within the corporate city limits. The previous flood damage estimates may have included a smaller area, but the primary reason for the increasing damage values is the rapid development of the flood plain at the time.

4.2.2.5. September 1977

Locally known as the Plaza Flood, this was a flash flood event that occurred 12–13 September 1977, throughout the Kansas City metropolitan area. It was caused by a high intensity storm following a long period of light rain. Approximately 16 inches of rain fell over the Kansas City, Missouri, area over the course of the event.

Most of the focus was on Brush Creek, which travels through the Plaza and discharges into the Blue River. During this event, there were 25 fatalities and 1,000 automobiles totaled, along with millions of dollars in damages to the commercial and residential areas surrounding Brush Creek.

In the Turkey Creek area, $8.1 million (1977 price levels) in damages occurred from flooding. Although damages were extensive, the urban study conducted after the flood determined implementing a program for managing flood events along Turkey Creek was not feasible.

4.2.2.6. July 1993

Extensive flooding occurred on 10 July 1993. Although this year was also a major flood year for the Missouri and Kansas Rivers in Kansas City, this was an unrelated flash flood event caused by high intensity storms on that date. When Turkey Creek flooded in 1993, it overtopped I-35 at five different locations. The flooding occurring during this event caused one fatality. Damages from the 1993 event were estimate at $3.4 million in Merriam, Kansas, and $20 million in the lower basin in 1993 price levels. A rainfall gage located at Overland Park City Hall (approximately 3.3 miles south of Merriam, Kansas) recorded the July 1993 storm event. Figure 20 provides a rainfall hyetograph of the event. According to this observed data, approximately 5.8 inches of rain fell over the upper portion of the Turkey Creek watershed within a 12-hour period.

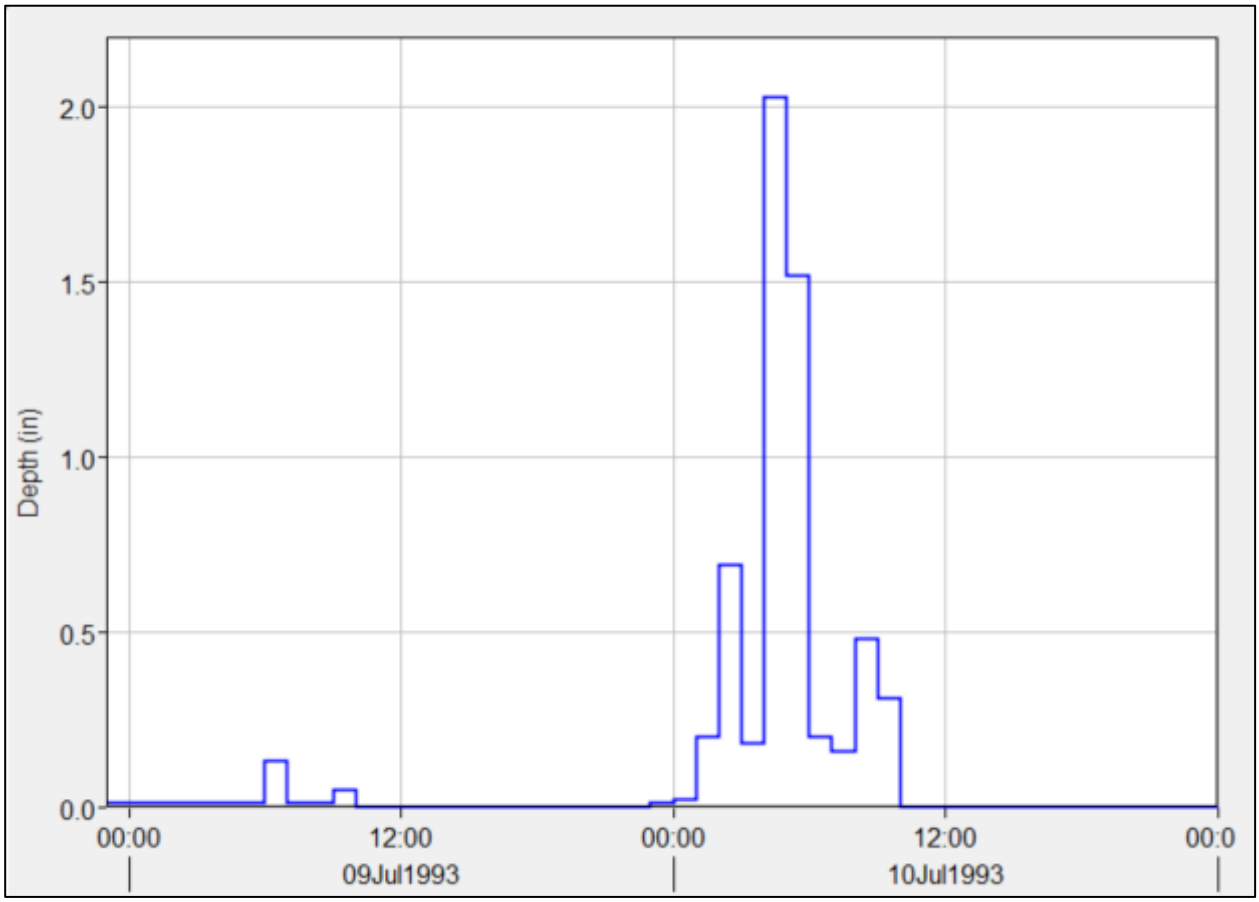


Figure 20. 10 July 1993 Rainfall hyetograph for Overland Park, Kansas City hall rain gage.

4.2.2.7. October 1998

The maximum flood of record for Turkey Creek occurred in October 1998 and resulted from the combined effects of saturated soil due to rain on 6 out of 7 days in one week and a high intensity storm that started 4 October 1998. The storm covered the Kansas City metropolitan area with rain for the entire day with afternoon rainfall totals of 0.25 to 1.00 inches. Around 7:00 p.m., another round of storms produced an additional 3 to 5 inches of rain in a 2-hour period. The rainfall produced by this event exceeded the rainfall amount defined as the 0.01 AEP (1/100-year event) at the time. Although this flood had the least amount of total rainfall when compared to the 1977 and 1993 floods, it produced the highest amount of rainfall in the peak hour. Figure 21 illustrates the 10 hours of highest rainfall intensity of the 1998 rainfall event hyetograph emphasizing the rapidness of this event. The worst damages caused by this flood occurred at Merriam, Kansas, near the midpoint of the Upper Turkey Creek reach. Multiple bridges and roadways were flooded during this event including I-35 and the BNSF railroad tracks south of 75th Street. The total cost of damages was estimated at $12 million in Merriam, Kansas, and $20 million in the lower basin area, in 1998 price levels.

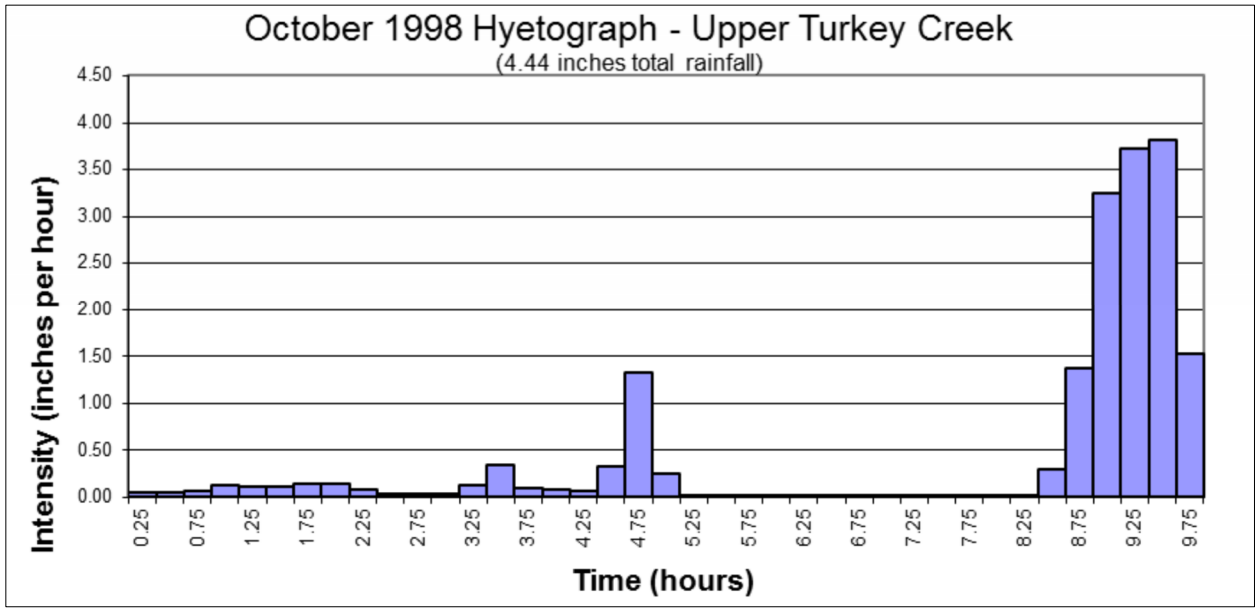


Figure 21. Hyetograph of 4 October 1998 Storm Event

4.2.2.8. 2017 Floods

Three significant rainfall events occurred in the Turkey Creek watershed during the summer of 2017 on 26–27 July, 5–6 August, and 21–22 August. Storm total precipitation was highest on 21–22 August, but the intensity was greater for 26–27 July and 5–6 August. Rainfall from the events produced some of the highest stages along Turkey Creek since the flood of record on 4 October 1998. Of the 2017 events, 26–27 July resulted in the record stage of 16.45 feet at the Johnson Drive gage.

Precipitation, flow rate, and stream stage from the 2017 events were compared to historical records and to published data. Precipitation totals for the 2017 events were determined using both Johnson County StormWatch rain gage data and NWS radar observations. The rain gages were located along Turkey Creek at Johnson Drive, Southwest Boulevard, and Lamar Avenue. Average basin precipitation totals

were estimated from the NWS radar data using the Meteorological Visualization Utility Engine (MetVUE) software.

26-27 July 2017 Flood

An average of 5.27 inches of rain fell over the Turkey Creek Basin between 12:00 p.m. on 26 July 2017, and 6:00 a.m. on 27 July 2017. Approximately 68 percent of this total fell in a three-hour period between 1:00 and 4:00 a.m. on the 27th; while the maximum hourly total was 1.52 inches, occurring between 2:00 and 3:00 p.m. on the 27th. Table 11 shows precipitation totals for the gages and the basin. All the rain gages recorded higher totals and more intense rainfall than the basin average. Figure 22 shows the basin average hyetograph.

Table 11. Precipitation totals for 26–27 July 2017.

Location	Total (inches)	Maximum 1-hour (inches)	Maximum 3-hour (inches)	Maximum 6-hour (inches)
Southwest Boulevard	5.92	1.80	4.12	5.32
Lamar Avenue	5.24	1.76	3.96	4.80
Johnson Drive	5.56	2.16	4.28	5.20
Basin Average	5.27	1.52	3.58	4.51

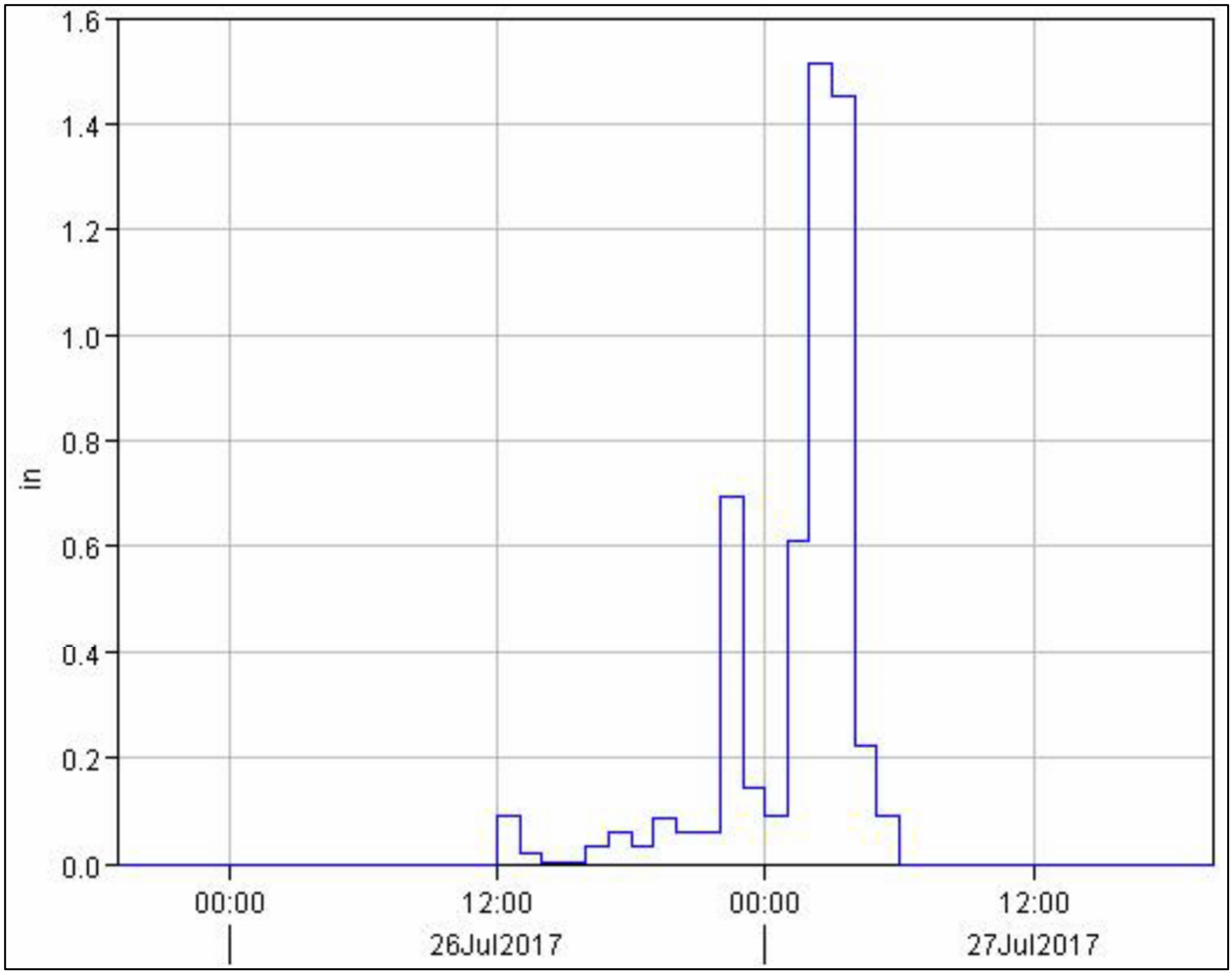


Figure 22. Turkey Creek Basin Average Hyetograph for 26–27 July 2017.

5-6 August 2017 Flood

A total of 4.75 inches of rain fell over the Turkey Creek Basin between 8:00 a.m. 5 August 2017, and 2:00 a.m. 6 August 2017. Approximately 75 percent of the total precipitation fell in a three-hour period between 8:00 p.m. and 11:00 p.m. on the 5th; while the maximum 1-hour intensity was 1.55 inches between 10:00 p.m. and 11:00 p.m. Table 12 shows storm precipitation totals for the gages and the entire basin. The rainfall totals were significantly higher at the Lamar Avenue and Johnson Drive gages than the basin average and the Southwest Boulevard gage. Peak discharge was estimated using the USACE lower Turkey Creek design model at Johnson Drive and Southwest Boulevard in the Turkey Creek 2017 flood report. Figure 23 shows the basin average hyetograph.

Table 12. Precipitation totals for 5–6 August 2017.

Location	Total (inches)	Maximum 1-hour (inches)	Maximum 3-hour (inches)	Maximum 6-hour (inches)
Southwest Boulevard	4.72	1.48	3.64	4.04
Lamar Avenue	6.32	2.24	4.48	5.20
Johnson Drive	6.12	2.68	5.20	5.40
Basin Average	4.75	1.55	3.58	3.89

*Note: Peak discharges estimated using RAS rating curves from lower Turkey Creek design model

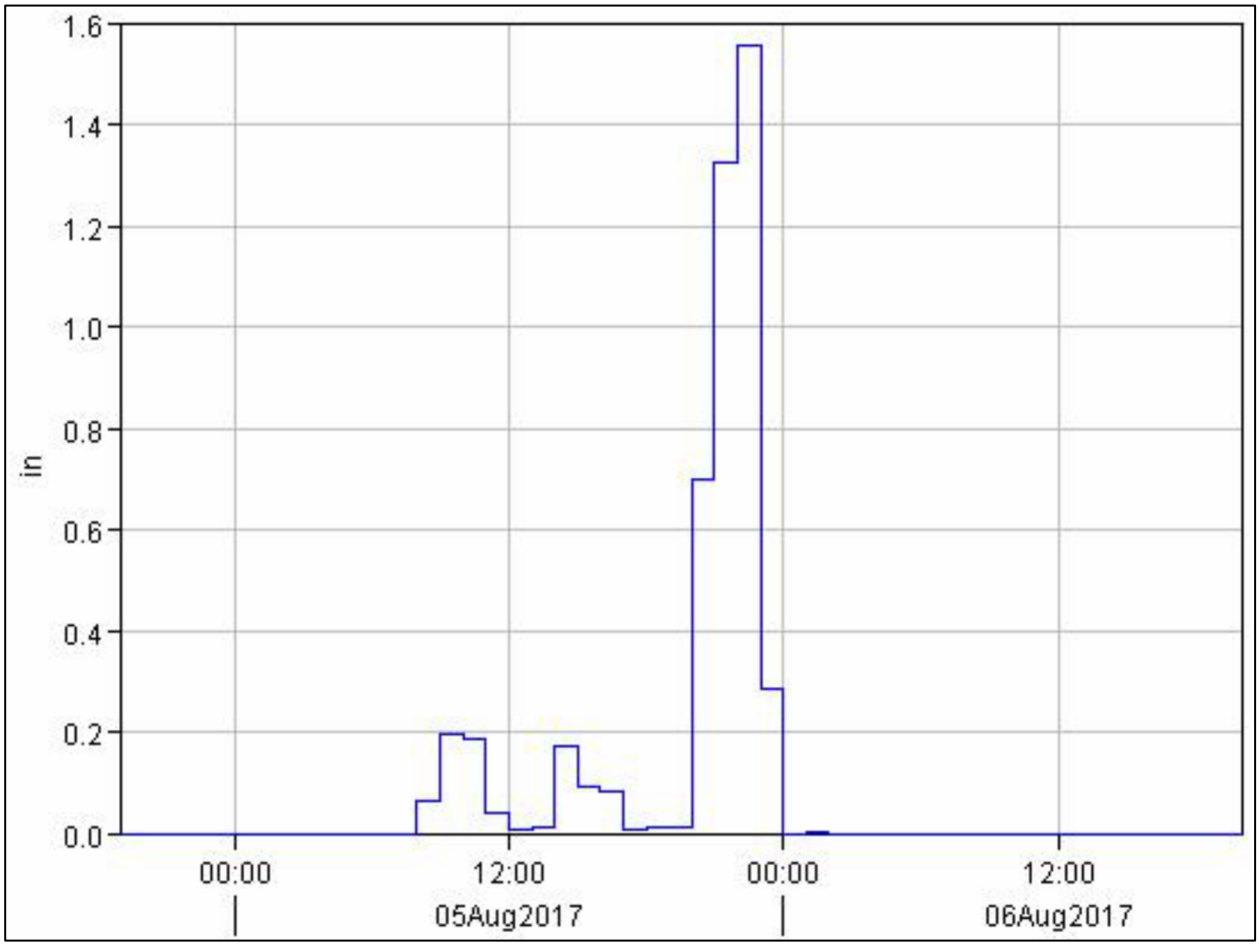


Figure 23. Turkey Creek Basin Average Hyetograph for 5–6 August 2017

21-22 August 2017 Flood

The total basin precipitation for 21–22 August 2017, was 5.71 inches, and occurred between 8:00 a.m. on the 21st and 6:00 a.m. on the 22nd. The maximum hourly total was 1.05 inches between 2:00 and 3:00 a.m. on the 21st, and the maximum three-hour total was 2.76 inches between 1:00 and 4:00 a.m. on the 21st. Table 13 contains the gage and basin rainfall totals for the event and indicates that the Southwest Boulevard gage recorded the greatest amounts. Figure 24 shows the hyetograph for the basin.

Table 13. Precipitation totals for 21–22 August 2017.

Location	Total (inches)	Maximum 1-hour (inches)	Maximum 3-hour (inches)	Maximum 6-hour (inches)
Southwest Boulevard	6.24	1.40	3.16	5.32
Lamar Avenue	5.24	1.08	2.72	4.64
Johnson Drive	4.96	1.08	2.84	4.56
Basin Average	5.71	1.05	2.76	4.60

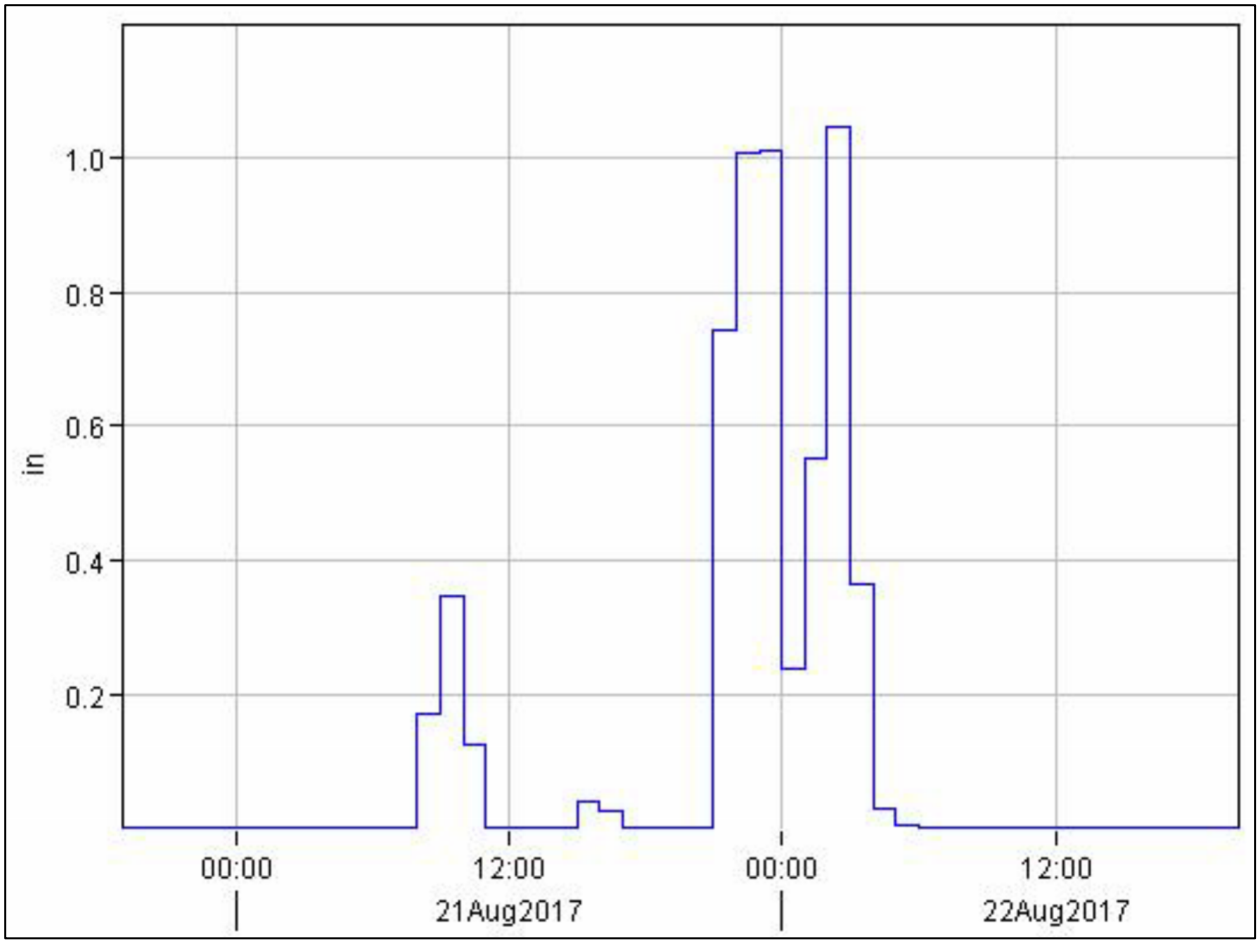


Figure 24. Turkey Creek basin average hyetograph for 21–22 August 2017.

4.2.3. Historic Trends and Ratings

The USGS has a published rating curve (https://waterwatch.usgs.gov/?id=mkrc), which relates the discharge to a water surface elevation (converted to "stage" at the gage). The rating curve is based on recent field measurements of water surface elevation and discharge and is typically "shifted" over time to better reflect the more recent data compared to older measurements. These shifts in the rating curve can be from a better understanding of the uncertainty of the field measurement data or could be an indication of geomorphologic changes to the river. Figure 25 shows the published USGS rating curve at Turkey Creek at Mission, KS (USGS 06892900). This gage has only been active since late 2022, so there has been little data of high magnitude events (field measurements) to develop the rating curve beyond the lower portion.

It should be noted that the USGS rating notes a stage of 3.43 feet as the 0 cfs flow, which corresponds to an elevation of 840.43 feet-NAVD88. This elevation differs significantly from the LiDAR channel bottom in the RAS model, which was determined to be 828.81 feet-NAVD88. LiDAR channel bottom is nearly 12 feet lower than the USGS rating curve zero flow stage which prevents the usability of this rating curve for calibration or validation purposes. The difference is illustrated below in Figure 26.

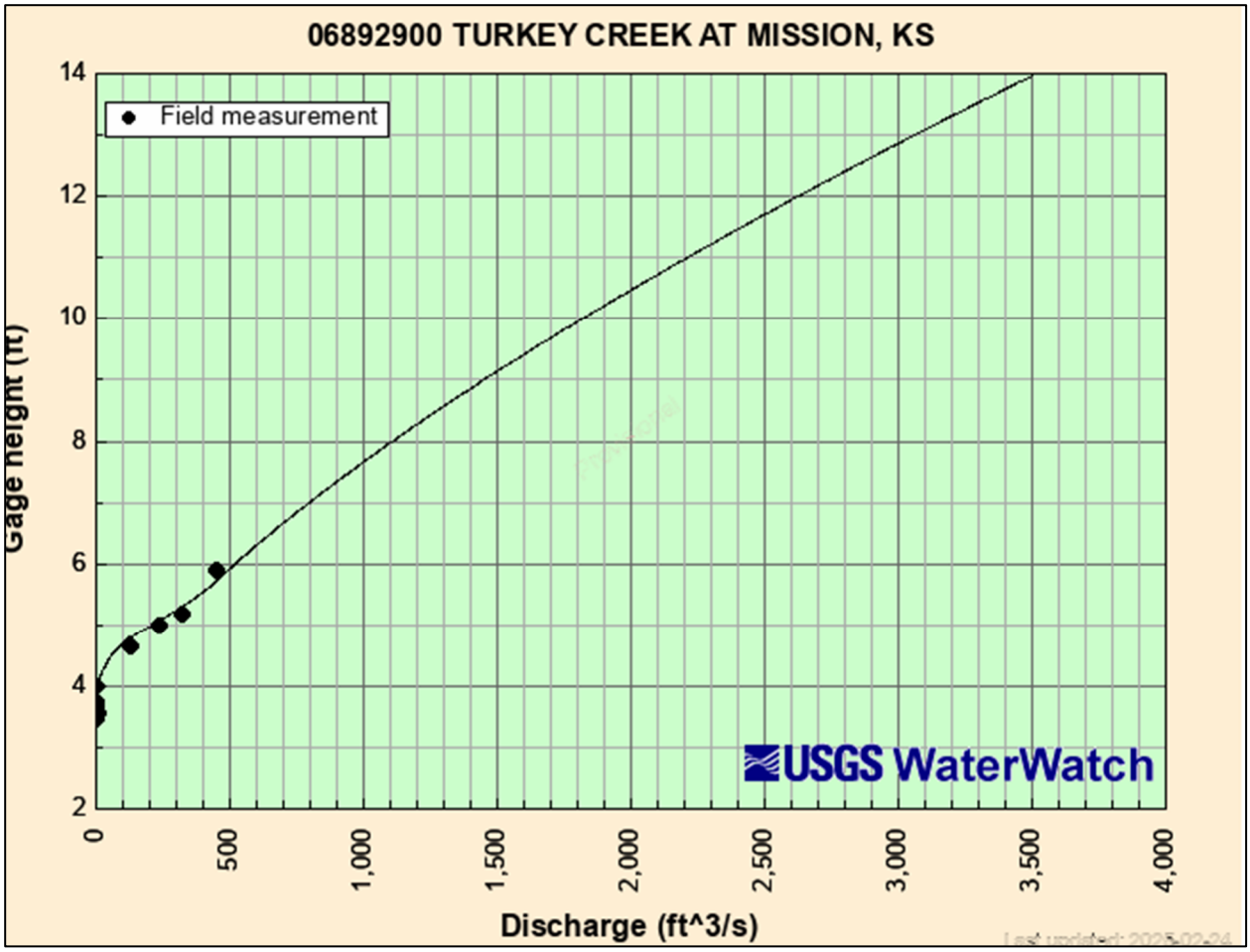


Figure 25. USGS discharge rating curve (zero gage height = 837.0 ft-NAVD88).

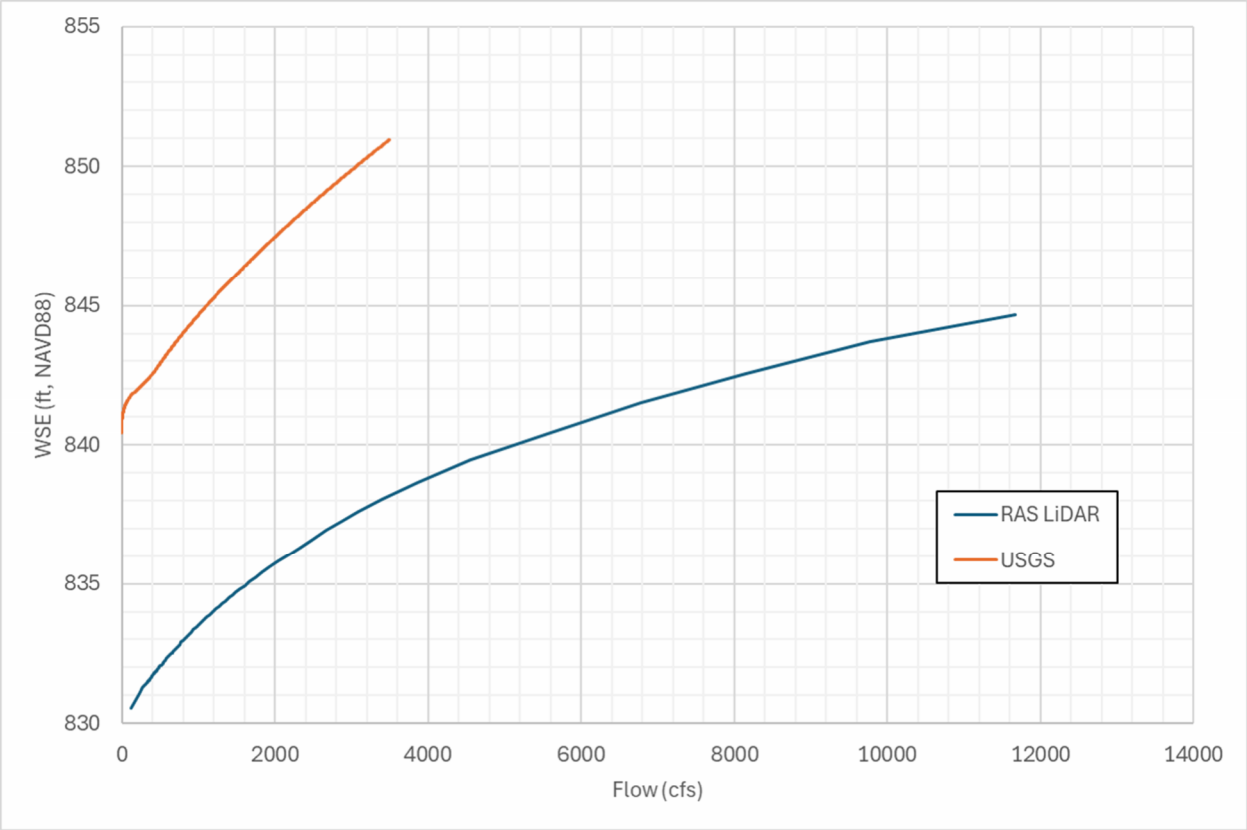


Figure 26. Comparison of USGS rating curve and HEC-RAS computed rating curve

4.2.4. Hydraulic Model Calibration/Validation

A fully two-dimensional rain-on-grid HEC-RAS hydraulic model was provided by Stantec which was developed in part to become the updated FEMA model for Johnson County, which was then modified to run 2D channel hydraulics instead of rain-on-grid for flood protection design (this model is referred to herein as the FEMA model). The FEMA model was previously calibrated with the August 22, 2017 and May 28, 2020 events and the July 2017 event was used for validation. It should be noted that calibration and validation of the FEMA model was completed using gridded precipitation of these historic events. There is not observed gage flow to serve as model inflows nor as calibration points. USACE found general uncertainty in the efficacy of channel and floodplain calibration of this model due to the use of rain-on-grid in such an urban area and a lack of stormwater modeling which significantly impacts routing times and volume accumulation. For this reason and to account for uncertainty, Manning's n-values for the channel were increased from 0.025 to 0.035 to account for any potential for underestimation of stages during levee profile design. The channel has seen sediment deposition over the last decade, so these changes were deemed appropriate.

Additionally, Manning's n-values meant to represent buildings were initially set to 1.0 in the model. This is typically only appropriate if cells with a 1.0 roughness value are exactly encompassing the building footprint. Otherwise, the large 1.0 value significantly skews the roughness where flow should be able to actively convey. In these situations, the HEC-RAS user's manual recommends setting the roughness values of buildings in the 2D mesh to 0.5. Since cells were not specifically configured to represent only

the building footprints, the building roughness value was changed to 0.5. Cell alignments within the channel and around critical areas were improved.

The model is not able to be calibrated in a non-rain-on-grid configuration due to the previously mentioned lack of observed flow data. This represents a major uncertainty in the hydraulics. HEC-RAS best practices were considered, and geometries were developed in such a way to lean towards conservatism given the lack of calibration data.

4.3. Hydrology

4.3.1. Peak Annual Discharge-Frequency

Hydrology was previously determined during the Upper Turkey Creek feasibility study phase and is being used for design. Flows were developed by taking Atlas 14 frequency precipitation depths and computing rainfall runoff to known inflow locations along Turkey Creek using an existing hydrologic model. HEC-1 was originally used during a 2005 Johnson County watershed study with TP40 precipitation and then was updated with Atlas 14 values during the feasibility study in 2015. Table 14 below details the frequency flows obtained by routing the Atlas 14 rainfall runoff in HEC-RAS at Johnson Drive bridge.

Table 14. Hydrology used for design.

AEP (1/year)	Peak flow at Johnson Drive (cfs)
0.002 (1/500yr)	22700
0.005 (1/200yr)	19900
0.01 (1/100yr)	17500
0.02 (1/50yr)	15600
0.05 (1/20yr)	13500
0.10 (1/10yr)	10500
0.20 (1/5yr)	8800
0.50 (1/2yr)	6800

During preliminary design, a Bulletin 17C analysis was conducted in HEC-SSP to check the validity of these results. Any available previous annual peak flow data from inactive USGS gages were taken and basin area adjusted to Turkey Creek at Kansas City, KS gage (USGS 06892940) along with best estimate flows for documented flood events as previously discussed in Section 4.2.1. Peak annual discharges from 1974 to 1987 and 1998 to 2023 were imported into HEC-SSP version 2.3. Discharge-frequency curves were developed for the observed period of record, with the addition of historical flows, and perception thresholds based on flood documentation. Regional skew values were not used in the Bulletin 17C analysis because they produced significantly lower flows for the 1% thru 0.2% AEP events. Generally regional skew values for Kansas and Missouri do not perform well in the urban Kansas City metro area. Table 15 provides the discharge-frequency statistics generated by HEC-SSP using the compiled record with historic data. Figure 27 provides the peak annual flow record for the Bulletin 17C analysis.

Table 15. Discharge-frequency statistics.

Statistic	Peak
Mean	3.710
Standard Deviation	0.200
Computed (Station) Skew	0.683
Regional Skew (Peak Flow)	N/A
EMA estimate of Mean Square Error (MSE)	0.165
Weighted Skew	N/A
Effective Record	94

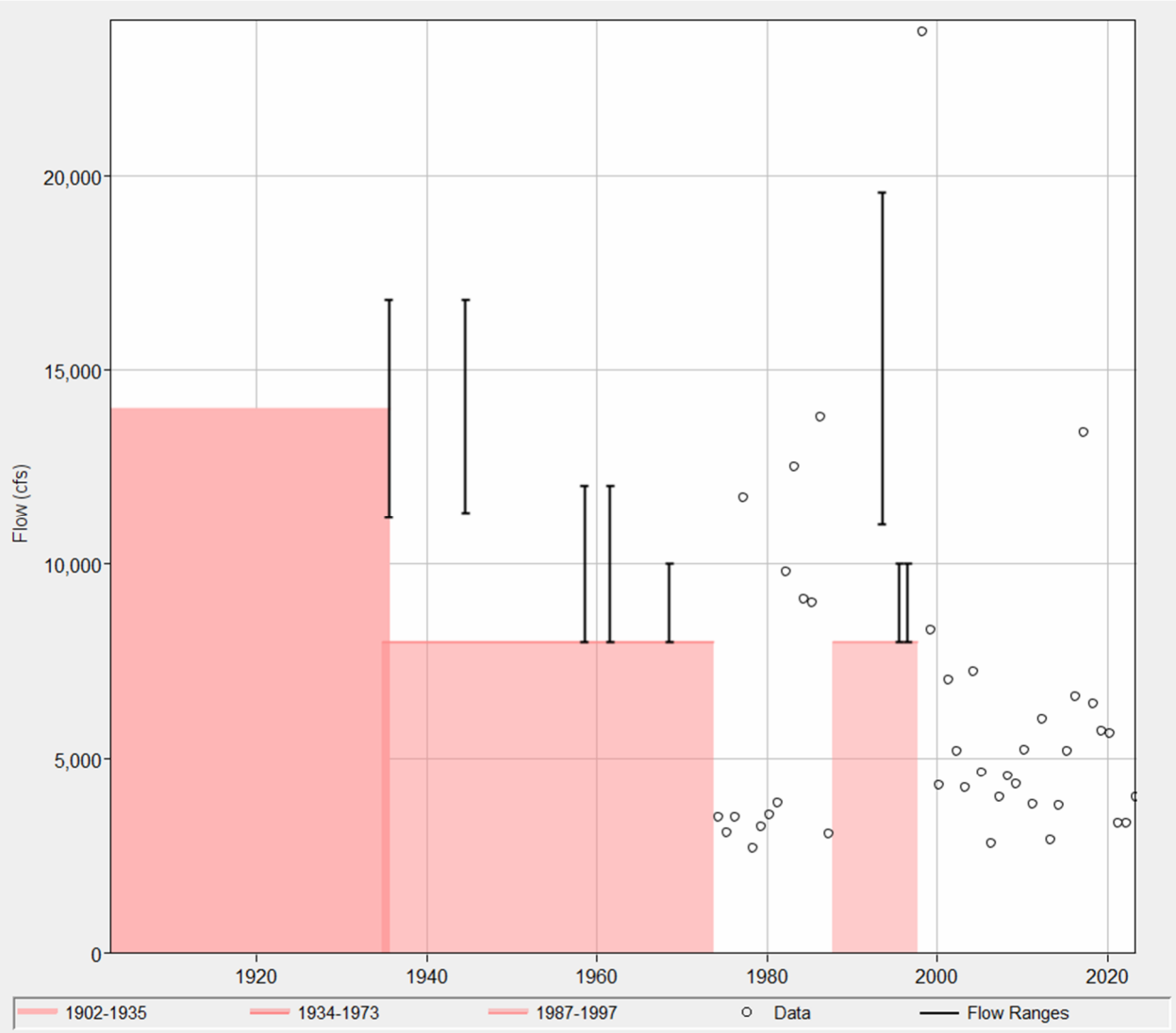


Figure 27. Bulletin 17C analysis: Peak annual flow record.

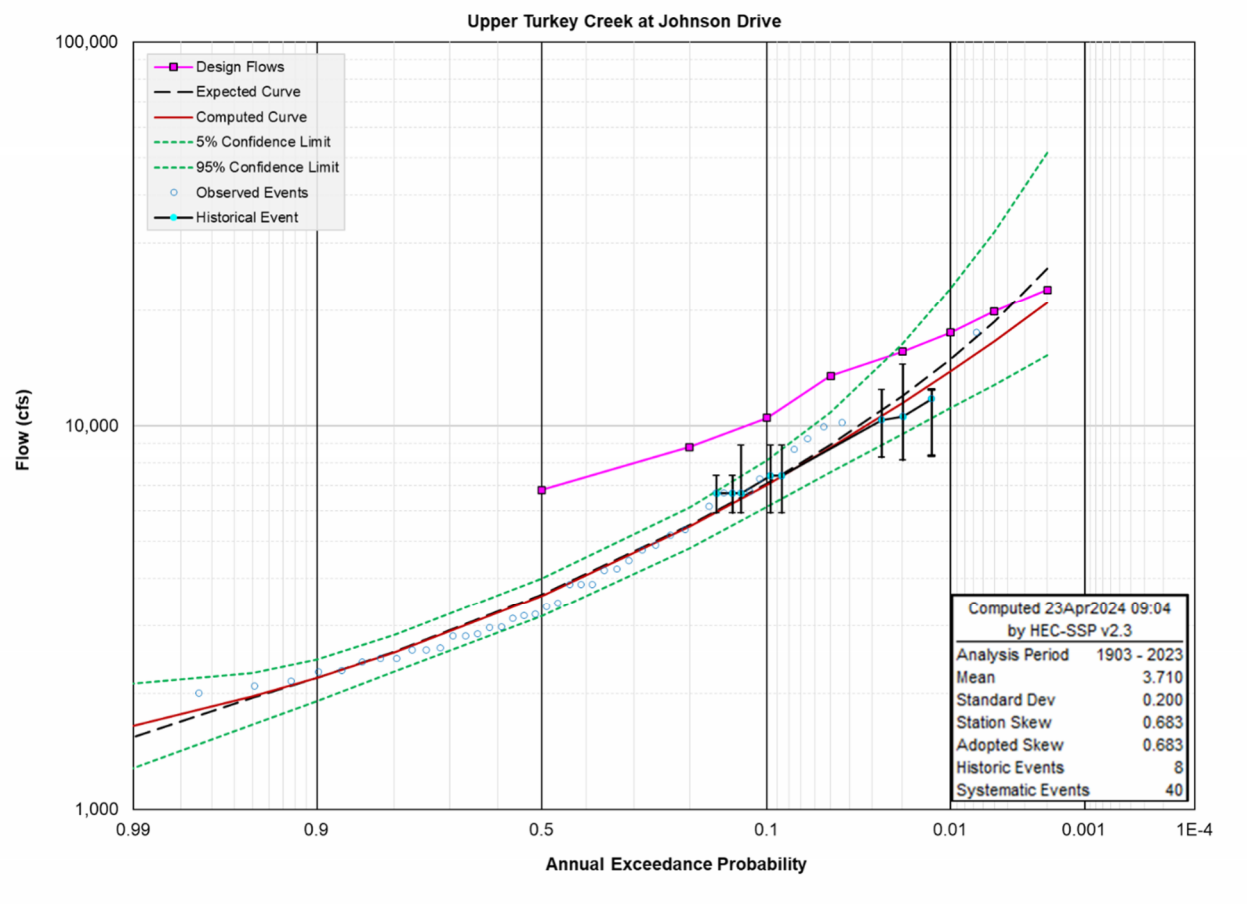


Figure 28. Bulletin 17C analysis: Peak discharge-frequency curves.

The discharge frequency curve was translated upstream to Johnson drive using an average regression ratio of 0.74. This ratio was developed by taking an average of the USGS 2017 regression equation (SIR 2017-5063) results between sites for each frequency. Table 16 below provides those computed results and a comparison to the HEC-1 routed frequency flows. The peak flows from routing the HEC-1 inflows through HEC-RAS trend higher than the Bulletin 17C analysis. These values fall within the confidence limits for the 0.02 AEP through 0.002 AEP flows. The more frequent discharges exceed the 0.05 confidence limit from the Bulletin 17C analysis. Less weight was given to these more frequent discharges given that they are outside of any magnitude of flow used for consequence modeling. The adopted design frequency of 0.01 AEP is within the confidence bounds of the Bulletin 17C curve but plots higher than the expected curve. Due to the lack of consistent observed annual peak flow data, and the resulting uncertainty in the flow frequencies, it was deemed reasonable to use the higher Atlas 14 HEC-1 computed inflows for design.

Table 16. Comparison of design hydrology to Bulletin 17C analysis.

AEP (1/year)	HEC-1 Peak Flow (cfs)	B17C Expected Flow (cfs)	Confidence Limits Flow (cfs)	
			0.05	0.95
0.002 (1/500yr)	22700	25800	51500	15300
0.005 (1/200yr)	19900	18700	32200	12800
0.01 (1/100yr)	17500	14900	22800	11100
0.02 (1/50yr)	15600	11900	16400	9520
0.05 (1/20yr)	13500	8900	10900	7570
0.10 (1/10yr)	10500	7100	8140	6170
0.20 (1/5yr)	8800	5500	6140	4800
0.50 (1/2yr)	6800	3600	4000	3180

4.3.2. Stage-Duration Curves

Annual stage-duration curves were developed using HEC-SSP at the Johnson Drive at Turkey Creek (StormWatch TU04) gage, which is located within the project extents. It should be noted that the period of record for the Stormwatch gage TU04 goes back to 1998 but observed stage data prior to 2014 was very inconsistent, including large shifts in average low flow stages for months at a time before suddenly shifting upwards or downwards. Additionally, there were numerous erroneous spikes in the stage data well beyond the record flood stage during smaller events. For this reason, only data from 2014 and onwards was used to develop the stage-duration curve. The stage-duration curve for Johnson Drive is shown in Figure 29.

Because the levee/floodwall does not experience a water surface elevation of 1 foot or higher above the elevation of the landside levee or floodwall toe at least once a day for more than 36 days (10 percent of the time) per year on average, it is considered infrequently loaded according to ER 1110-2-1806.

A stage-duration curve was also computed further downstream using the Lamar Avenue at Turkey Creek (StormWatch TU08) gage to approximately represent the downstream boundary of the hydraulic model. Similar to the Johnson Drive gage, the pre-2014 data was quite inconsistent and was not used in the analysis. There is no levee system at this location and so an approximate channel capacity stage was shown instead for Figure 30.

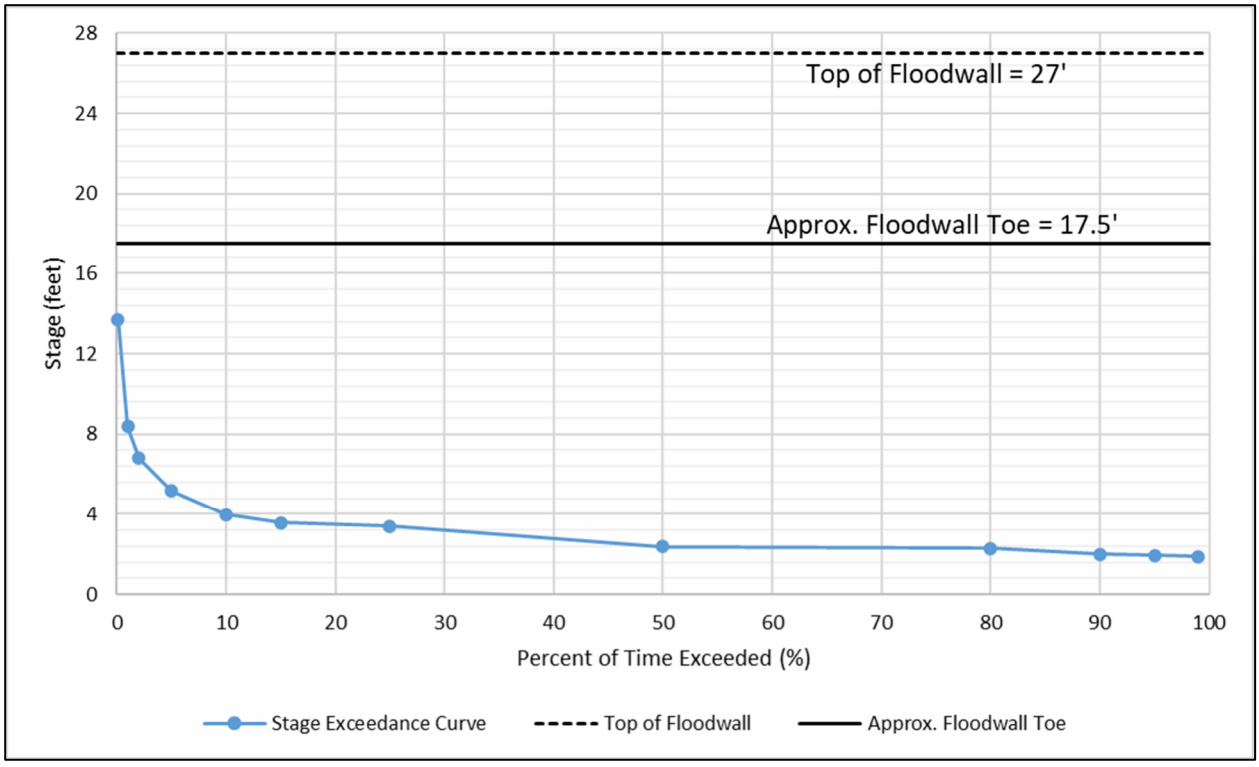


Figure 29. Stage-duration curve at Johnson Drive at Turkey Creek gage (StormWatch TU04).

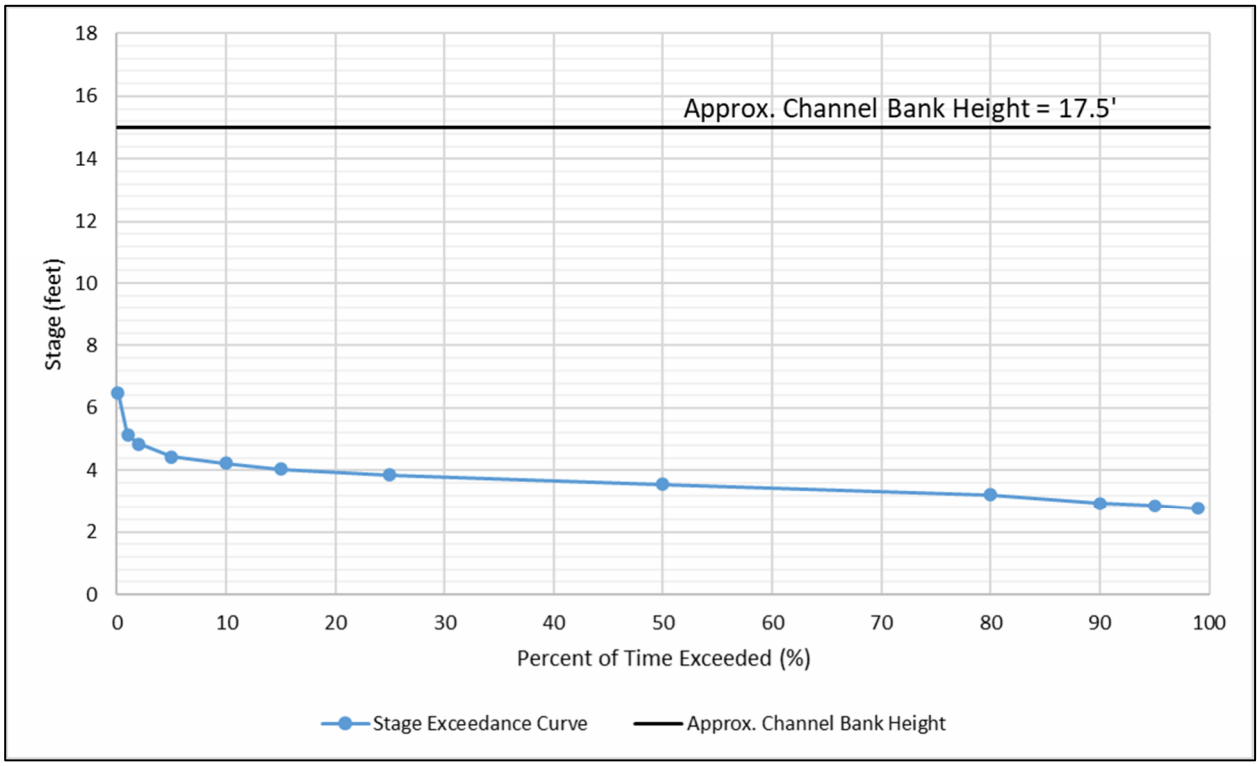


Figure 30. Stage-duration curve at Lamar Avenue at Turkey Creek gage (StormWatch TU08).

4.3.3. Volume-Frequency

Volume-frequency analysis for durations beyond the peak discharge frequency was not performed. Turkey Creek is characterized as a flashy system and flood events are typically less than 1 day in duration. Additionally observed daily or sub-daily flow data is not available.

4.3.4. Expected Flood Durations

Figure 31 provides the synthetic frequency hydrographs used for the SQRA analysis as well as the observed July 2017 stage hydrograph at the Johnson Drive StormWatch gage. The July 2017 flood event is the flood stage of record in the Johnson Drive gage record. The 2017 flood exceeded the banks at this location but not the toe height. The 0.002 AEP (1/500 year) hydrograph overtops the levee slightly in this location. More significant overtopping occurs at other locations of the levee system for this frequency event.

It should be noted the expected flood durations for both the synthetic and observed events are very short. As seen in the figure, these hydrographs exceed the banks and then recede back below the bank in 2-6 hours depending on the flood magnitude. Flood events on Turkey Creek are very short lived even with extreme cases such as the 0.002 AEP (1/500 year) hydrograph.

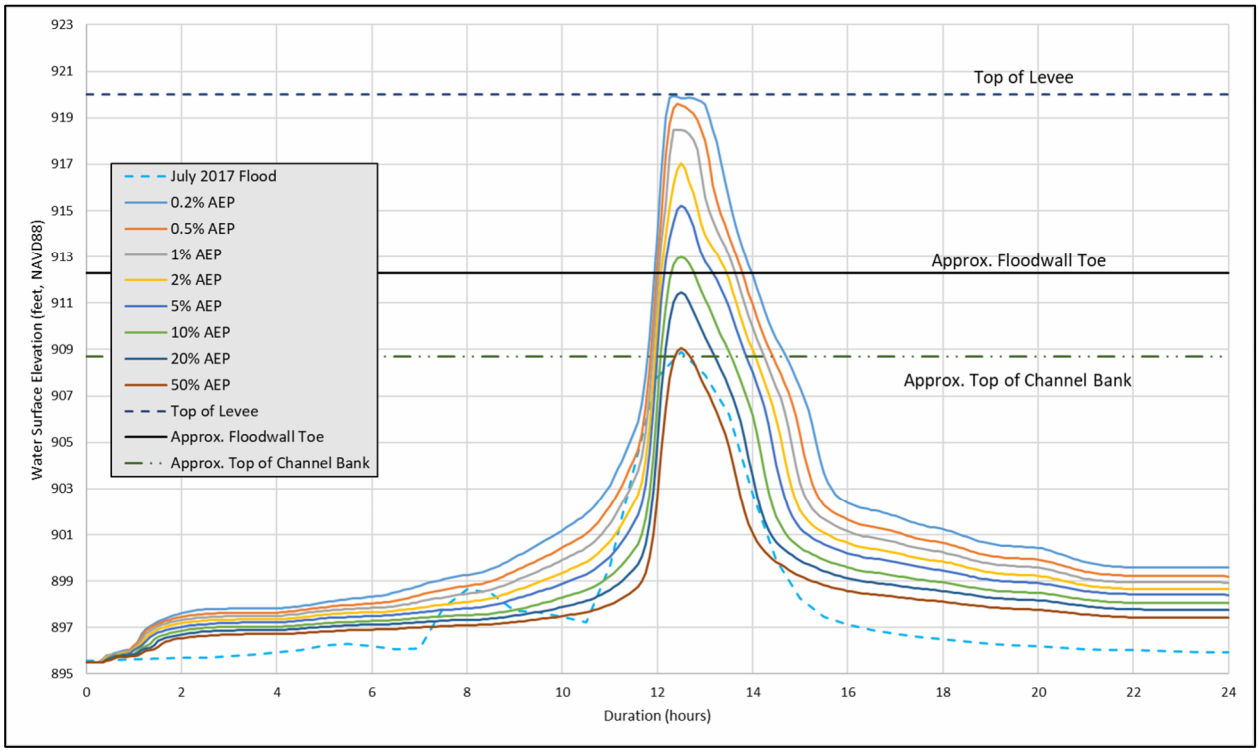


Figure 31. Synthetic flood hydrographs with July 2017 flood hydrograph.

4.4. Hydraulics

4.4.1. Boundary Conditions

The HEC-RAS hydraulic model was carried forward with modifications that were discussed. This includes the truncation of the upper portion of the model to switch from rain-on-grid to flow hydrograph boundary conditions. The downstream boundary was set to normal depth, using the approximate bed slope of 0.00561 at that location. Figure 32 provides a schematic of the hydraulic model. Numerous local inflow hydrographs were set according to the HEC-1 hydrologic output locations along Turkey Creek to better represent natural channel inflows. Figure 33 details all inflow locations. More detailed information on the model setup and parameter selection can be found in the 35% DDR.

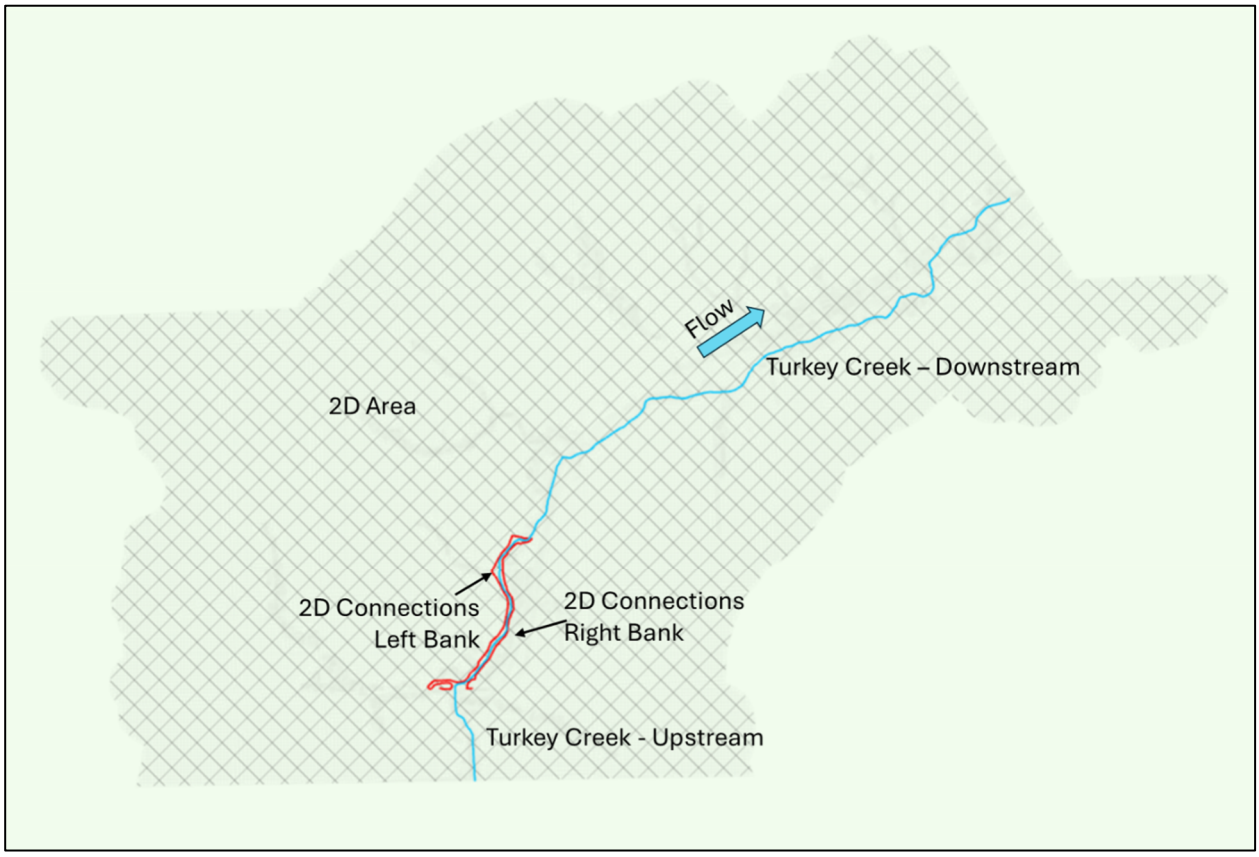


Figure 32. Hydraulic model schematic.

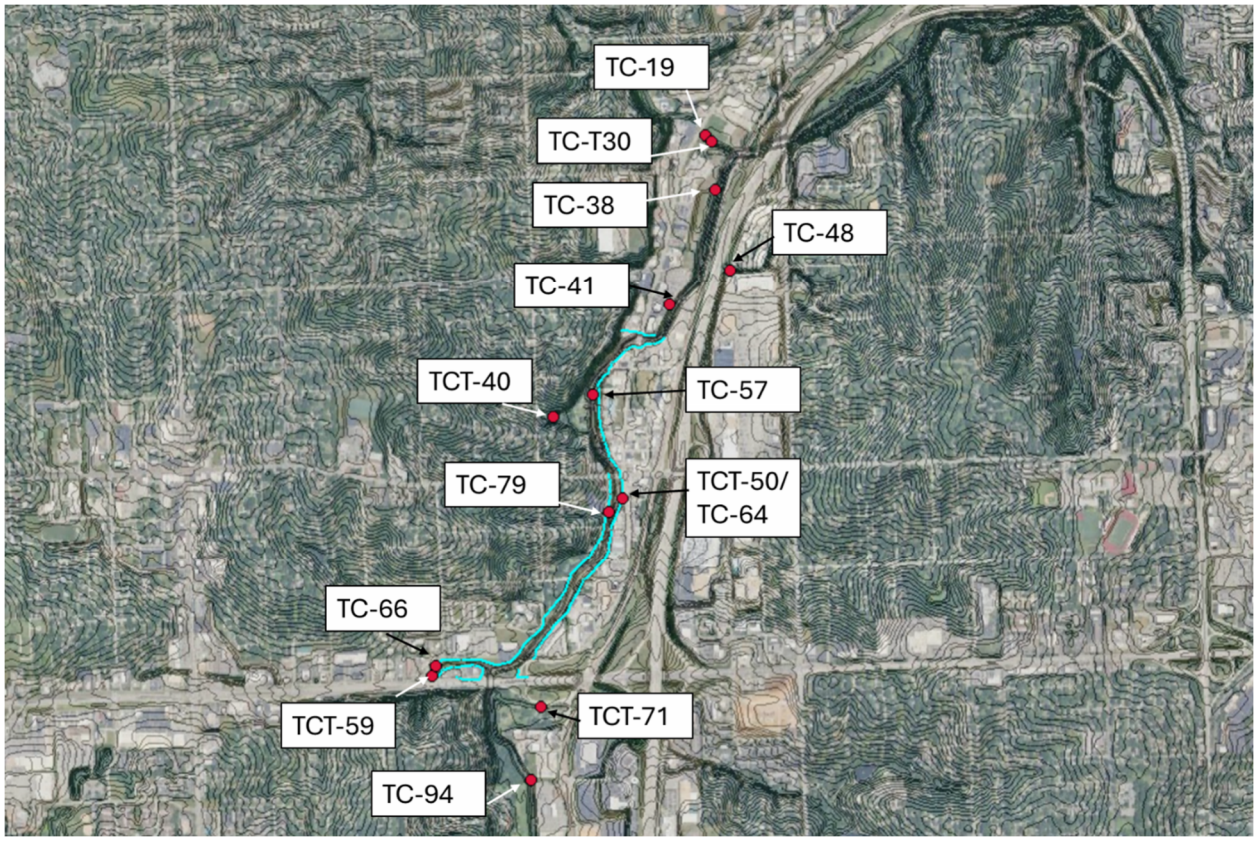


Figure 33. Flow hydrograph boundary condition locations.

4.4.2. Coincident Frequency Analysis

Water surface elevations along the levee system can theoretically be influenced by flood flows coming from a portion of Turkey Creek upstream of the project and/or the tributary at the start of the levee alignment. Given the very small drainage area of this watershed, there is not a case where the upstream Turkey Creek drainage area would experience extreme flooding and the tributary drainage area does not, or vice versa. Therefore, a coincident frequency analysis was not performed.

4.4.3. Flow Factoring

Frequency flows were routed through the HEC-RAS model using the synthetic design frequency flows. Historic flow hydrographs were not available given the lack of any sub-daily observed flow at any of the Turkey Creek gages. Frequency hydrographs are generally preferred because historic inflow hydrographs can get notably distorted when flow factor magnitudes get larger for very infrequent events such as the 0.002 AEP flow. This also assumes that the synthetic hydrographs are representative of typical flood durations experienced in the project area, which has shown to be the case previously in Section 4.3.4. These synthetic hydrographs allow for use of less extreme inflow factoring and prevent distortion of hydrograph shapes. All inflow hydrographs were factored uniformly to better represent larger storm

events, leading to larger local inflows within the project extents. Table 17 provides a summary of the frequency events and flow factors used for each loading scenario.

Table 17. Flow factoring for loading scenarios.

Loading Scenario	Elev at LCL	Base Flow Frequency	Flow Ratio
50%	909.03	100yr	0.904
75%	909.6	100yr	0.956
Top of Levee	910.1	200yr	0.908
0.5 ft OT	910.6	200yr	0.958
1.0 ft OT	911.1	500yr	1
2.0 ft OT	912.1	500yr	1.155

4.4.4. Levee Control Location

The levee control location (LCL) was identified at channel station 65+40 or right bank station 56+63 as shown in Figure 34. It should be noted that while this is the location of incipient overtopping, numerous other portions of the left and right bank systems begin overtopping within 5 to 10 minutes after. The current iteration of the levee profile design behaves as balanced overtopping. Levee superiority will be considered as design progresses to ensure overtopping does not occur in the upstream portions of the levee and lead to increases in consequences.

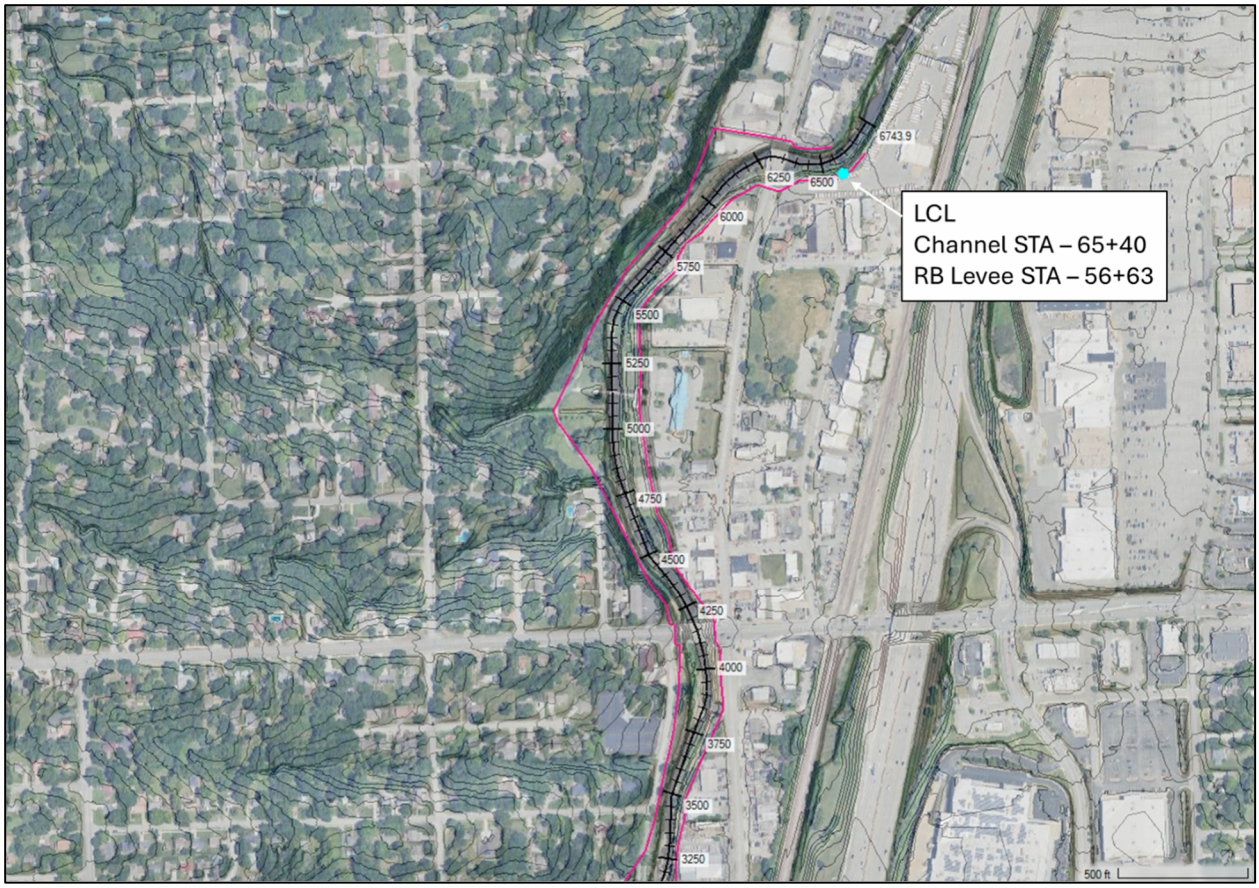


Figure 34. Location of incipient overtopping

4.4.5. Stage-Frequency Curves

Stage-discharge-uncertainty was estimated in accordance with EM 1110-2-1619 (USACE 1996) by performing a sensitivity analysis with Manning's n-values where upper limit and lower limit scenarios were modeled. The standard deviation was estimated by taking the mean stage difference between these estimates and dividing the difference by four. When compared to Table 5-2 in EM 1110-2-1619, the standard deviations were found to be lower than the fair and poor manning's n reliability classifications of 0.7 and 1.3, respectively. There is higher confidence in the quality of terrain data and manning's n values were set to be slightly higher than expected to account for uncertainty in design due to lack of calibration and validation. For this reason, the computed standard deviations were kept for development of the stage-frequency curve.

Table 18 summarizes the stage-discharge-uncertainty, which is also graphically portrayed in Figure 35. The stage uncertainty generally decreases as flow increases due to changes in hydraulic conditions. Conveyance area increases as stage increases and so the same flow increases produce smaller changes in water surface, thereby resulting in less sensitivity to manning's n variance. This effect is much more pronounced once overtopping occurs and flow conveyance area expands to the full floodplain width. Future analysis and design will perform sensitivity testing with stage uncertainty and evaluating its effect on levee height. It will be determined whether using larger stage uncertainties is appropriate for design, moving forward.

Table 18. Summary of standard deviation in stage uncertainty.

AEP Range	Flow Range (cfs)	Standard Deviation for Stage Uncertainty (ft), Index Location A
1-0.5	0 to 6,800	0.5
0.5-0.2	6,800 to 9,200	0.411
0.2-0.1	9,200 to 11,000	0.358
0.1-0.05	11,000 to 14,300	0.378
0.05-0.02	14,300 to 16,600	0.365
0.02-0.01	16,600 to 18,600	0.356
0.01-0.005	18,600 to 18,900	0.301
<0.005	>18,900	0.136

To prevent flatlining of the stage-frequency curve, the RAS rating curve at the LCL was linearly extrapolated out beyond the 1E-04 frequency as shown in Figure 35 to a flow and elevation of 40,000 cfs and 915.5 feet, respectively. Water surface elevations were extracted from the modeling results using a reference point at a 2D channel cell nearest the LCL.

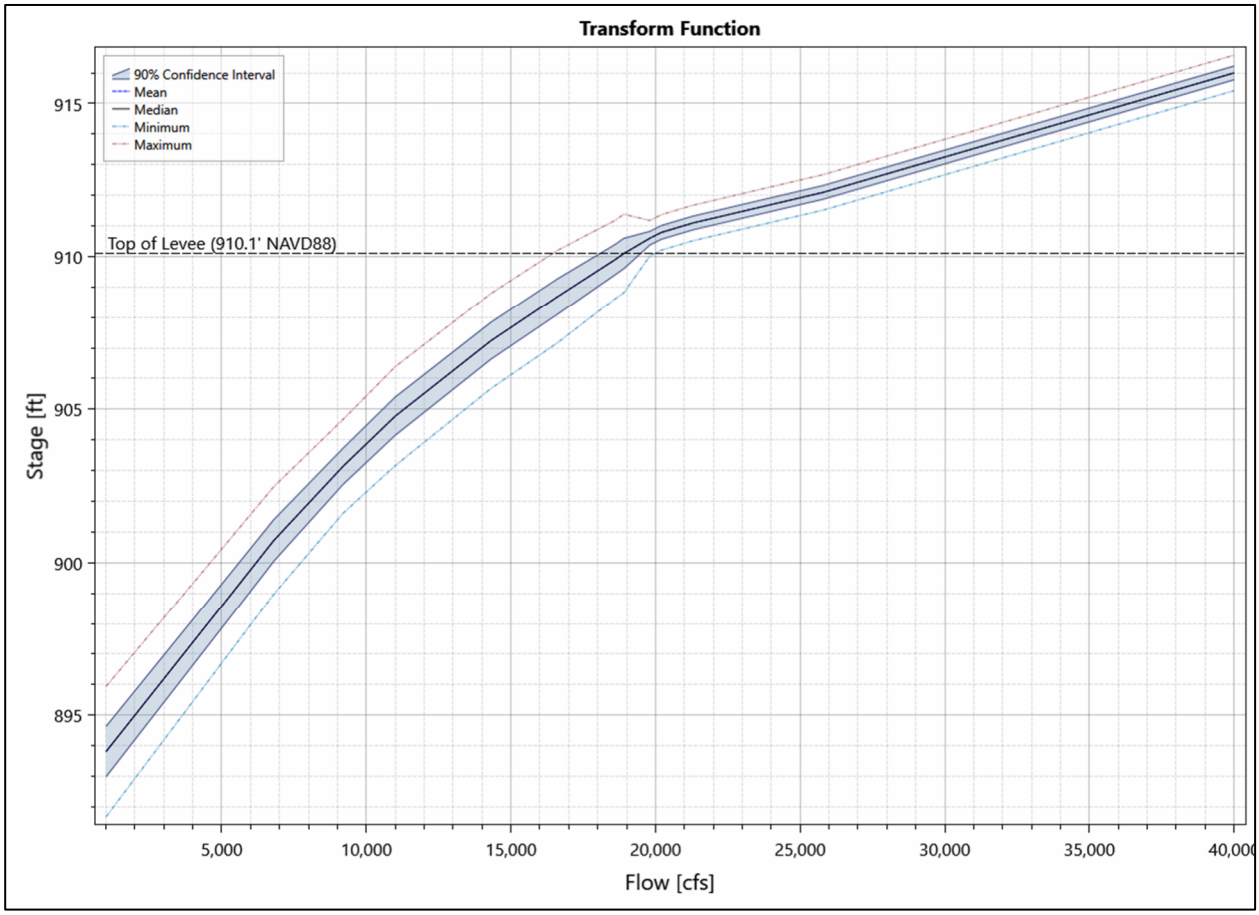


Figure 35. Elevation-discharge relationship and uncertainty at LCL (Channel STA 65+40).

Table 19. Change in flows between Johnson Drive and LCL location.

AEP	Flow at Johnson Drive (cfs)	Flow at LCL (Channel STA 65+40) (cfs)
0.5	6,700	6,800
0.2	8,900	9,200
0.1	10,600	11,000
0.05	13,900	14,300
0.02	16,000	16,600
0.01	18,000	18,600
0.005	19,600	20,200
0.002	22,200	21,300

The resulting stage-frequency curve (Figure 37) at the LCL with uncertainty was developed using RMC-TotalRisk by applying the flow-frequency and uncertainty for the hazard and the stage-discharge relationship and uncertainty as a transform. Uncertainty bounds were not developed as part of the design frequency flows form the HEC-1 outputs. Therefore, an effective record length of 25 years was entered in manually to represent the approximate gage record length of the Johnson Drive StormWatch gage. This flow-frequency curve was assumed to be representative of the LCL location given the little change in flow between Johnson Drive and the LCL as shown above in Table 23. Peak velocity and location for loading scenarios.

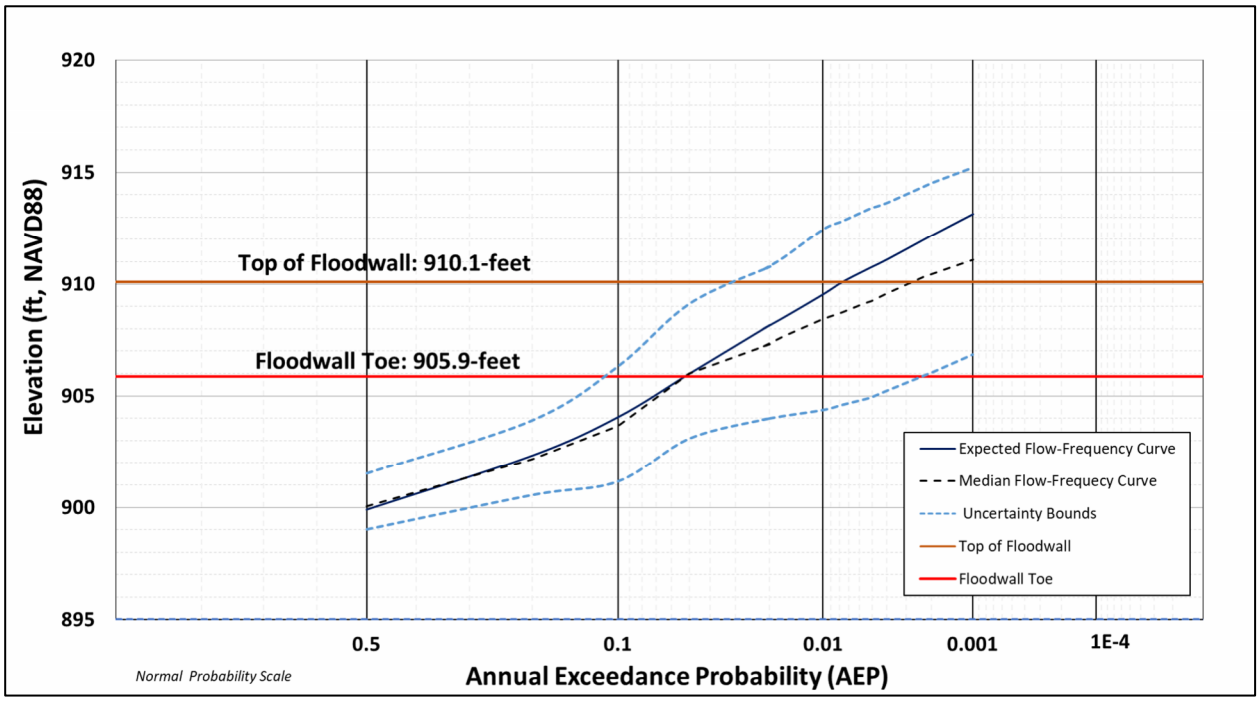


Figure 36. Elevation-frequency curve at LCL (Channel STA 65+40).

Based on the above computed stage-frequency curve in Figure 36, the incipient overtopping frequency at the identified LCL is approximately 7.65E-03 AEP (1/130yr) using the expected curve. This frequency seems realistic given that the levee was designed to the 0.01 AEP design flow with 85% reliability. Table 20 provides a summary of the stage frequency results including 0.5, 1, and 2-foot overtopping depths.

Table 20. Computed Stage-Frequency Results at LCL.

AEP	1/year	Expected Stage (feet)	Upper 90% CI Stage (feet)	Lower 90% CI Stage (feet)
1.00E-03	1,000	913.15	915.24	906.88
2.00E-03	500	912.19	914.53	906.04
2.13E-03 (2ft (OT)	470	912.10	914.46	905.99
4.00E-03 (1ft (OT)	250	911.10	913.64	905.23
5.00E-03	200	910.78	913.42	904.95
5.56E-03 (0.5ft OT)	180	910.60	913.27	904.85
7.69E-03 (TOL)	130	910.10	912.83	904.61
1.00E-02	100	909.55	912.46	904.37
2.00E-02	50	908.16	911.19	904.10
5.00E-02	20	905.99	910.77	904.00
1.00E-01	10	904.05	910.63	903.94
2.00E-01	5	902.32	909.12	903.09
5.00E-01	2	899.89	906.32	901.16

4.4.6. Levee System Profile

The proposed configuration has a left-bank and right-bank alignment. The left bank flood protection begins near Shawnee Mission Parkway and ties into high ground near Johnson Drive bridge. The left-bank system then resumes in a separate segment near Merriam Drive Bridge. The right-bank flood protection extends from Shawnee Mission Parkway all the way to just downstream of Merriam Drive Bridge. Figure 37 and Figure 38 detail the alignments and stationing for the left-bank and right-bank, respectively. It should be noted that alignment stationing is different for the left and right bank systems. A channel centerline stationing is used to correlate the levee alignments as shown in Table 21.

Table 21. Alignment stationing crosswalk.

Channel STA	Left Bank STA (and wraparound STA)	Right Bank STA
0+40	13+10 (11+70)	-
2+00	14+85 (10+10)	-
4+00	17+25 (8+35)	-
6+00	19+30 (6+40)	-
8+00	21+50 (4+55)	-
10+00	23+20	-
15+00	28+05	6+25
20+00	33+00	11+50
25+00	37+70	16+70
30+00	43+10	21+40
35+00	47+55	28+65
40+00	52+20	32+00
45+00	56+90	37+05
50+00	62+85	41+85
55+00	68+60	46+40
60+00	73+75	51+40
65+00	80+30	55+90
65+66	-	56+70

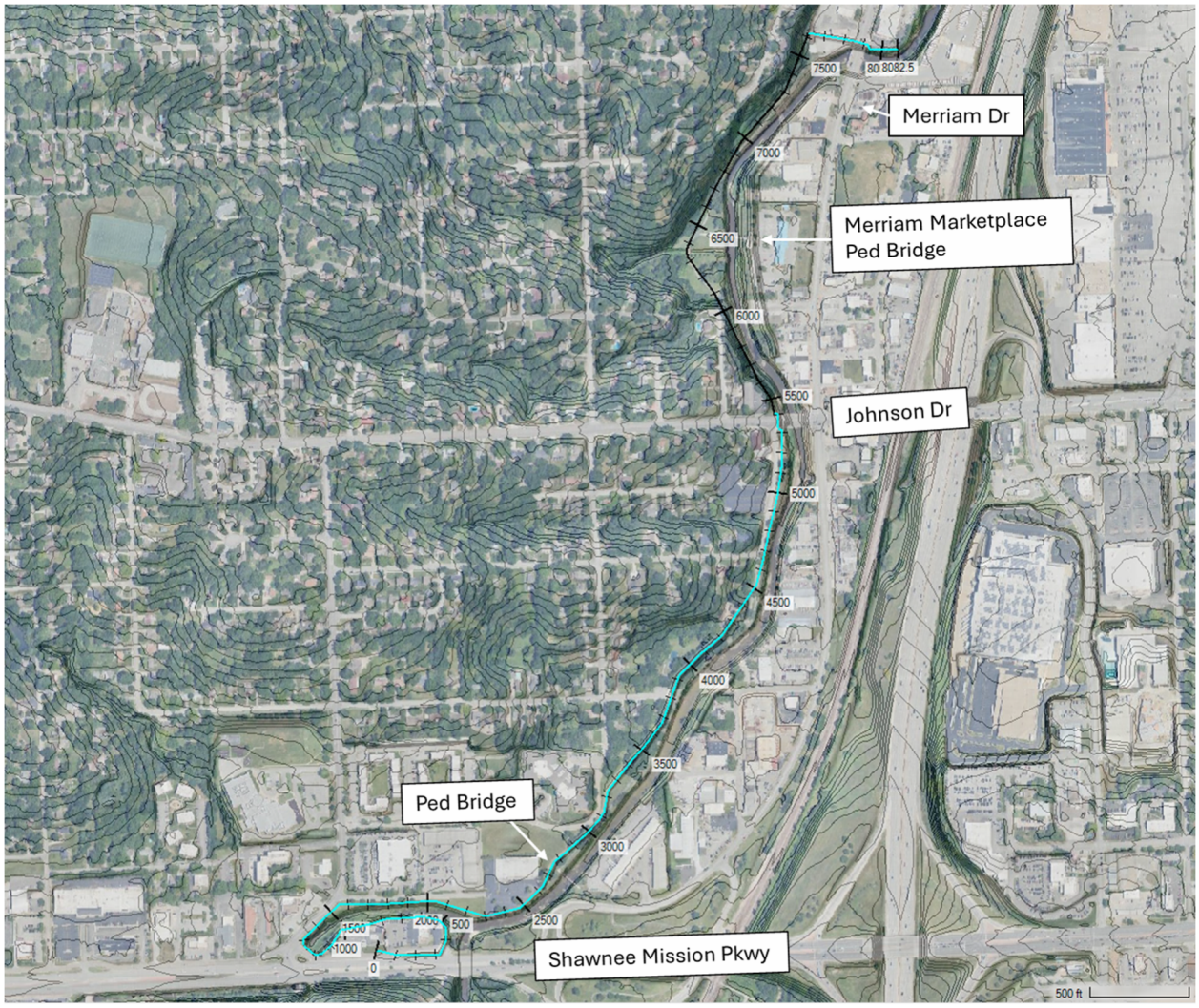


Figure 37. Left-bank flood protection alignment.

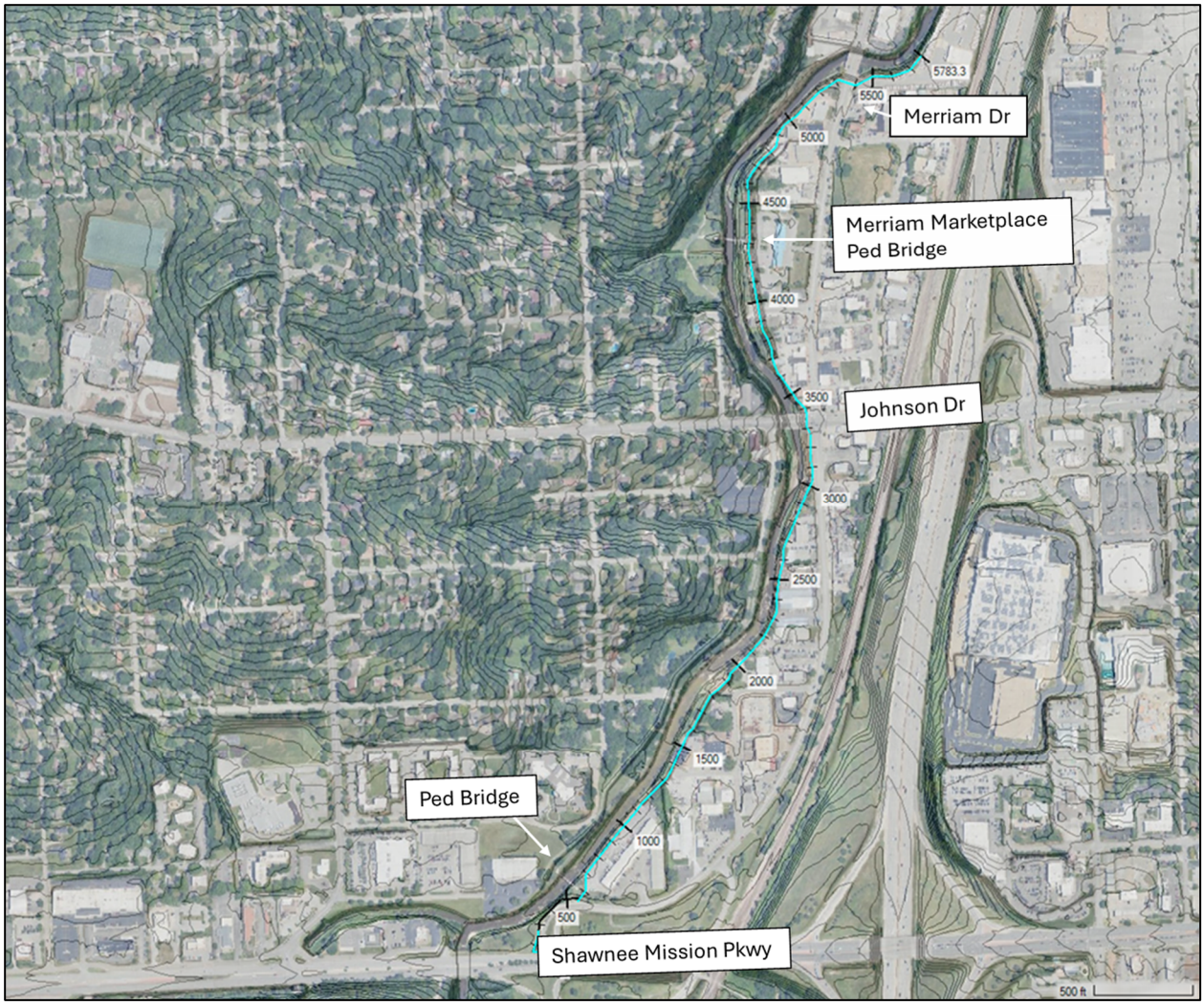


Figure 38. Right-bank flood protection alignment.

A levee system profile was developed in excel with hydraulic modeling results and relevant system features. The profile plots were broken into two separate figures, a left bank and right bank plot. The left and right bank levees have their own stationing which was utilized for each respective plot. Profiles such as the channel invert and water surface profile were represented using the levee stationing. Chosen breach locations for the SQRA were also plotted. It should be noted that the left bank levee profile increases in elevation at the upstream end of the system with increasing stations before sloping back down. This is the portion of the levee that wraps around the small tributary. Approximate left bank levee stations 11+50 through 13+50 represent the most upstream end of the tributary project extents.

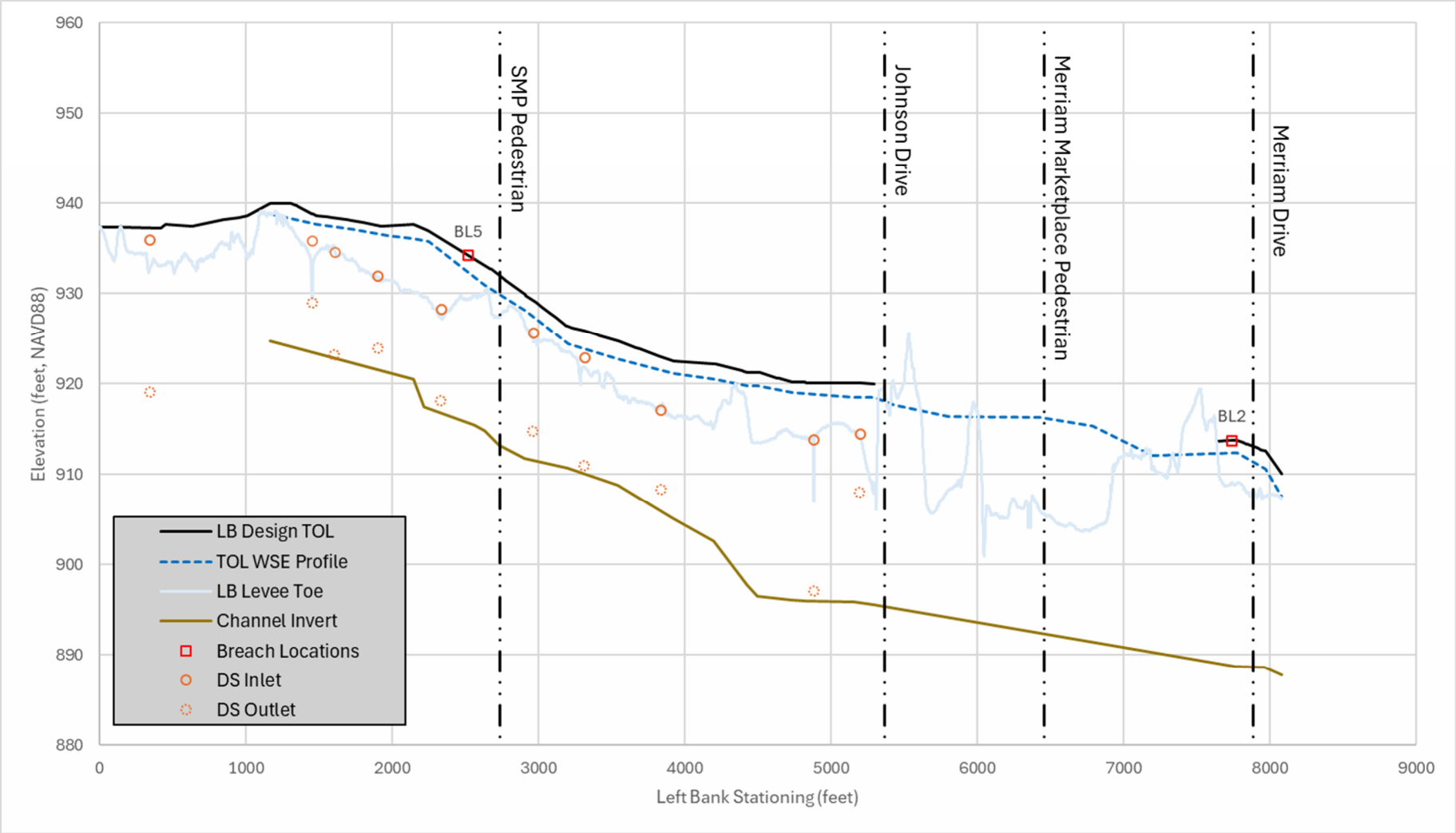


Figure 39. Left Bank comprehensive system profile with relevant hydraulic, observed, and system profile data points.

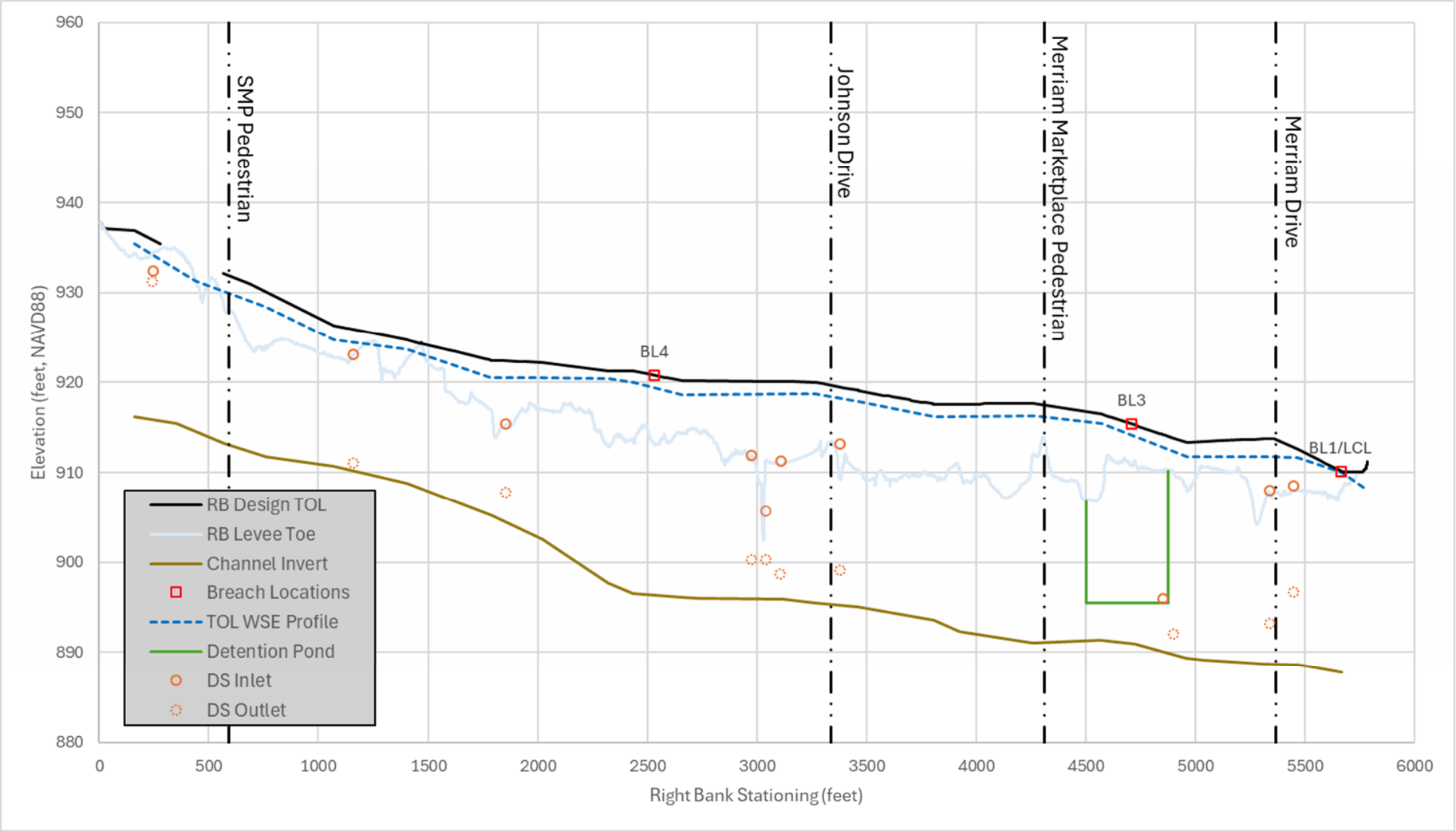


Figure 40. Right Bank comprehensive system profile with relevant hydraulic, observed, and system profile data points.

4.4.7. Overtopping

The synthetic frequency inflow hydrographs were scaled and routed through the hydraulic model to hit peak stages at the LCL that result in 0.5 ft, 1 ft, and 2 ft of overtopping. The resulting durations of overtopping, within 0.1 feet of peak, and duration above toe were extracted from the model results as an estimate of duration for the overtopping potential failure mode (PFM). Depth and duration summaries are shown in Table 22 and Figure 41.

Overtopping durations are very short, even for the more extreme 2-foot overtopping loading condition. Durations above the toe elevation at the LCL are also very short. This is likely because the LCL location occurs at a portion of floodwall that is only approximately 2 feet tall. Breach parameters based on the overtopping scenarios are discussed later in Chapter 6.

Table 22. Overtopping depth and duration summary at LCL.

Peak Overtopping Depth (ft)	Total Overtopping Duration	Peak Overtopping Stage Duration (within 0.1 ft of peak)	Duration Above Toe (Elev 907.9 feet)
0.5	0.5 hrs	0.25 hrs	1 hr
1	1 hr	0.1 hrs	1.5 hrs
2	1.5 hrs	0.25 hrs	1.67 hrs

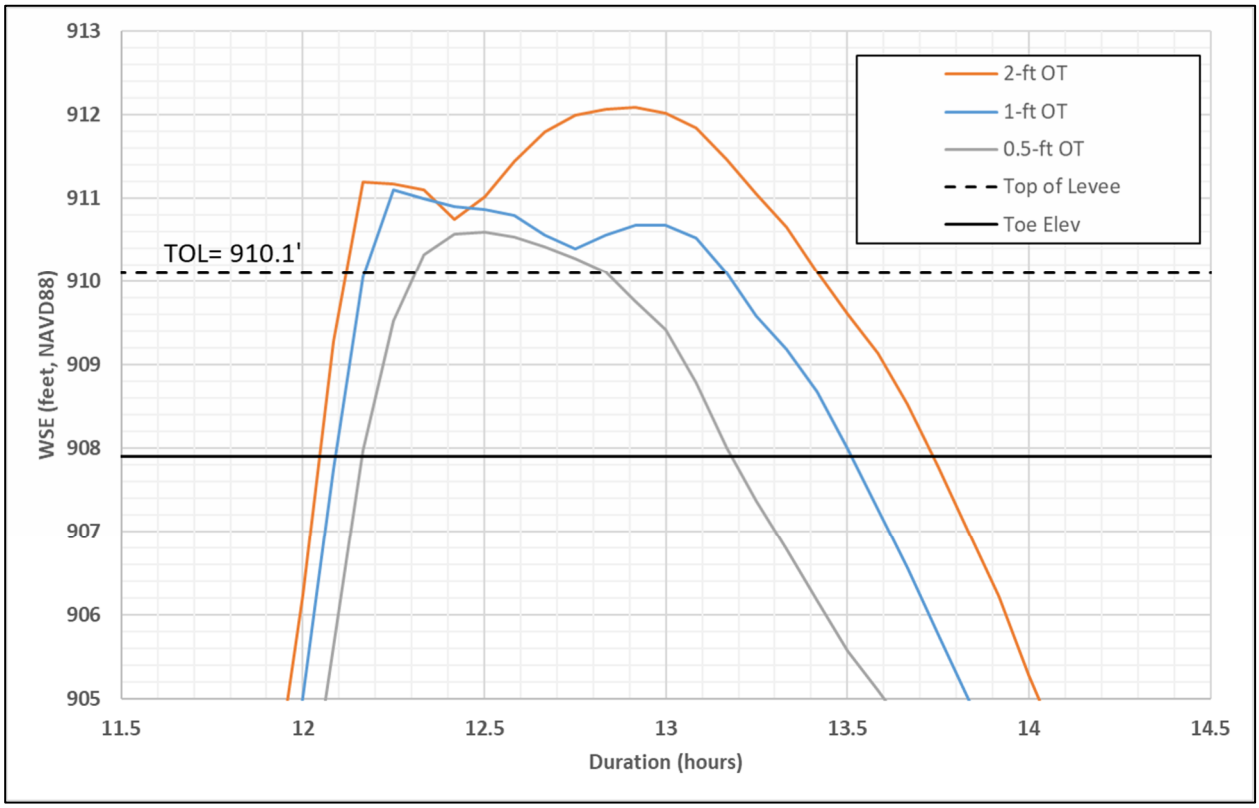


Figure 41. Expected overtopping durations at LCL based on routing the scaled synthetic hydrographs in the model.

4.4.8. Outflanking

Based on the current design, there is some slight outflanking on the left bank downstream end of the proposed levee system. The levee ends without tying into any high ground. This is due to the recently recomputed levee profile which may require some realignment to find appropriate high ground. Outflanking at this location seems to occur at approximately the same time as overtopping at Merriam Bridge on the left bank as seen in the below figure. The outflanking is not expected to contribute much to incremental consequences given that the flow mainly spills onto Merriam Drive and not the surrounding elevated properties.

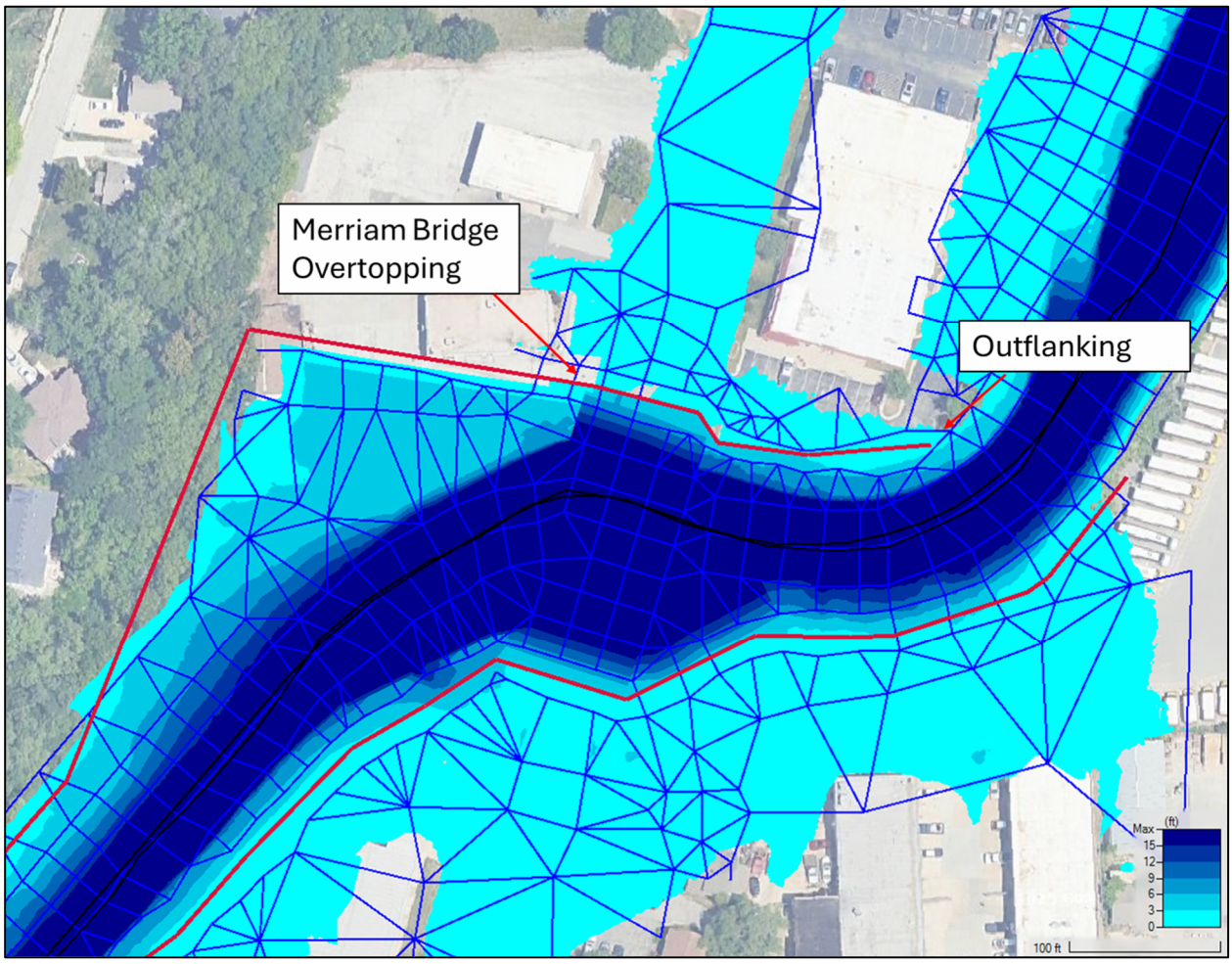


Figure 42. Outflanking of left bank floodwall at end of project.

4.5. Supporting Analyses

4.5.1. Riverine Velocity Estimates

Riverine velocity ranges from approximately 6 to 18.5 feet per second along the channel centerline for the top of levee loading scenario. Locations of higher velocities are generally located at pinch points where the levee alignment on the left and right banks narrows down. Areas of low velocities exist around bridges where flow is backed up against the upstream bridge face as well as the portion of the channel between Johnson and Merriam drive bridges where there is no left bank levee or floodwall.

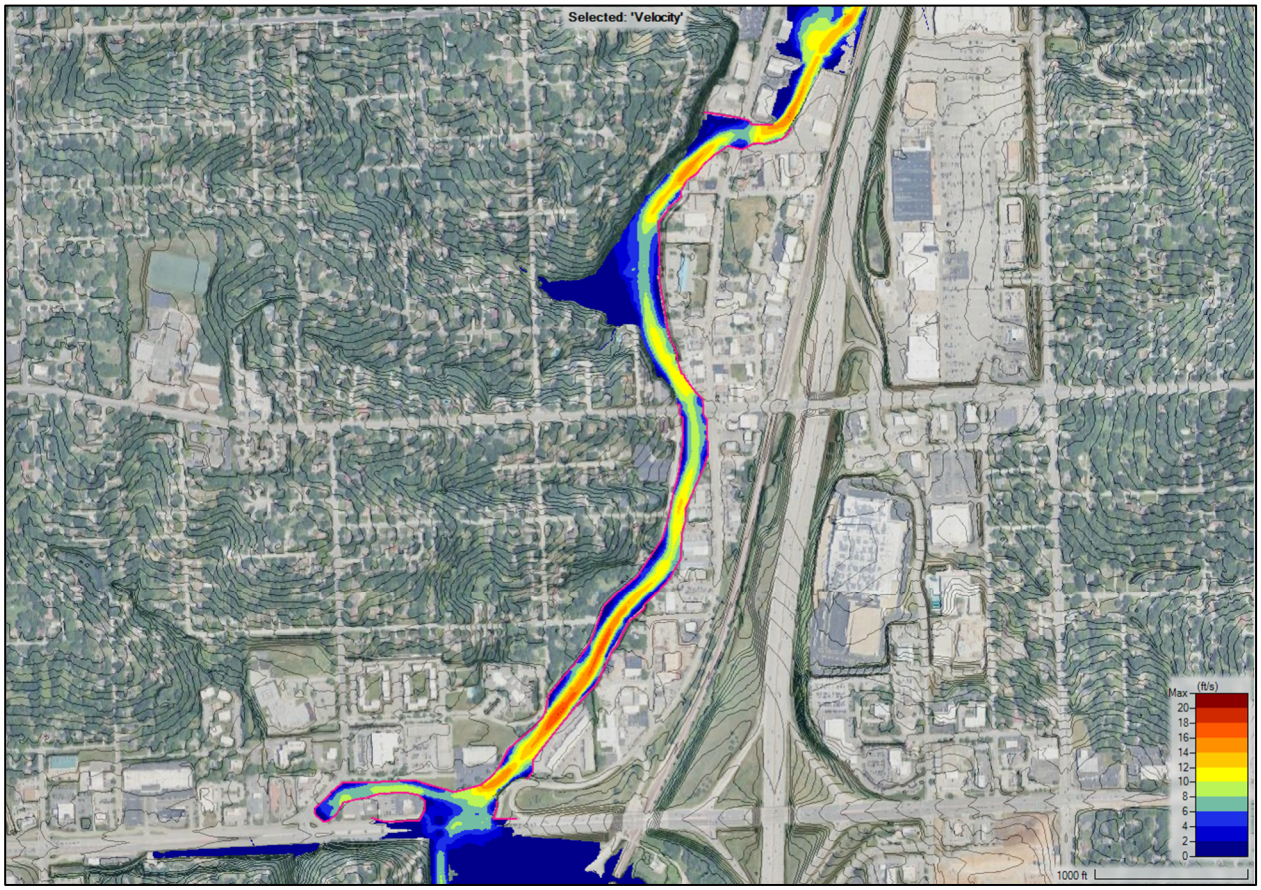


Figure 43. Peak velocity mapping for top of levee loading scenario.

Velocities increased with increasing overtopping depth scenarios. Most notable magnitude change in velocities were around river station 26+00 to 41+00 as shown in Figure 44. Peak velocity and velocity locations are shown in Table 23. Like the top of levee scenario, max velocities are occurring in straightaways in the channel near pinch points in flow conveyance area, or more specifically, top width. Outside bends do not seem to be a driving factor of max velocity locations, likely because the channel is already so constrained by levees and/or floodwalls on either side. Table 24 also provides a summary of peak and average increase in velocity for the overtopping scenarios when compared to top of levee loading. These locations of max increase are likely because the top of bridge decks begin to overtop and the damming effect of the upstream face of the bridges diminishes.

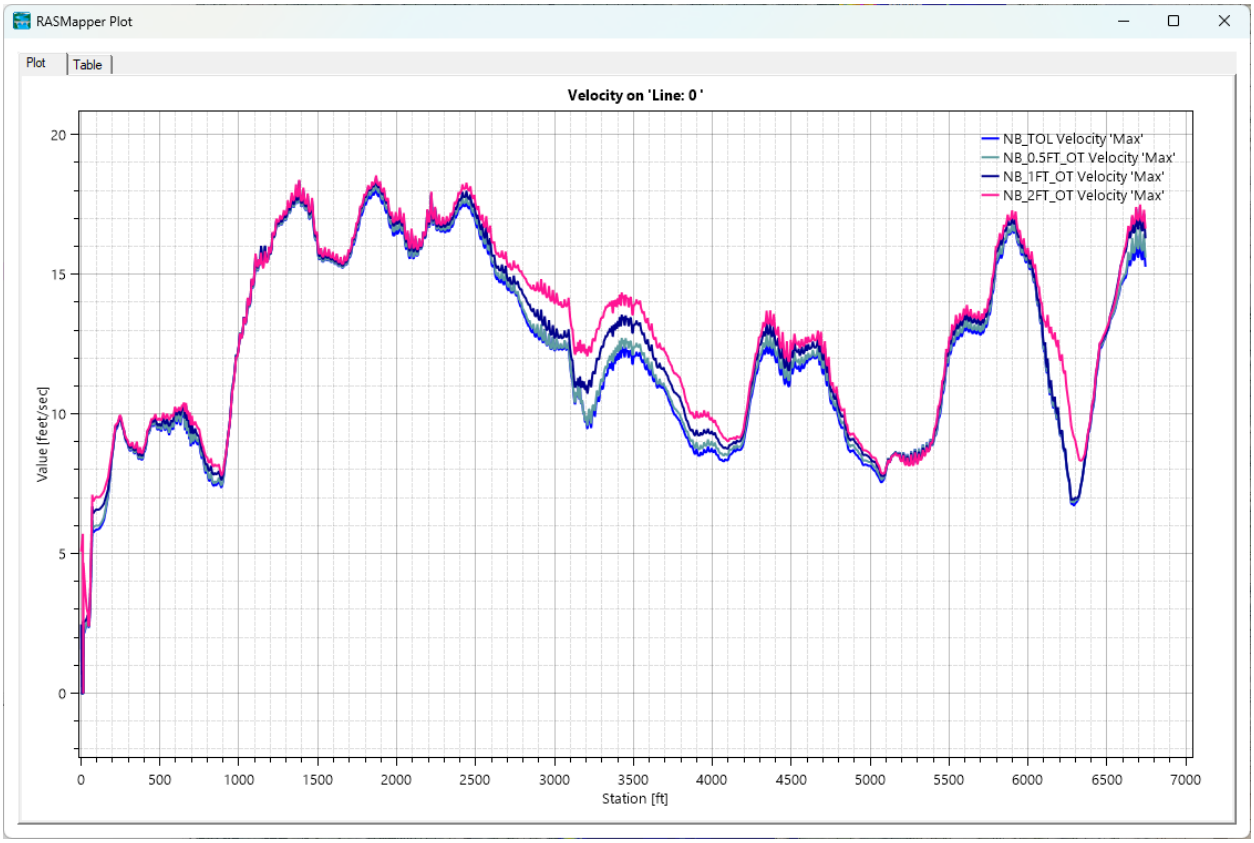


Figure 44. Velocity profile plot for loading scenarios.

Table 23. Peak velocity and location for loading scenarios.

Loading Scenario	Peak Velocity (ft/s)	Location of Peak Velocity (STA, feet)	Description
TOL	18.2	13+80	Constriction point immediately upstream of Shawnee Mission Parkway pedestrian bridge
0.5ft OT	18.3	18+70	Constriction point approximately 400’ downstream of Shawnee Mission Parkway pedestrian bridge
1ft OT	18.5	18+70	
2ft OT	18.5	18+70	

Table 24. Average and peak change in velocity from top of levee loading for overtopping scenarios.

Loading Scenario	Average Change in Velocity (ft/s)	Peak Change in Velocity (ft/s)	Location of Peak Change in Velocity (STA, feet)
0.5ft OT	0.1	0.3	34+80 (~700’ upstream of Johnson Drive)
1ft OT	0.4	1.5	32+60 (~900’ upstream of Johnson Drive)
2ft OT	0.8	2.8	62+40 (60’ upstream of Merriam Drive)

4.5.2. Wind-Wave Estimates

There is no known history of wind-wave estimates along Turkey Creek. Due to the short loading durations and general topography of the area preventing suitable fetch distances, wind-wave action is not considered a concern.

4.5.3. Overtopping Velocities

The LCL is located along a floodwall with asphalt pavement on the landside toe. Because of this, velocities or plunge scour were not estimated at this location. For the purposes of the SQRA, overtopping velocities were estimated at a representative levee section of 10 feet tall. These are used to estimate grass stripping on the landside of the levee for breach modeling and elicitation of levee overtopping failure modes. Breach locations are shown below in Figure 45. Breach locations 2 and 3 are at levees on the left and right bank, respectively. Overtopping depths and velocities are summarized in Table 28. Justifications for the chosen breach locations are discussed in Chapter 6.



Figure 45. Breach locations.

Table 25. Overtopping depths and normal depth velocities at 10-foot levee section.

Loading Scenario	AEP (1/year)	Peak Velocity (ft/s)	Velocity at landside edge of crest (ft/s)
0.5 ft OT	5.56E-03 (1/180-year)	8.3	2.8
1 ft OT	4.00E-03 (1/250-year)	12.6	4.0

2 ft OT	2.13E-03 (1/470-year)	18.7	5.7

Normal depth velocities were calculated to be used for estimating grass stripping. The critical flow was computed at the landside edge of the levee crest to help inform erosive energy at the crest for potential failure mode analysis. Table 29 below summarizes the parameters and assumptions used to compute the overtopping velocities.

Table 26. Overtopping velocity estimation parameters

Parameter	AEP (1/year)
Levee Height (ft)	10
Crest Width (ft)	10
Levee Slope (ft/ft)	0.33
Slope Roughness	0.02
Crest Roughness	0.03
Weir Coefficient	2

4.5.4. Interior Drainage and Ponding

Current designs for interior drainage include a detention pond and numerous gravity drainage structures along the left and right banks of the levee system to prevent ponding of the interior. Pump stations are not being considered in the design due to the flashiness of the system and the general understanding that there is not enough warning time get personnel there in time to operate the pumps. Interior drainage is being modeled separately from the levee design channel hydraulics.

The current design of the detention pond is sized for a 0.1 AEP event, which was part of the recommended design plan from the feasibility study. The detention pond layout and depth is still being refined, but the current size was implemented into this geometry as a terrain modification so it could be considered during overtopping and breach scenarios of the levee and/or floodwalls. An outlet structure will drain any ponded water through the levee and into Turkey Creek.

The drainage structures are all gravity drains with passive flap gate closures. Pipes with diameters of 48 inches or above currently are designed with a dual closure system. The second closure being another passive system (flap gate or duckbill) inside the gate well. Figure 46 details the locations of the currently proposed drainage structure locations and detention pond. An active secondary closure such as a sluice gate is not likely to be beneficial, again because of the flashiness of the watershed. It is unlikely personnel would be able to get out in time to close a sluice gate. Additionally, the interior area is likely to experience the same storm event that is driving the riverine flooding, and improper timing of sluice gate closure runs the risk of worsening interior ponding if it is not allowed to drain under gravity conditions.



Figure 46. Proposed drainage structure locations.

Interior drainage capabilities are not part of the model used for the SQRA; therefore, it was not included in the modeling. The assumption for this SQRA model is that any overtopping or breach scenarios will lead to ponding of the interior under gravity denied conditions for the duration of event. The detention pond was assumed to be empty for all loadings and breach scenarios so that it can be shown how overtopping flow or breach flow drains into the pond, potentially mitigating life safety or economic consequences. It should be noted a storm event capable of producing overtopping loading will also cause significant interior flooding in the project area. Modeling coincident interior flooding and channel flooding would be beyond the scope of this assessment.

It should be noted that a design criteria waiver is being pursued for the dual closure requirement for drainage structures larger than 36 inches. This would result in just a single flap gate closure on the riverside of the flood protection system, similar to drainage structures less than 36 inches in diameter. Below are two examples of larger diameter drainage structures with a single flap gate being stuck fully open for the full duration of a top of levee loading event. The first is drainage structure ST-15 (48-inch diameter RCP) on the left bank, shown in Figure 47. The second is drainage structure ST-20 (48-inch diameter RCP) on the right bank just upstream of Johnson Drive bridge, shown in Figure 48.

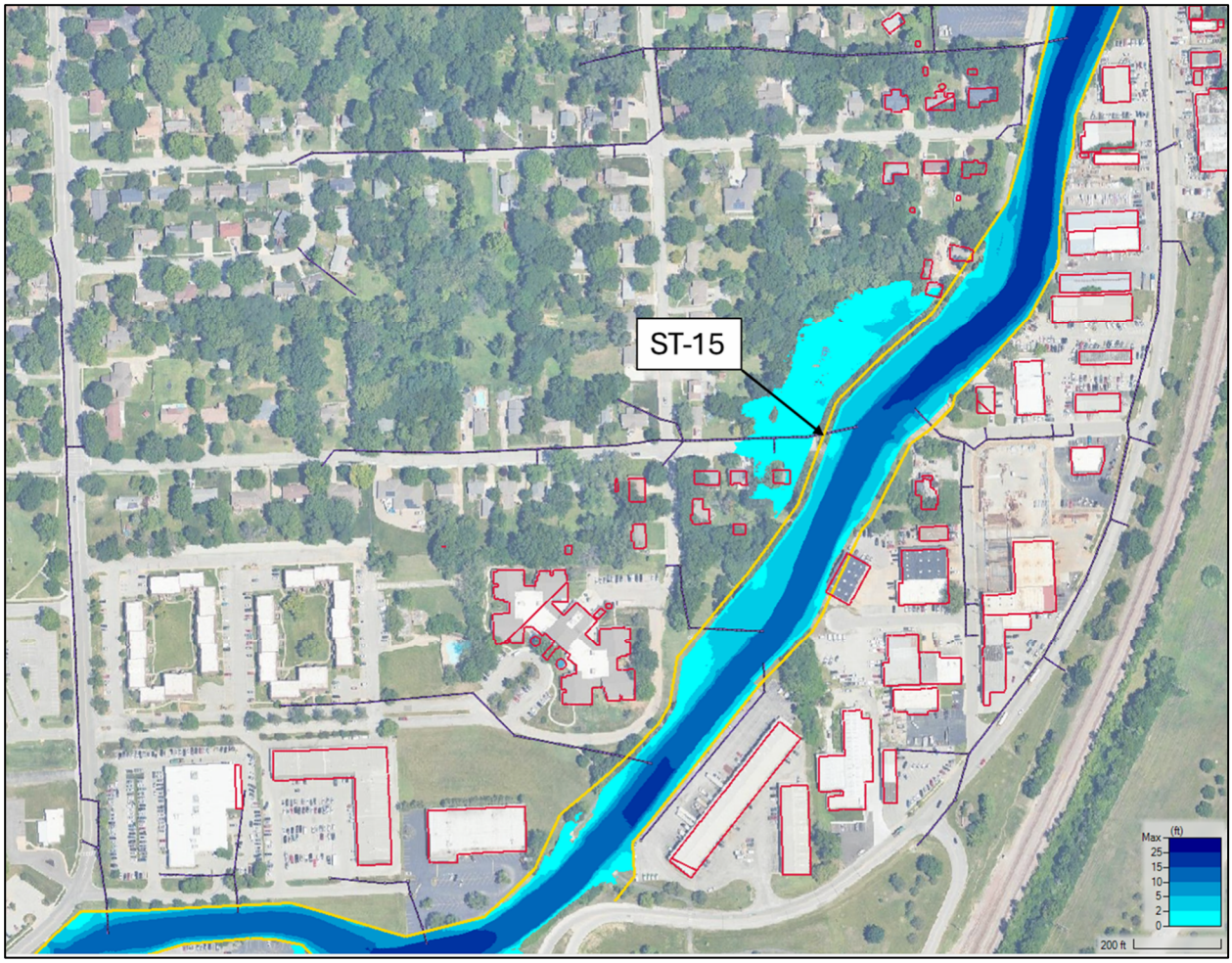


Figure 47. Interior flooding with drainage structure ST-15 flap gate stuck fully open.

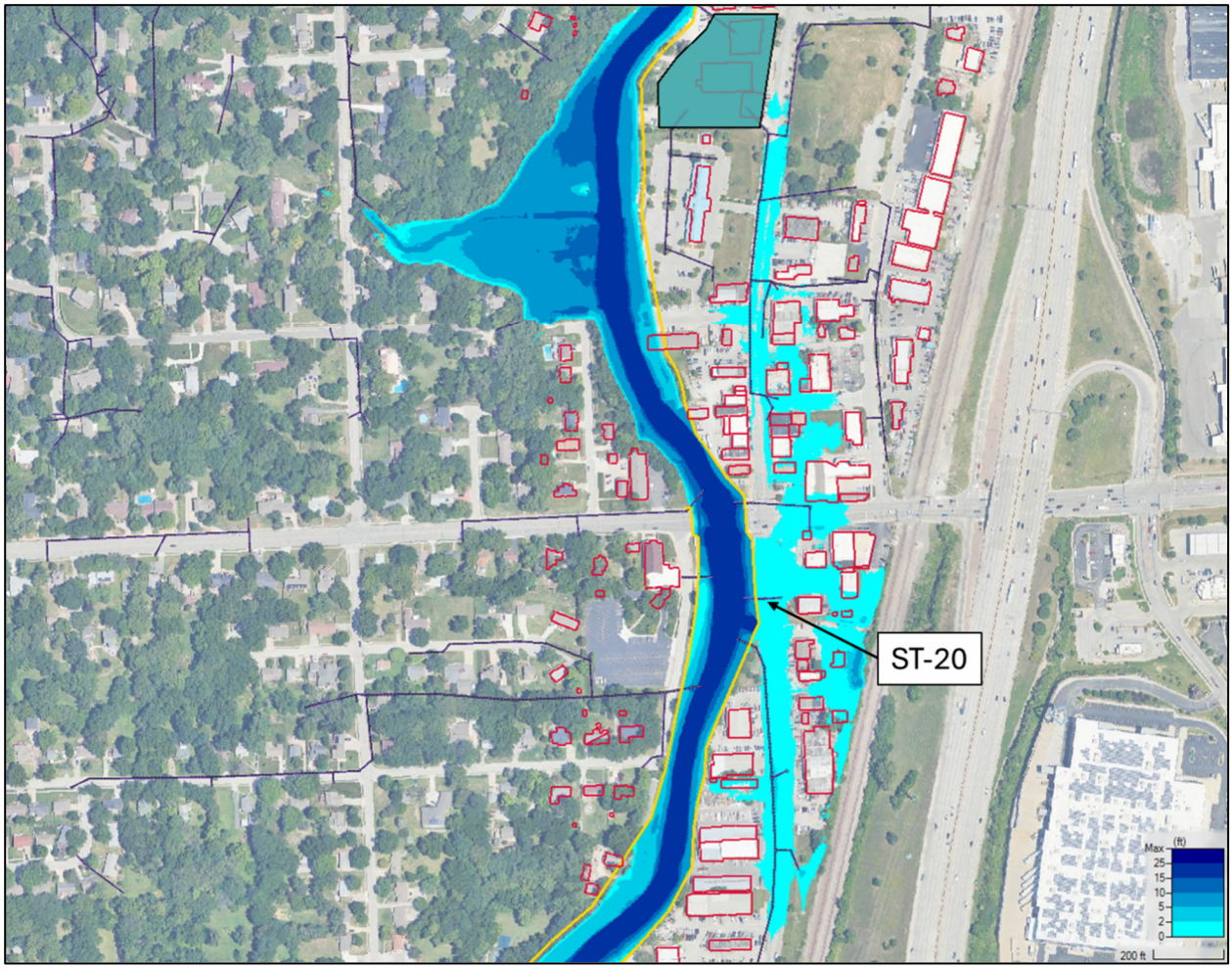


Figure 48. Interior flooding with drainage structure ST-20 flap gate stuck fully open.

Inundation depths in both scenarios are almost entirely less than 2 feet deep, which poses little risk of life loss. Inundation is mainly contained to park land, streets, and parking lots, with some buildings seeing inundation of less than 2 feet.

4.5.5. Bridges and Inline Structures

Bridges are modeled as 2D connections in the HEC-RAS model. The bridges have an impact on the hydraulic profile depending on the flow scenario. It should be noted that Johnson Drive and Merriam Drive bridges are planned to be replaced as part of the design and are being implemented into the flood protection system. The current design has these two bridges being replaced and raised so that they do not overtop during the 0.01 AEP (1/100yr) event. This allows the project design team to remove the need for closure structures such as stoplog gaps or swing gates on either side of the bridges. The reason for not including closure structures on the bridges is due to the flashiness of the system and the low likelihood of having personnel out to the system in time to operate the closures. Additionally, there are two pedestrian bridges located within the project extents. These are not being removed or replaced but do influence the hydraulic profile and velocities as previously discussed. Figure 50 below details the locations of the bridges within the project extents.

It was assumed that any guardrails or pedestrian rails on the upstream and downstream faces of the bridges would not convey flow. This was to account for uncertainty in the flow conveyance capacity that the spaces between railing would present as well as uncertainty in debris blockage that would occur. Bridge dimensions are based either on USACE survey data or dimensions provided for current replacement bridge designs.

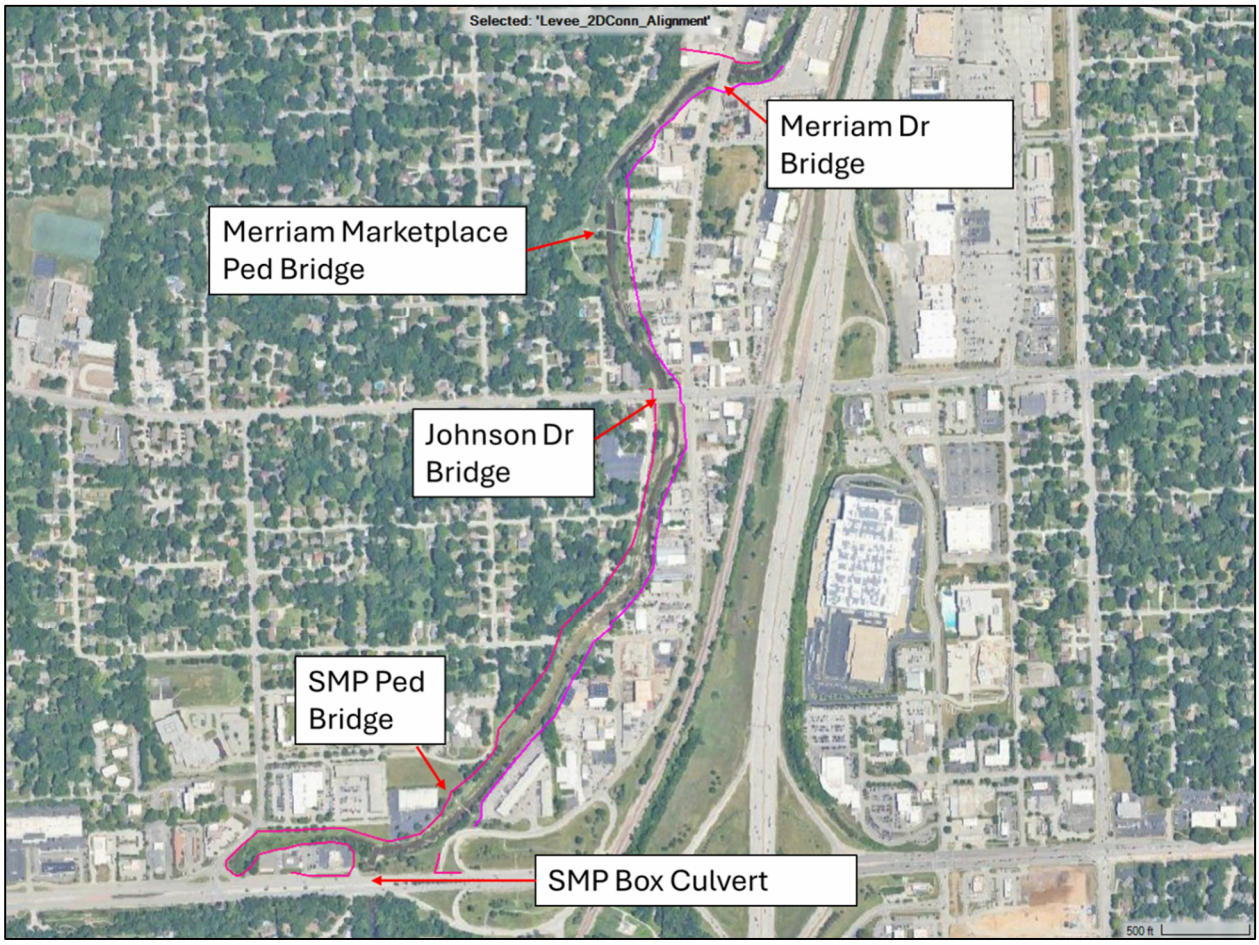


Figure 49. Bridge locations.

4.5.6. Ice and Debris Effects

No incidents of ice or debris jams have been noted in previous flood documentation. Based on historic documentation, flooding generally occurs during non-freezing months of the year.

4.5.7. Sediment Impacts

After the August 2017 flood, a survey of sediment deposits was conducted in November 2017 and again in the summer of 2018. This was used to update the Southwest Boulevard rating curve for the Lower Turkey Creek project. These updates and survey data were included in the Turkey Creek 2018 Sediment Analysis Report. The report did not include any analysis of the upper portion of Turkey Creek, instead looking at three primary reaches of lower Turkey Creek. The restored channel and walled channel reaches have experienced sediment deposition while the natural reach immediately upstream experienced primarily erosion.

For upper Turkey Creek, a significant portion of the channel upstream of the project extents and the extents themselves are lined with block wall channels. This likely reduces the magnitude of erosion and sediment transport to the project area that the lower portions of Turkey Creek experience, both in typical flows over time and flood events.

The tributary just south of Shawnee Mission Parkway appears to have experienced notable sediment deposition between 2010 and 2023 (and beyond) as seen in Figure 50 and Figure 51 below. The LiDAR for the model terrain was flown in 2020 and generally reflects the current sediment deposition at this location. Increased deposition such as the current conditions of the tributary lead to increased stages. Clearing out the channel would lead to improved conveyance capacity but would likely sediment back in over time unless added in as an operations and maintenance item. Similar issues are present further downstream near the Merriam Marketplace pedestrian bridge.

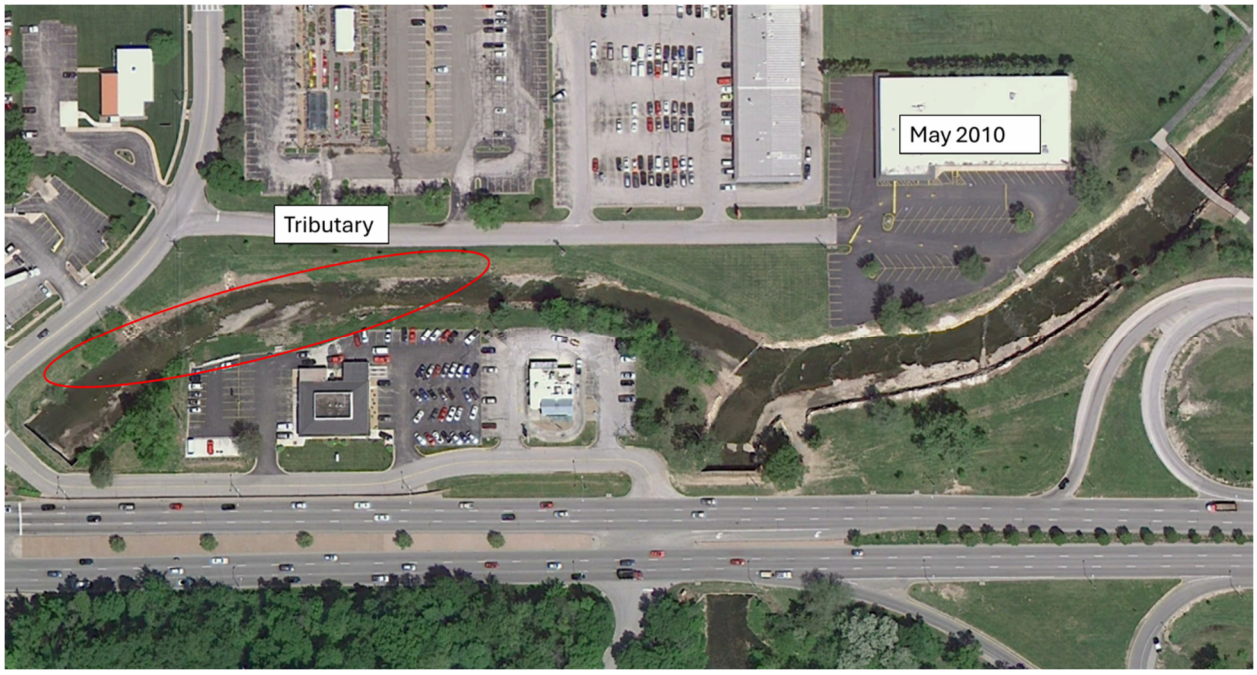


Figure 50. May 2010 Aerial of tributary sediment deposition.

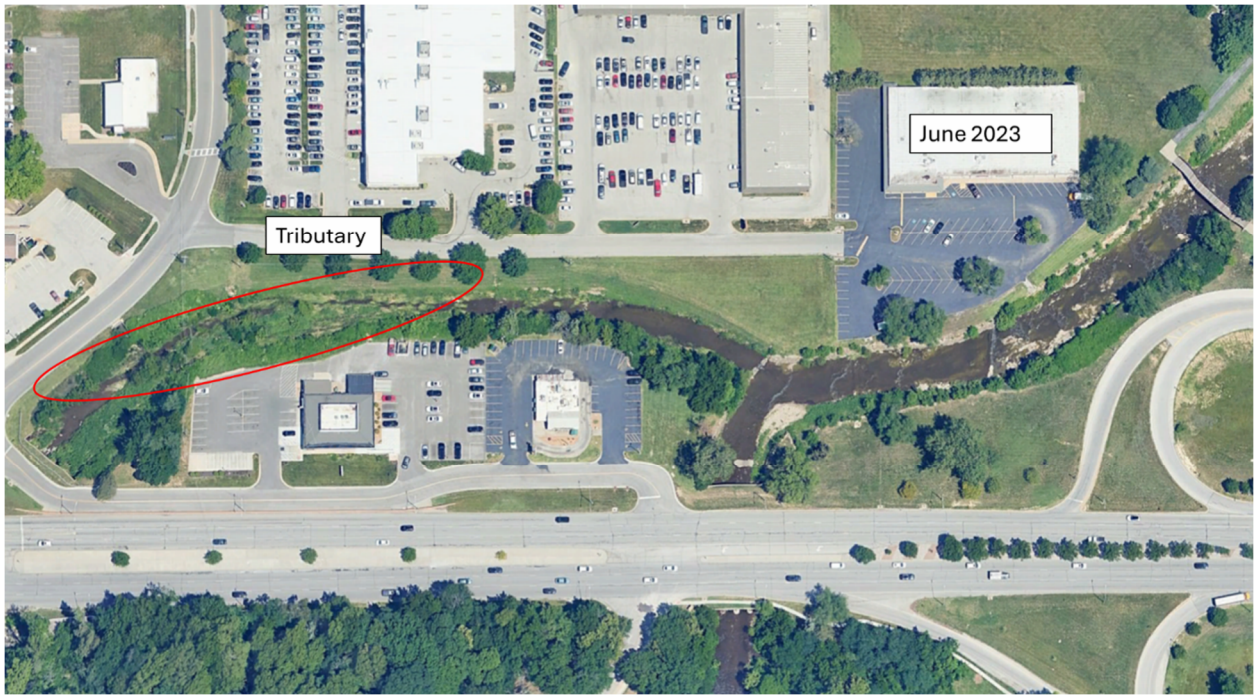


Figure 51. June 2023 aerial of tributary sediment deposition.

4.6. Major Uncertainties

There is major uncertainty in the hydrology due to a lack of consistent flow records anywhere along Turkey Creek, as previously discussed. The gage at Johnson Drive within the project area is not a USGS gage and does not record flow data, nor does it have an accompanying stage-discharge rating curve. Thus, an RMC-BestFit, flow frequency analysis is not possible to perform at this location. Instead, frequency flows (inflows) were developed using HEC-1, a hydrologic modeling software during the feasibility study, with Atlas 14 frequency precipitation depths for the 24-hour storm that would then be routed through the developed hydraulic model. Uncertainty is present in the duration and distribution of the frequency storm used in HEC-1 as this is often subjective and/or based on regional trends. Additionally, the Turkey Creek watershed is heavily urbanized and fully accounting for underground stormwater network routing and overland runoff is nearly impossible without significant additional data and time, which is out of the scope of this study.

There is major uncertainty in the hydraulic modeling because of a lack of useable calibration data. Despite the base HEC-RAS model being set up for 2D rain-on-grid modeling, there are significant limitations to its ability to route storm network flow which could drastically affect routing timings and therefore affect peak flow magnitude, timing, and hydrograph shapes in the Turkey Creek channel. Using historical gridded precipitation to calibrate historic storms may lead to unrealistic calibration parameters by compensating for the stormwater network routing limitations. For this reason, best engineering judgement was used for selection of the calibration parameters while leaning towards more conservative modeling assumptions to account for hydraulic uncertainty in the design. Recent considerations were given to simulating 2017 storm events in the HEC-1 model, if possible, to compute inflows for the current set up of the HEC-RAS model and calibration efforts. But this was beyond the time allotted and scope of this SQRA. Highwater marks from the 2017 flood are also available for calibration purposes should these historic inflows be computed in the future. But again, there is significant uncertainty in the historic flows and if they are representative of current conditions.

5. Seismic Hazards

5.1. Background

The levee is located at latitude 39.027318º, longitude -94.693427º in the central and eastern western United States. The 2023 update of the National Seismic Hazard Model (NSHM) by the U.S. Geological Survey (USGS) (Petersen et al. 2020) provides consensus-based regional analyses of seismic hazard, using Central and Eastern United States (CEUS) seismic sources, Western United States (WUS) seismic sources, combined geologic/geodetic information, and revised ground-motion models. This report represents the most current assessment of seismic hazard for large regions of North America. The 2023 NSHM declustered seismicity catalog is shown in Figure 52.

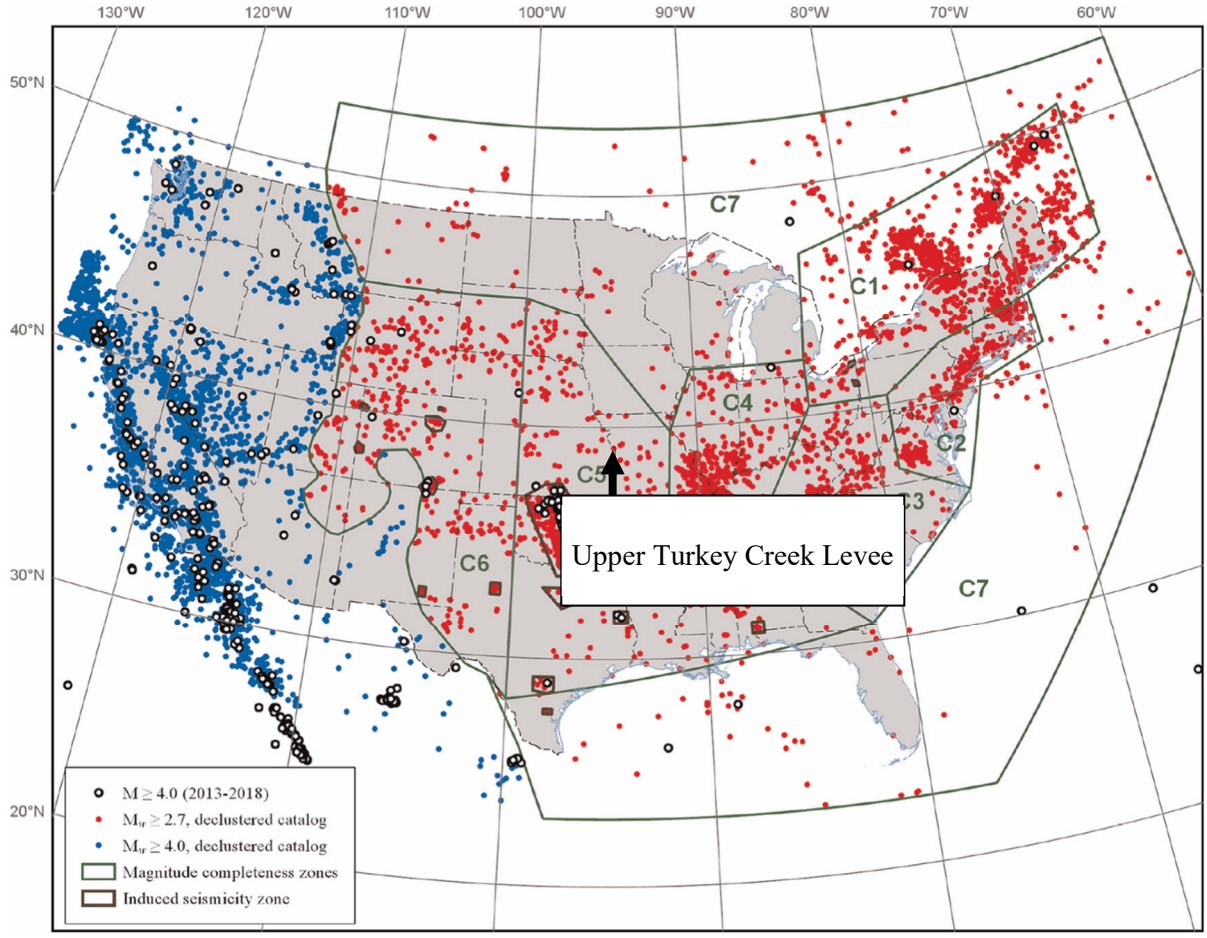


Figure 52. 2018 NSHM declustered seismicity catalogs for $M_w \geq 2.7$ (Petersen et al. 2020)

The NSHM for the CEUS considers both seismicity-based background earthquake source zones and fault-based sources and utilized data and models from the CEUS Seismic Source Characterization for Nuclear Facilities (CEUS–SSCn) project, which accounts for broader uncertainties and replaces older seismic source models. The seismicity-based source model for the CEUS assumes that future large earthquakes are more likely to nucleate near previous earthquakes with moment magnitude (M_w) greater than or equal to 2.7. The model also distinguished seismotectonic zones in the CEUS with distinct seismicity and

maximum earthquake magnitudes, in order to accommodate some possibility that the historical seismicity does not fully represent likely sources of background earthquakes.

The dam lies in the "North American Craton" seismotectonic zone but is less than about 520 kilometers west of the "Paleozoic Rift" seismotectonic zone, as shown in Figure 53, and could be affected by strong ground shaking resulting from earthquakes in either of these zones. The two seismotectonic zones have slightly different rates of seismicity occurrence, as well as interpreted maximum earthquake magnitudes. Peterson et al. (2014) indicate that it is reasonable to expect an earthquake with M_w between 6.5 and 7.8 could happen in the craton.

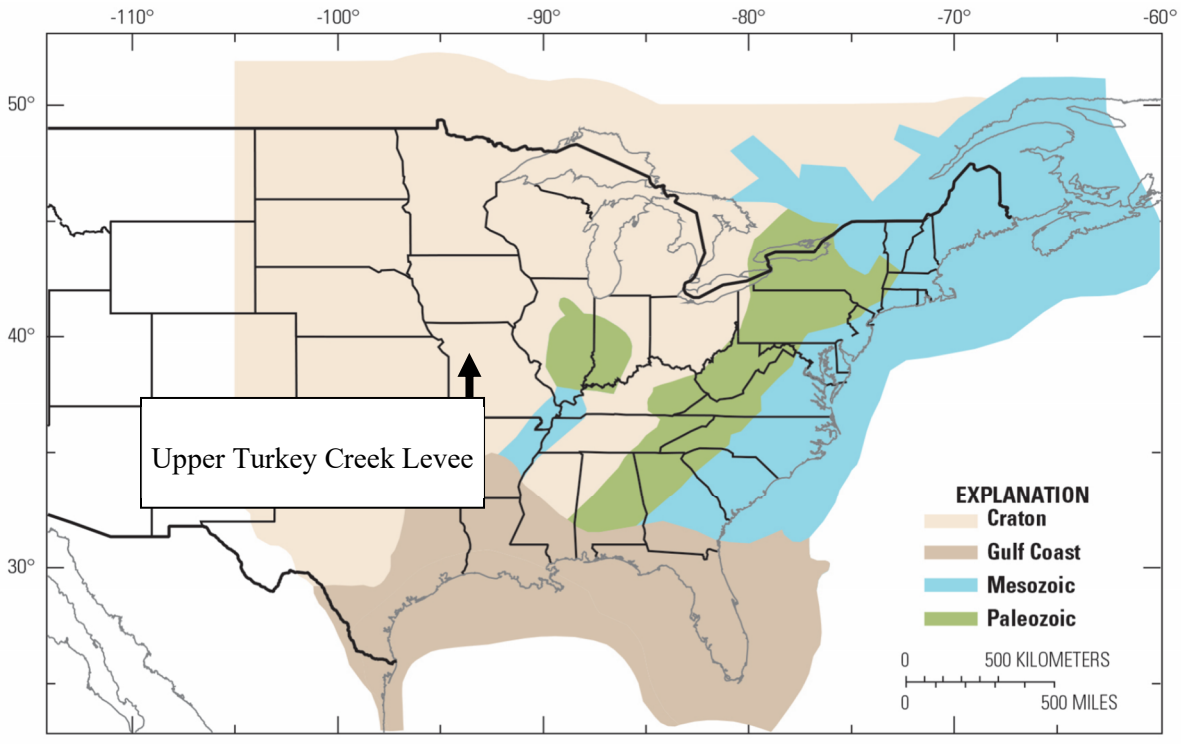


Figure 53. 2018 Generalized seismotectonic zonation of the CEUS (CEUS-SSCn 2012)

The USGS Quaternary Fault and Fold Database of the United States and interactive fault map (https://doi.org/10.5066/F7S75FJM) were reviewed for faults and associated folds in the vicinity of the site that demonstrate geological evidence of coseismic surface deformation in large earthquakes during the past 2.58 million years (Myr). The results are portrayed in Figure 54 and summarized in Table 27.

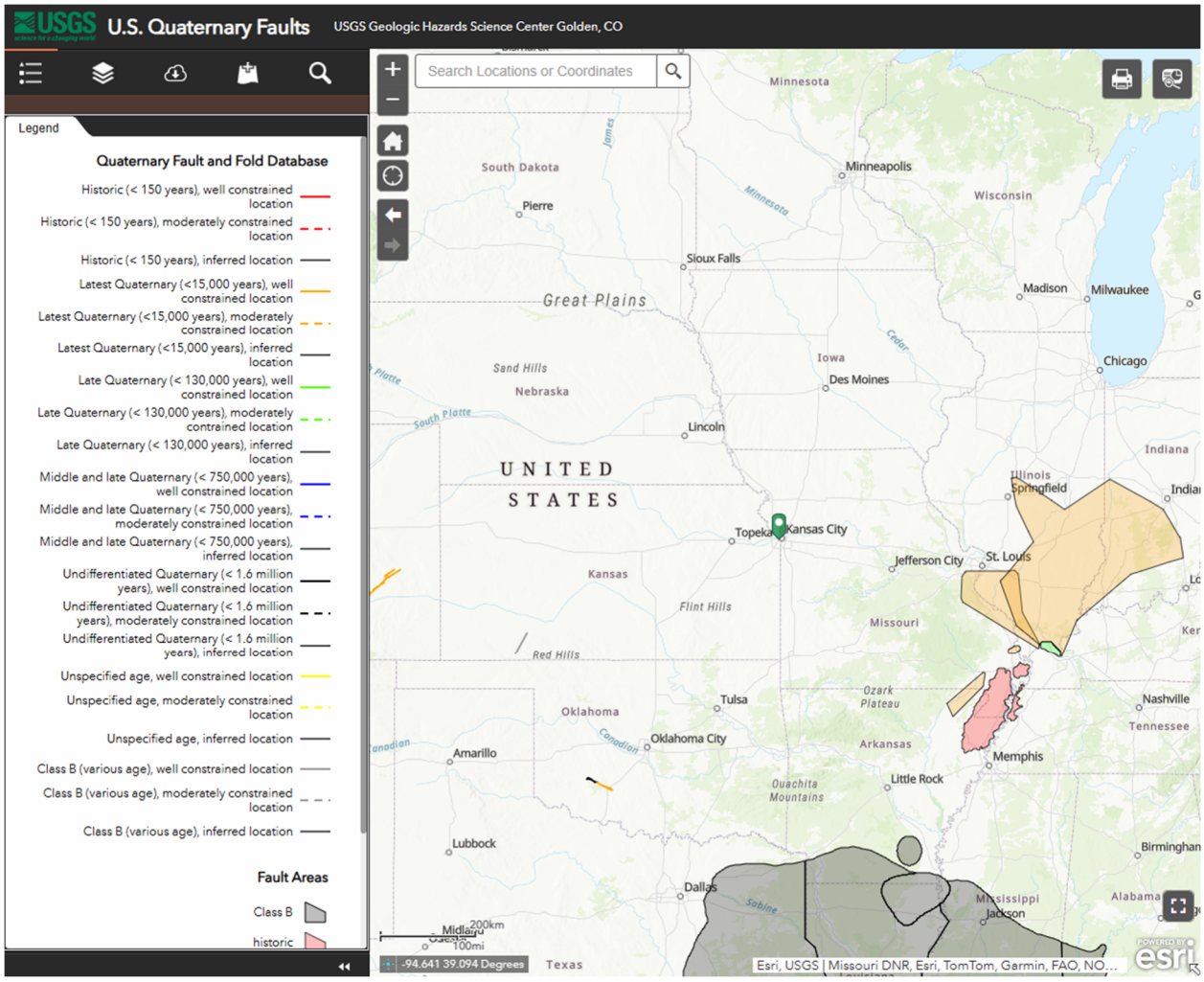


Figure 54. Quaternary faults (USGS 2025)

Table 27.
Quaternary faults (USGS).

Name	Primary County, State	Length (km)	Time of Most Recent Deformation	Slip-Rate Category
St Louis-Cape Girardeau Liquefaction Features	Multiple Counties in Missouri and Illinois	542 (perimeter)	<15 ka	No data

NOTE: https://doi.org/10.5066/F7S75FJM accessed on 15 APR 2025.

5.2. Historical Seismicity

The USGS Earthquake Catalog was searched for events with magnitude (M) greater than or equal to M4.5. Figure 55 summarizes these events with respect to magnitude and epicentral distance. Table 28 provides details of some selected earthquakes of interest.

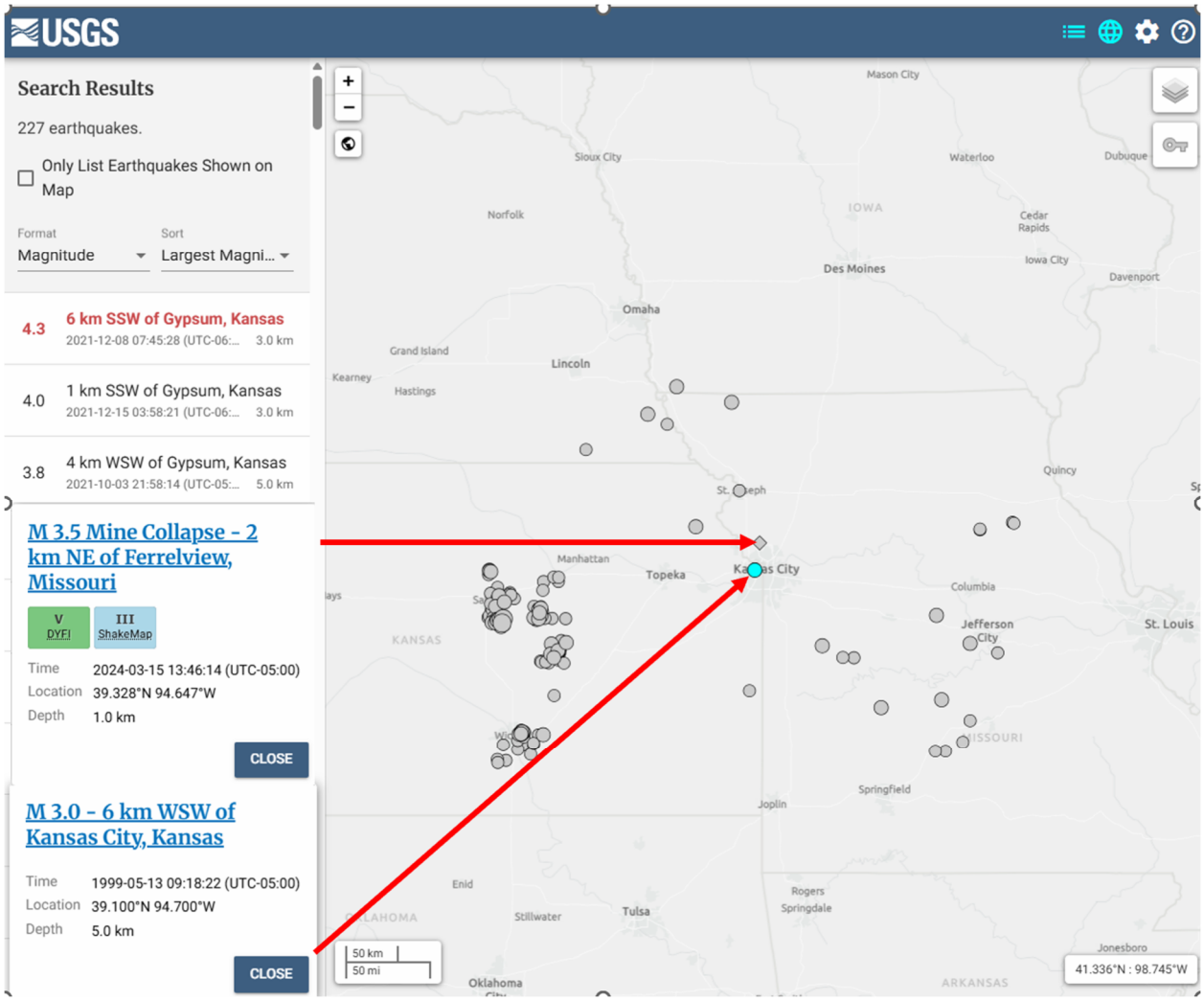


Figure 55. Historical seismicity near the Upper Turkey Creek Project Area since 1960 (USGS 2025)

Table 28.
Historical seismicity since 15 APR 1960 (USGS).

Magnitude Interval	Search Radius from Site	Number of Events
4.5 ≤ M < 5	10 mi (16 km)	0
5 ≤ M < 6	50 mi (80 km)	0
6 ≤ M < 7	75 mi (121 km)	0
7 ≤ M < 8	125 mi (201 km)	0
M ≥ 8	200 mi (322 km)	0

NOTE: https://earthquake.usgs.gov/earthquakes/search/ accessed on 15 APR 2025.

Table 29.
Selected historical earthquakes with M4.5+ since 15 APR 1960 (USGS).

Date	Name	M	Latitude (deg)	Longitude (deg)	Depth (km)	Distance (km)
03 SEP 2016	Pawnee, Oklahoma	5.8	36.425	-96.929	5.6	335
20 OCT 1965	Centerville, Missouri	4.6	37.479	-90.944	7.0	375

NOTE: https://earthquake.usgs.gov/earthquakes/search/ accessed on 15 APR 2025.

5.3. Induced Seismicity

There are currently active deep injection wells for wastewater or other fluids within 20 kilometers of the site.

Increases in seismicity in the CEUS have been documented since about 2009. Most of the human-induced earthquake activity in the CEUS is caused by injection of wastewater or other fluids into deep disposal wells that causes pressure changes that can weaken a fault and therefore bring it closer to failure (Petersen et al. 2017). The seismic hazard may be higher in these areas of potentially induced seismicity than the hazard depicted in the NSHM. Assessing the hazard is difficult because induced seismicity can vary rapidly in time and space based on changes in wastewater injection which can be caused by economic or policy decisions (Petersen et al. 2016).

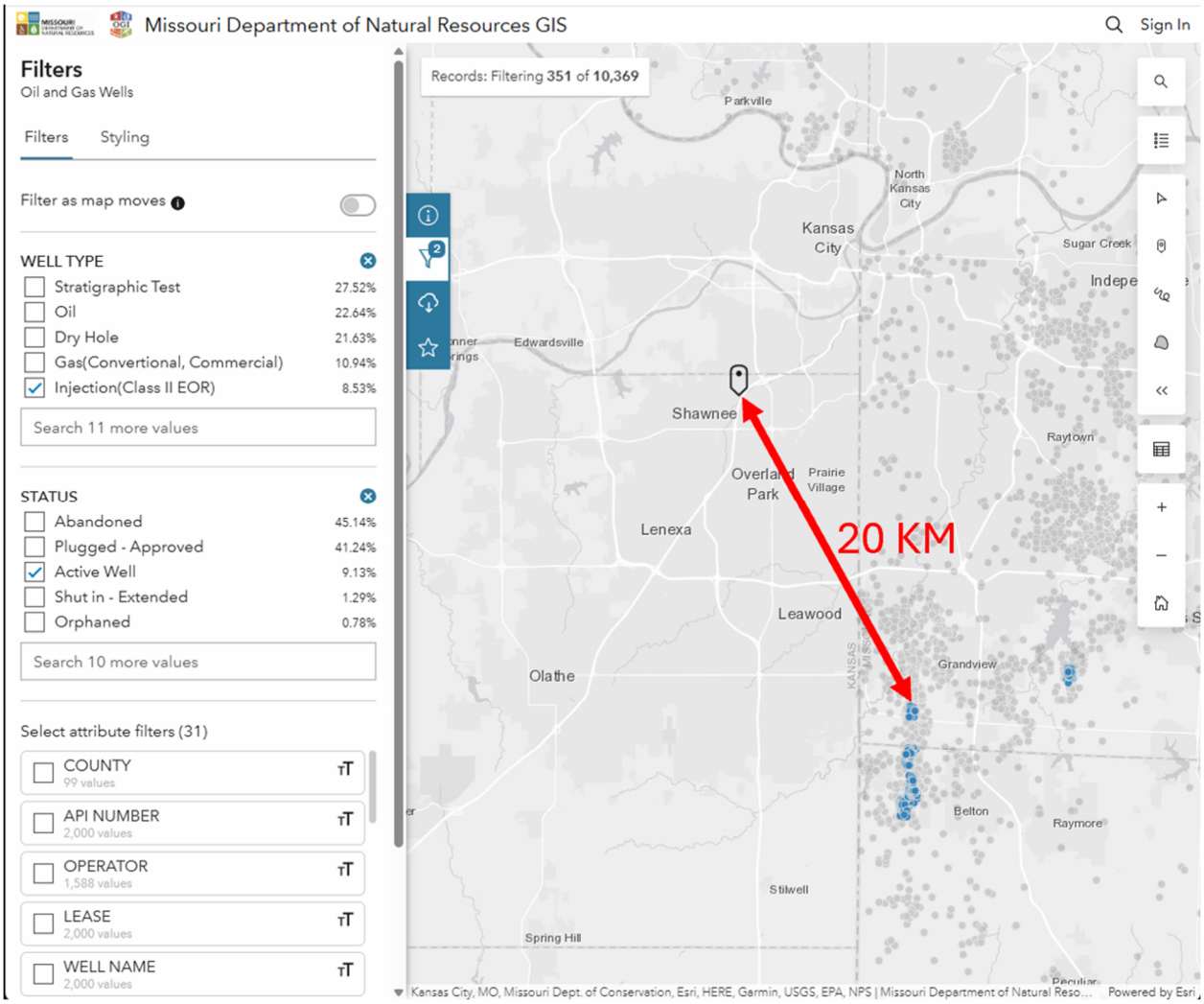


Figure 56. Injection Wells within 20 KM of the Upper Turkey Creek Project Area (MODNR 2025)

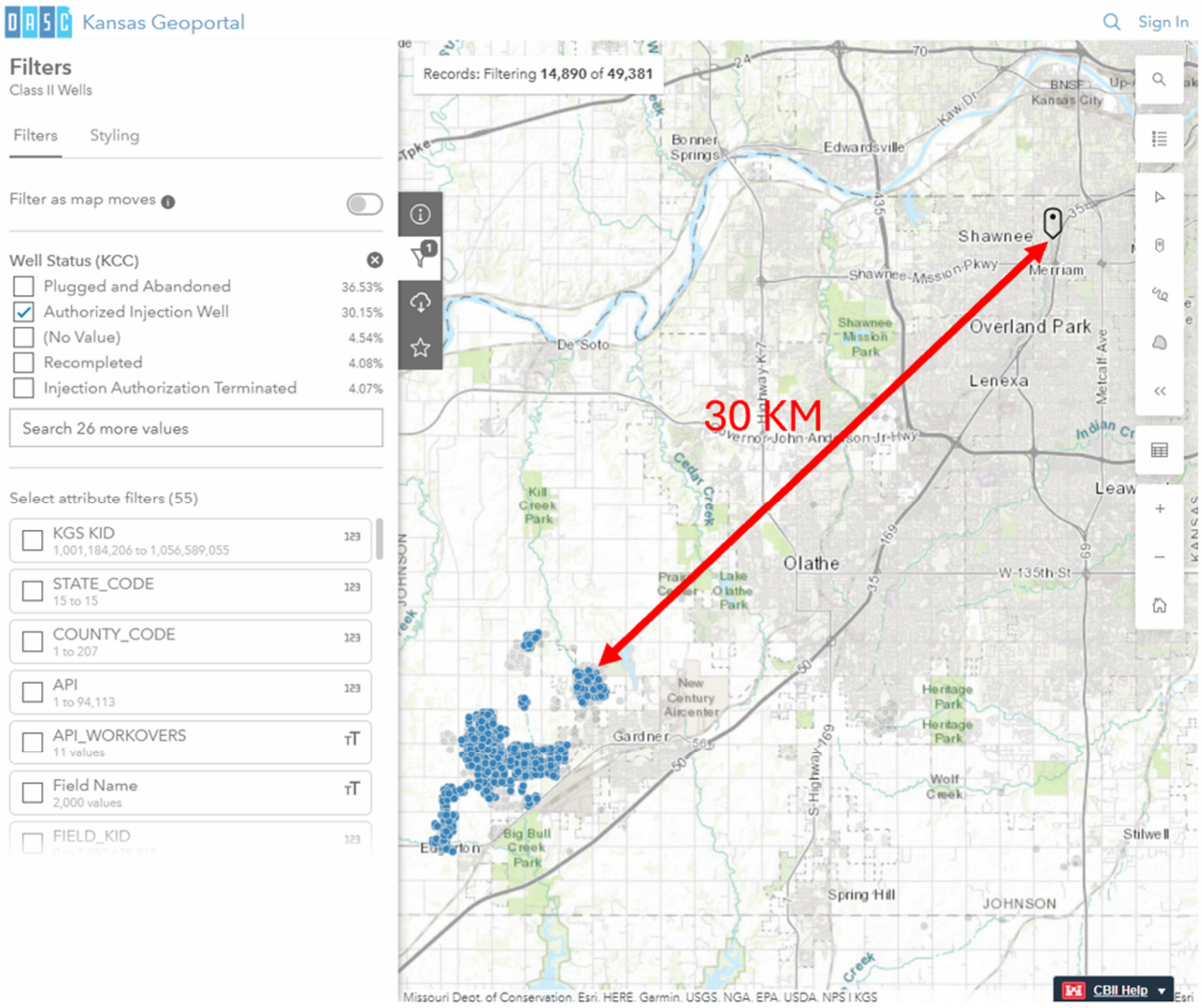


Figure 57. Injection Wells within 20 KM of the Upper Turkey Creek Project Area (KGS 2025)

5.4. Site Classification

The dam is founded on Pleistocene and Holocene alluvium that has a maximum thickness of about 8 to 18 feet over bedrock. The subsurface materials consist predominantly of clay and silt, with some gravel at the top of weathered bedrock.

The shear wave velocity (v_s) is the velocity that shear waves, such as those produced by an earthquake, will have as they pass through the soil or bedrock. For the purposes of estimating ground motions, the foundation soil and rock profile can be classified based on the average shear wave velocity in the uppermost 100 feet (30 meters, v_{s30}) of the site profile. The v_{s30} is currently the preferred parameter for characterizing (classifying) the site conditions in developing estimates of ground shaking using ground motion attenuation relationships.

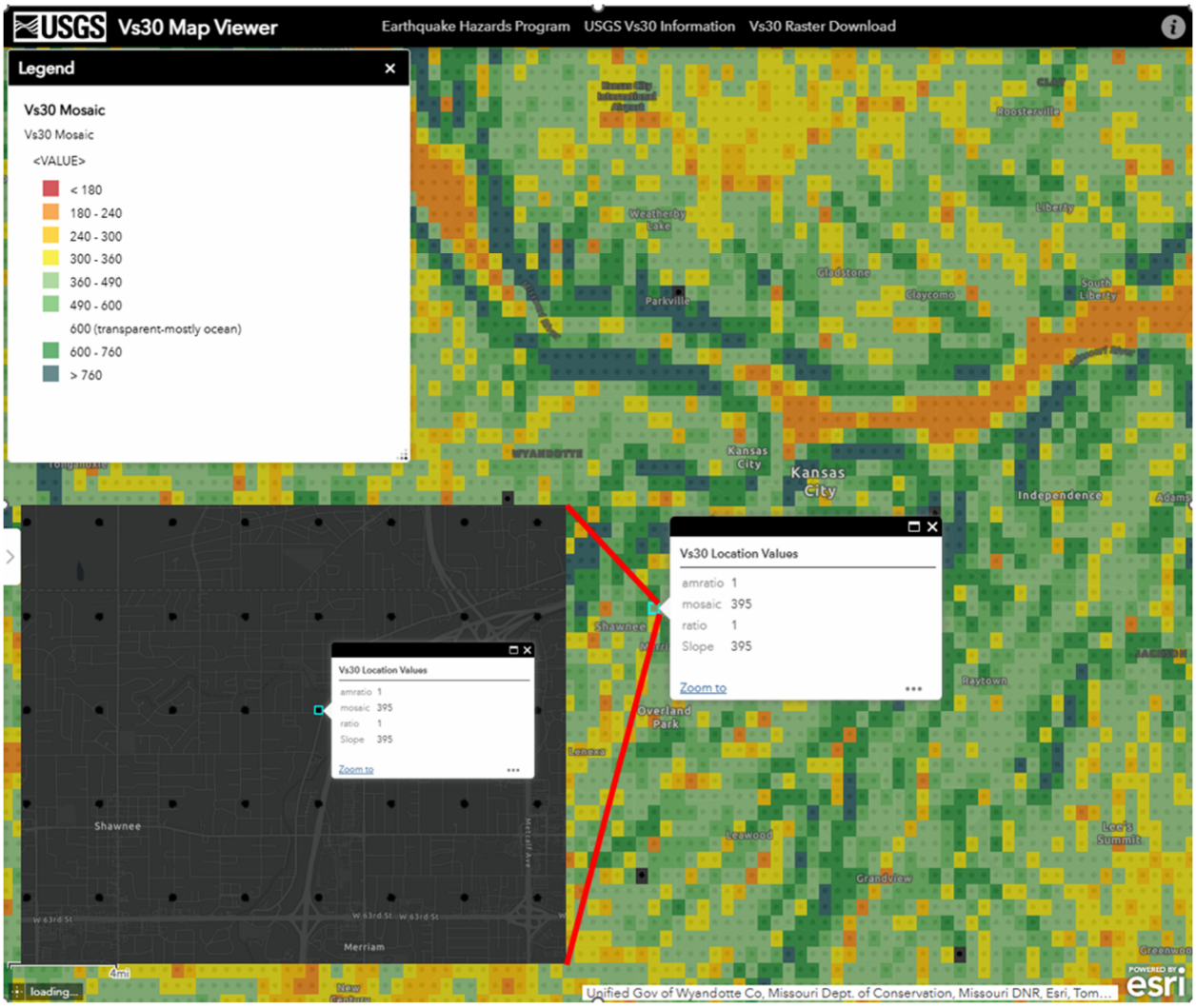


Figure 58. USGS Compilation Values of v_{s30} in the United States (McPhillips, D.F., Herrick, J.A., Ahdi, S., Yong, A.K., and Haefner, S., 2020, Updated Compilation of VS30 Data for the United States: U.S. Geological Survey data release, https://doi.org/10.5066/P9H5QEAC**).**

Shear wave velocity data was not available for the site. The USGS has compiled measured v_{s30} funded by the USGS and other governmental agencies for thousands of sites in the US (https://earthquake.usgs.gov/data/vs30/us/). The time-averaged shear-wave velocity (v_{s30}) of 1296 feet per second (ft/s) (395 meters per second (m/s)) was judged appropriate for estimating purposes for the site. Therefore, the foundation corresponds to Site Class CD, according to the National Earthquake Hazards Reduction Program (NEHRP), but Site Class D was still used as a conservative approach.

Shear wave velocity data was available for the site at the USGS website, but the value used on the website was not used. The overburden thickness at the valley center was at most 25 feet of lean clay and gravelly clay above bedrock. SPT field-measured blow counts (N_{ch}-values) of the 25 feet cohesionless soils typically ranged from 0 to 38 blows per foot (bpf), with an average of 8 bpf. Undrained shear strengths (s_u) of the 25 feet of cohesive soils ranged from 1043 to 2766 pounds per square foot (psf), with an average value of 1853 psf. The underlying rock generally consisted of hard, interbedded limestone and

sandstone, and the rock layers to a depth of 100 feet were assigned N_{ch} of 100 bpf or s_u of 5,000 psf. Since the average SPT blow count ($\overline{N}_{ch}$) was 8 bpf and the average undrained shear strength ($\bar{s}_u$) was 1853 psf, the foundation corresponds to Site Class D, according to the National Earthquake Hazards Reduction Program (NEHRP). Based on the $\overline{N}_{ch}$ of 8 bpf and $\bar{s}_u$ of 1853 psf the foundation corresponds to Site Classes E and D, respectively, according to the National Earthquake Hazards Reduction Program (NEHRP). Therefore, the site was assigned the softer site class of Site Class D. It should be noted the shear wave velocities of the shallow bedrock are unknown and only the upper 8-10 meters of the v_{s30} profile have experimental shear wave velocities and verifiable lab data. Additional data would be required to change the site class to verify the experimental data available from the USGS v_{s30} Map Viewer.

5.5. Seismic Hazard Curves

A local (site-specific) probabilistic seismic hazard analysis (PSHA) has not been performed for this project. The mean seismic hazard curve for the peak horizontal ground acceleration (PGA) and Site Class D was generated using the 2023 NSHM, as shown in Figure 59. The PGA corresponding to selected common values of return periods were interpolated from this mean hazard curve and are shown in Table 30.

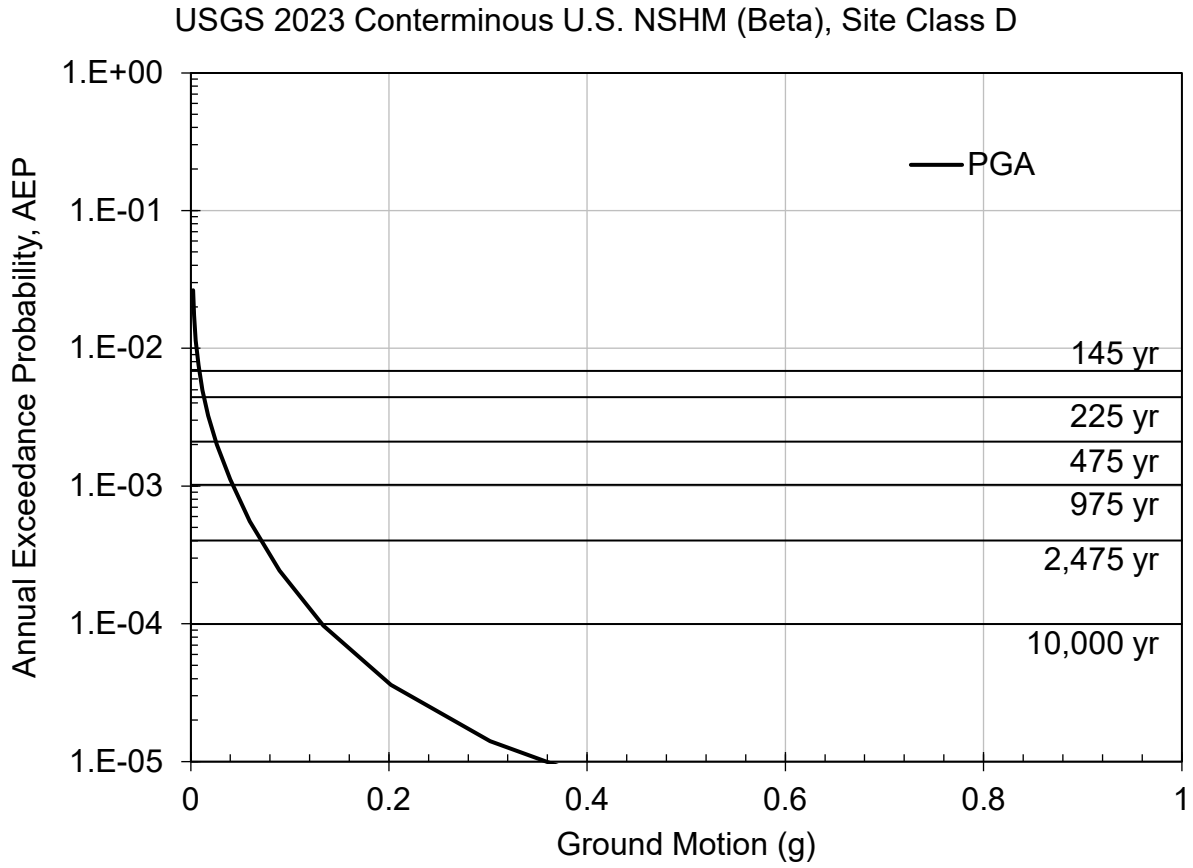


Figure 59. Seismic hazard curve for PGA and Site Class D (USGS 2023)

Table 30.
Peak horizontal ground acceleration summary for Site Class D (USGS 2023).

Return Period (years)	PGA (g)
145	0.00864
225	0.0132
475	0.0254
975	0.0419
2,475	0.0701
10,000	0.132

5.6. Disaggregation

The disaggregated seismic hazard for the PGA is shown in Figure 60 and Figure 61 using the USGS Earthquake Hazard Toolbox (https://earthquake.usgs.gov/nshmp/). Since shear wave velocity data was not available for the site, Site Class D corresponding to an average v_{s30} of 260 m/s (853 ft/s) was used to perform the disaggregation and generate these figures.

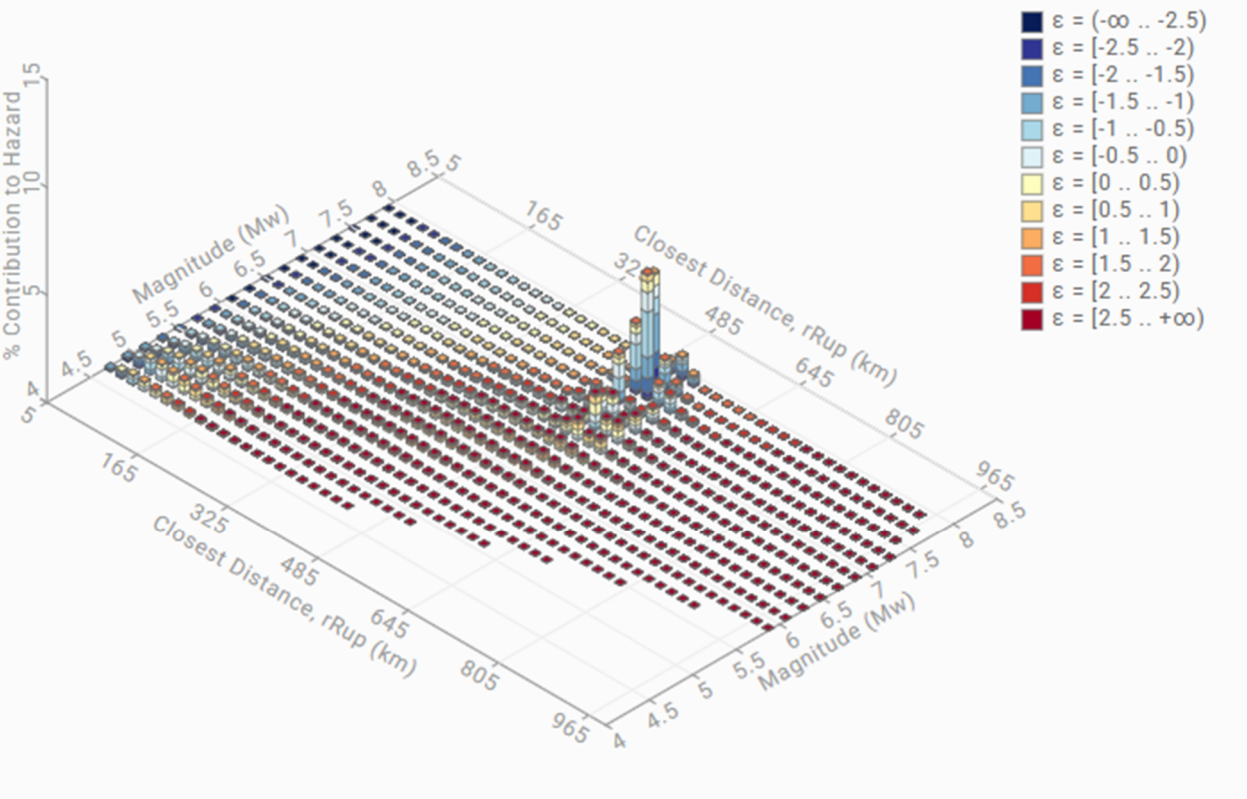


Figure 60. Probabilistic seismic hazard disaggregation for PGA and Site Class D with 1/ 145 AEP (USGS 2023)

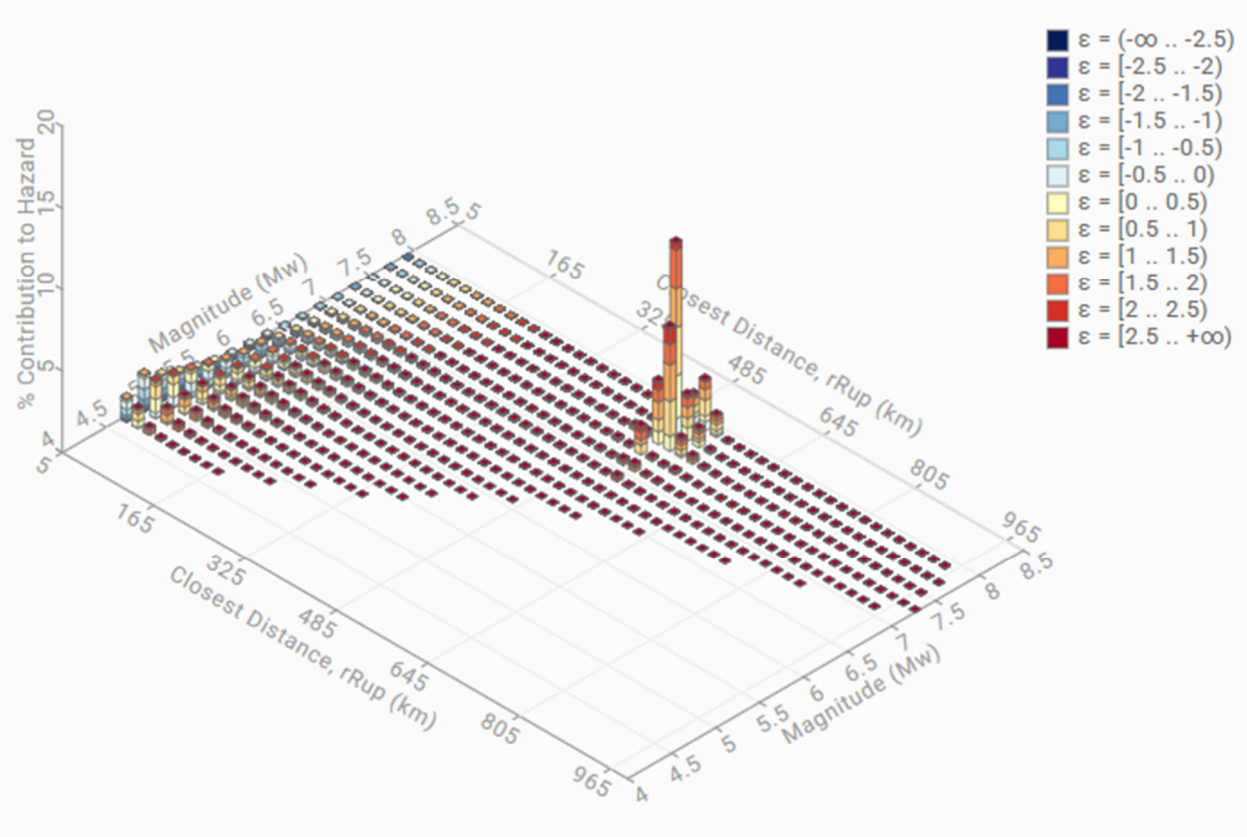


Figure 61. Probabilistic seismic hazard disaggregation for PGA and Site Class D with 1/ 2475 AEP (USGS 2023)

The disaggregation suggests that the primary contributors to the seismic hazard at the site include a M_w 7.5 earthquake on the New Madrid Seismic Zone at a distance of 511 kilometers, as well as local background seismicity ($M_w \leq 6$) at distances of less than 80 kilometers.

6. Consequences

6.1. Background

USACE has established a national standard of modeling procedures to support the estimation of consequences for breach and non-breach flood inundation scenarios for a range of loading conditions. The Modeling, Mapping and Consequence (MMC) Production Center is charged with producing scalable hydraulic levee breach models, flood inundation maps, and consequence estimations to support risk assessments. The MMC's SOP was followed by the district in the development of the hydraulic and consequence modeling for this risk assessment.

The MMC Production Center developed baseline consequence estimates for breach and non-breach inundation scenarios, uncertainty statistics for life loss estimates, and inundation mapping products. The difference between breach and non-breach consequences for a particular loading condition is the incremental consequences (i.e., those directly attributable to the levee breach for that loading condition). This information was used by the risk assessment team to assign an appropriate order-of-magnitude estimate for the risk-driving potential failure modes. Specific details and assumptions associated with the modeling, mapping, and consequences are provided with the various MMC Production Center products.

6.2. Inundation Scenarios

In order to evaluate a range of possible flood scenarios, five breach locations were included in the district's levee models. The locations and rationale for selection are summarized in Table 31, and the breach locations are shown in Figure 62.

Table 31.
Breach locations.

Breach Location	Station	Rationale for Selection
Levee Control Location (LCL)	RB 56+63	Location where significant overtopping first begins to occur
Breach Location 2 (BL2)	LB 77+43	Evaluating breach location of left bank downstream section
Breach Location 3 (BL3)	RB 47+08	Levee breach location to evaluate breach and spill into detention pond
Breach Location 4 (BL4)	RB 25+32	Evaluating right bank breach in middle of project reach in higher consequence area
Breach Location 5 (BL5)	LB 25+20	Evaluating left bank breach near upper portion of project reach



Figure 62. Breach locations.

Flood loading scenarios were developed using scaled frequency inflow flood hydrographs to produce the desired maximum water surface elevations at the Levee Control Location (LCL). The flood loading scenarios are schematically illustrated in Figure 63. Flood loading scenarios.

. The LCL maximum water surface elevation and ratio of the inflow hydrograph for each loading scenario are summarized in Table 32.

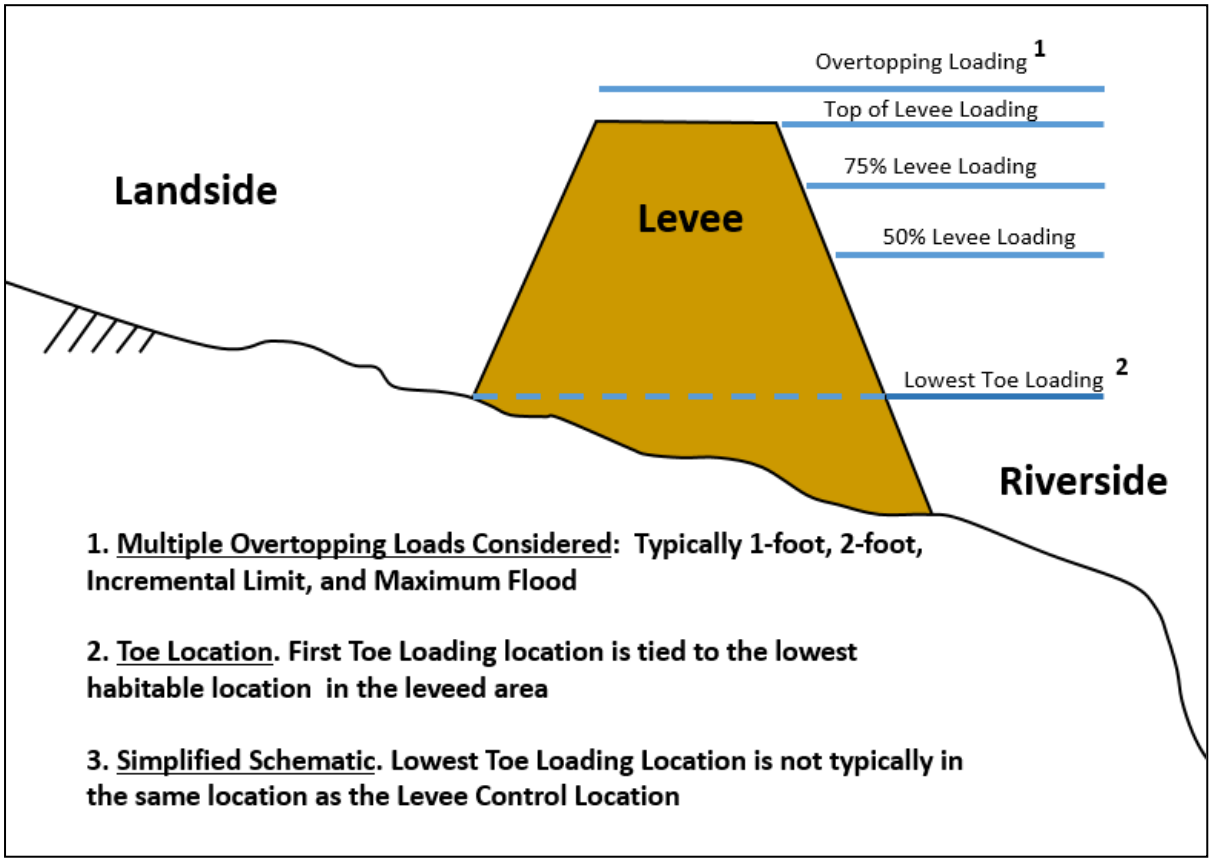


Figure 63. Flood loading scenarios.

Table 32.
Hydrologic loading event parameters.

Scenario	Water Surface Elevation at LCL (ft-NAVD88)	Ratio of Inflow Hydrograph
2-ft Overtopping	912.1	1.115
1-ft Overtopping	911.1	1.000
0.5-ft Overtopping	910.6	0.958
Top of Levee	910.1	0.908
75% Levee Height	909.6	0.956
50% Levee Height	909.3	0.904
Levee Toe	WAITING on INFO from H&H	

6.3. Breach Assumptions

The simplified physical breaching method in HEC-RAS was used to estimate breach parameters based on the physical conditions at the breach location. The method requires user-selected relationships for breach widening rate and breach downcutting rate as functions of velocity through the breach, which are used dynamically to estimate breach progression versus the actual velocity being computed through breach on a time-step by time-step basis. For this SQRA, the downcutting and widening rates were assumed the same. Erosion rates in HEC-RAS were based on a generalized descriptor of erodibility, and its associated

erodibility coefficient (k_d), which can be correlated with embankment material type (USACE 2022). It is assumed the embankment will consist of clayey sand, clayey sand with silty sand and well graded sand which corresponds to erodible materials. The assumed breach parameters are summarized in Table 33, and the estimated final breach dimensions are summarized in Table 34.

Table 33.
Simplified physical breach parameters for Sta. LB 77+43 BL2.

Velocity through Breach (ft/s)	Breach Widening Erosion Rate (ft/hr)
0	0
1	0.1607662
1.5	0.361724
2	0.6430649
3	1.446896
4	2.572259
6	5.787583
8	10.28904
10	16.07662
15	36.1724
20	64.30648
25	100.4789
30	144.6896

NOTE: Breach widening erosion rate is based on 4-ft tall erodible (k_d = 3.93 (ft/hr)/psf) embankment.

Table 34.
Simplified physical breach parameters for Sta. RB 47+08 BL3.

Velocity through Breach (ft/s)	Breach Widening Erosion Rate (ft/hr)
0	0
1	0.1492421
1.5	0.3357948
2	0.5969685
3	1.343179
4	2.387874
6	5.372716
8	9.551497
10	14.92421
15	33.57948
20	59.69685
25	93.27633
30	134.3179

NOTE: Breach widening erosion rate is based on 5-ft tall erodible (k_d = 3.93 (ft/hr)/psf) embankment.

Table 35.
Final levee breach dimensions.

Location (Station)	Loading Scenario	Final Breach Bottom Elevation (ft-NAVD88)	Final Breach Bottom Width (ft)
LB 77+43	TOL	909.57	200
RB 47+08	TOL	910.64	300

Breach dimensions and formation time for breach of a floodwall are drastically different than that of a breach of an embankment. Breach dimensions are often limited to the unsupported length of the critical floodwall section along with various other factors. In addition, floodwall failures are often considered to be brittle failures with very short breach formation times. For these reasons, a separate set of breach parameters was developed for floodwall breach at what was determined to be the critical section for floodwall instability. Due to the likely brittle nature of the breach, the breach was considered to happen instantaneously. A summary of the breach parameters for a floodwall breach are provided in Table 36.

Table 36.
Floodwall breach parameters.

Location (Station)	Loading Scenario	Final Breach Bottom Elevation (ft-NAVD88)	Final Breach Bottom Width (ft)
RB 56+63	LCL 2-ft OT	907.9	40
RB 25+32	BL4 OT	913.09	40
LB 25+20	BL5 OT	929.39	40

NOTE: Instantaneous breach formation time was assumed.

6.4. Description of Impacted Area

The project site of Merriam, Kansas sits in Johnson County and straddles Interstate 35 and is part of the Kanas City Metropolitan area. Upper Turkey Creek runs through the original downtown area of Merriam. At the lower end of the emergency planning zone, Upper Turkey Creek crosses under Shawnee Mission Parkway which is a major east-west thoroughfare and provides entrances and exits to Interstate 35. Upper Turkey Creek meanders in a north-northeast direction through Merriam before crossing under Interstate 35 near Antioch Road. The project is 1.2 river miles in length. There is a BNSF railroad track that lies between Merriam and the interstate. Merriam Drive generally parallels the interstate and crosses over Upper Turkey Creek north of downtown Merriam and ends at Shawnee Mission Parkway. From the point where Merriam Drive crosses over Upper Turkey Creek southwards, all structures to the east of the creek to the railroad are all smaller commercial and light industrial and residential to the west of the creek except for just north of Shawnee Mission Parkway where there are a few commercial structures. North of the point where Merriam Drive crosses Upper Turkey Creek all the structures in the emergency planning zone are commercial or light industrial except along the inflow to the northwest where there are some residential structures around the 51st area. The north end has some larger commercial structures than the other areas. Downtown has a large public park that is almost an entire block in size and across the street is a large farmer's market that is near Upper Turkey Creek.

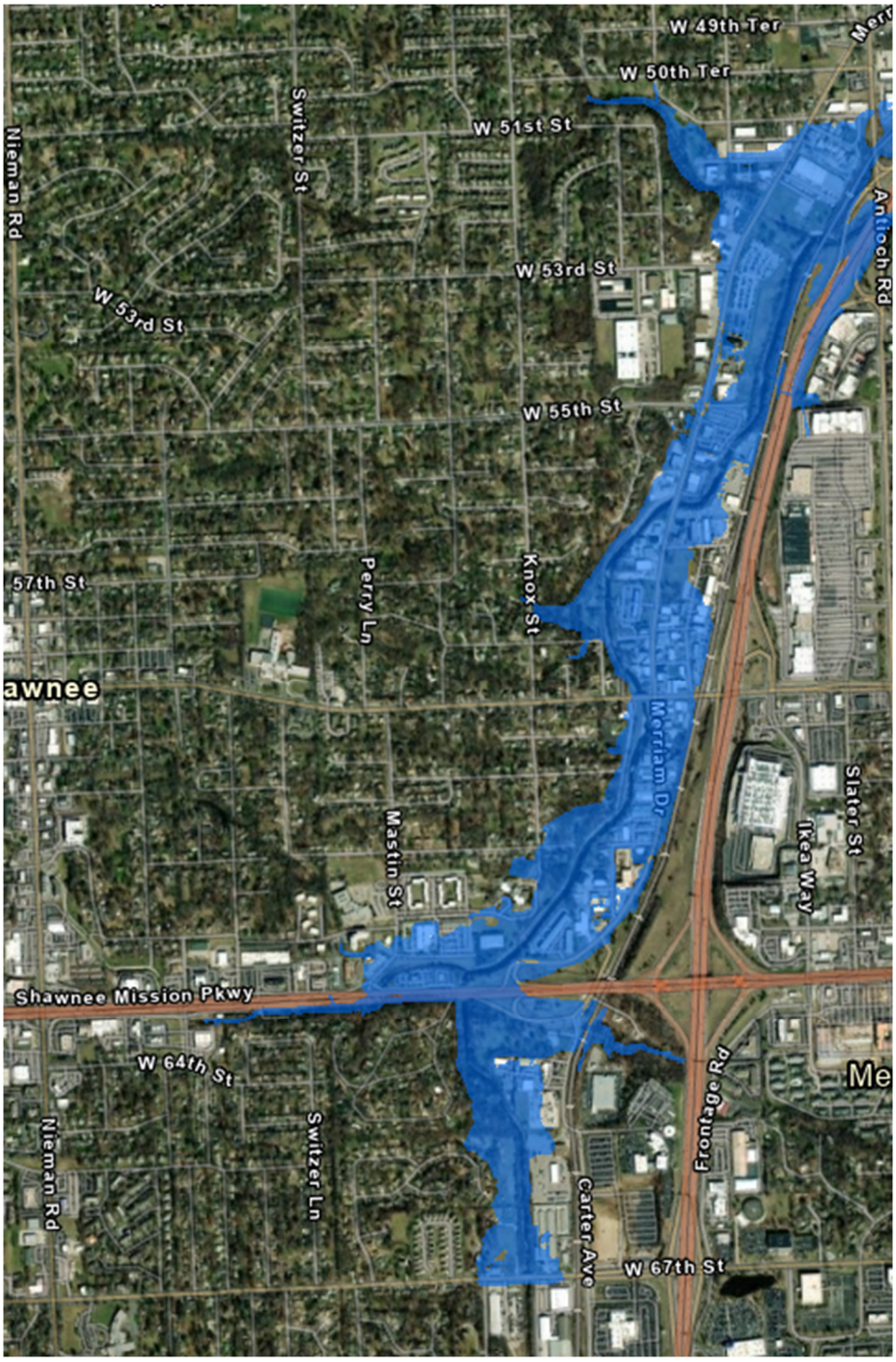


Figure 64. Sta. RB 56+63 LCL 2-ft OT flood inundation extent at Merriam, Kansas.

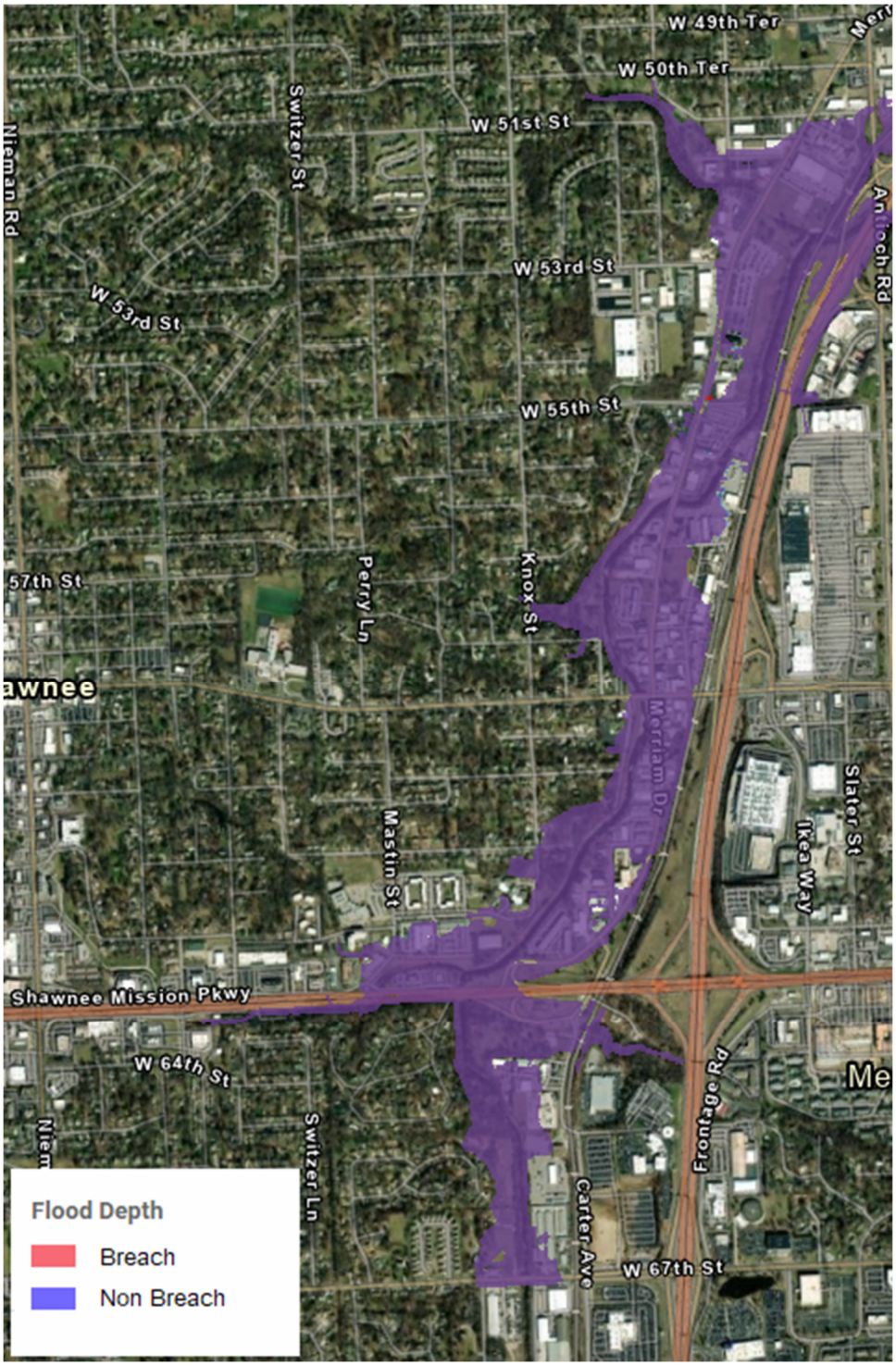


Figure 65. Sta. RB 56+63 LCL 2-ft OT breach and non-breach inundation at Merriam, Kansas.

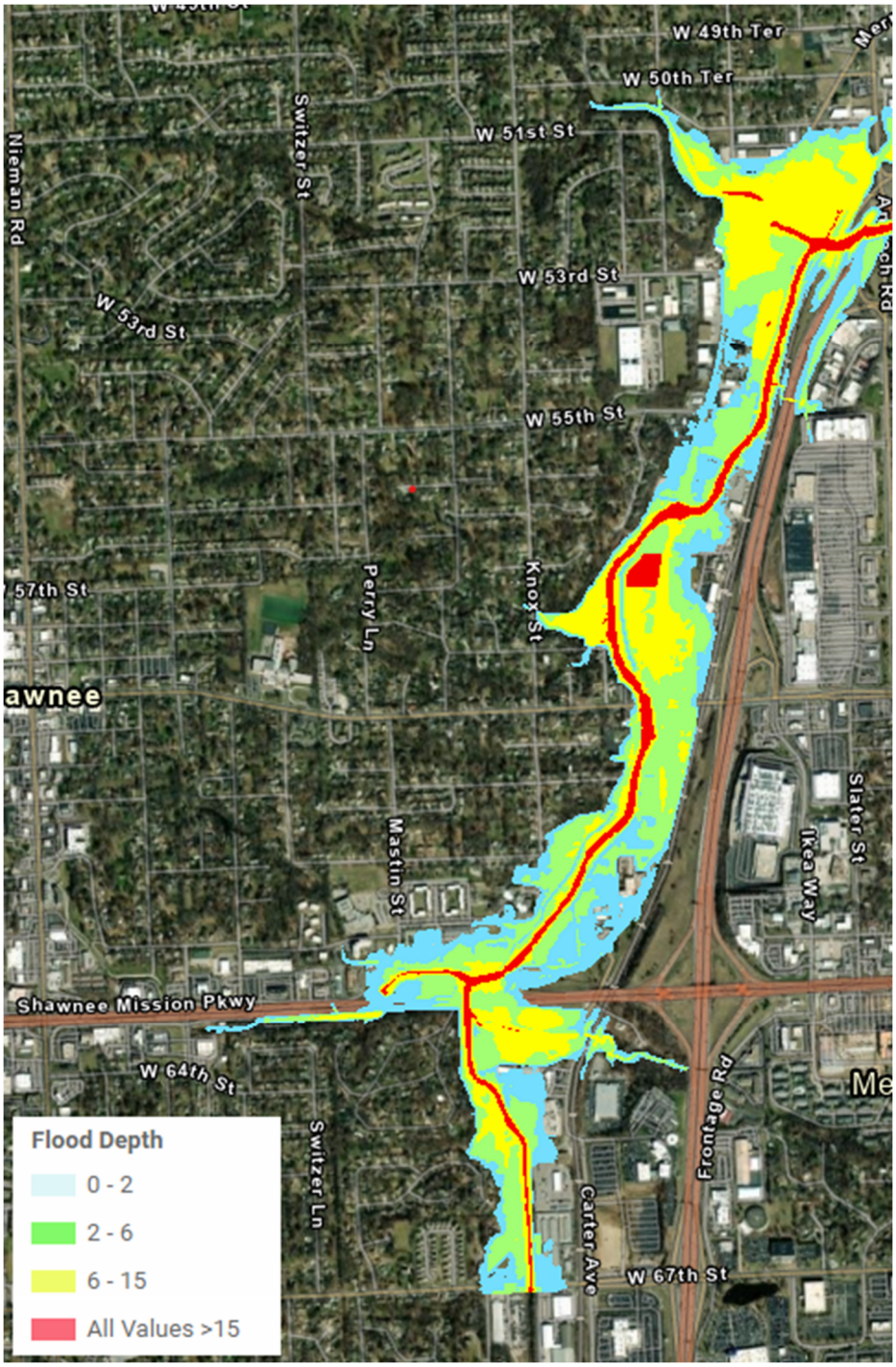


Figure 66. Sta. RB 56+63 LCL 2-ft OT breach flood depths at Merriam, Kansas.

6.5. Life Loss Methodology

This consequence modeling was performed using USACE's life loss and direct property damage estimation software LifeSim version 2.1.5. To determine the percentage of population at risk (PAR) within a structure that is warned and mobilized over time, several parameters are used within LifeSim to estimate the probable values of warning and mobilization percentages at each time step. These include

when hazards are identified and warnings will be issued (hazard identification and delays), how long they will take to become effective (warning diffusion), and the rate at which PAR will mobilize in response (mobilization or Protective Action Inhiation (PAI)). Figure 67 is an example breach warning and response timeline.

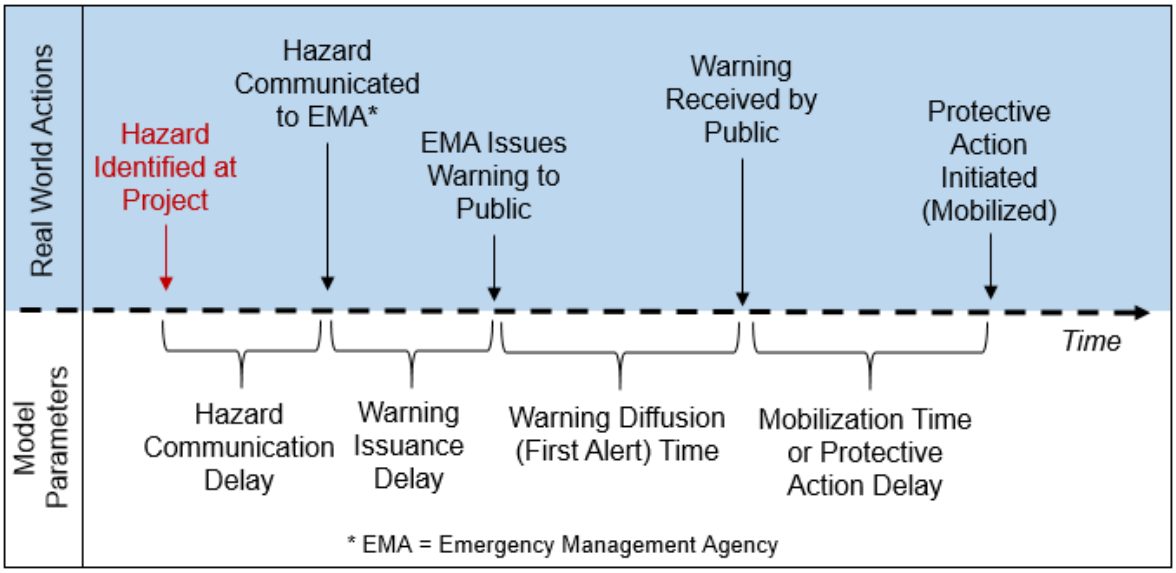


Figure 67. Warning and response timeline.

6.6. Population at Risk

Population at risk (PAR) is defined as the number of people within a leveed area that would be subject to inundation during a flood hazard event. Estimates of PAR were generated using LifeSim for breach and non-breach inundation scenarios. The estimated number of inundated structures and PAR are summarized in Table 37 and Table 38, respectively.

Table 37.
Estimated number of inundated structures.

Loading Scenario	Breach	Non-Breach	Incremental
LCL 2-ft OT	102	102	0
LCL 1-ft OT	75	75	0
LCL 0.5-ft OT	1	1	0
LCL TOL	1	0	1
LCL 75p	0	0	0
LCL 50p	0	0	0
BL2 TOL	0	0	0
BL2 75P	0	0	0
BL2 50P	0	0	0
BL3 TOL	0	0	0
BL3 75P	0	0	0
BL3 50P	0	0	0

Loading Scenario	Breach	Non-Breach	Incremental
BL4 TOL	43	0	43
BL4 75P	35	0	35
BL4 50P	24	0	24
BL5 TOL	3	0	3
BL5 75P	0	0	0
BL5 50P	0	0	0

Table 38.
Estimated mean population at risk.

Loading Scenario	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Sta RB 56+63 LCL 2-ft OT	510	105	510	105	0	0
Sta. RB 56+63 LCL 1.5-ft OT	446	102	446	102	0	0
Sta. RB 56+63 LCL 0.5-ft OT	4	0	4	0	0	0
Sta. RB 56+63 LCL TOL	4	0	0	0	4	0
Sta. RB 56+63 LCL 75p	0	0	0	0	0	0
Sta. RB 56+63 LCL 50p	0	0	0	0	0	0
Sta. LB 77+43 BL2 TOL	0	0	0	0	0	0
Sta. LB 77+43 BL2 75P	0	0	0	0	0	0
Sta. LB 77+43 BL2 50P	0	0	0	0	0	0
Sta. RB 47+08 BL3 TOL	0	0	0	0	0	0
Sta. RB 47+08 BL3 75P	0	0	0	0	0	0
Sta. RB 77+43 BL3 50P	0	0	0	0	0	0
Sta. RB 25+32 BL4 TOL	136	23	0	0	136	23
Sta. RB 25+32 BL4 75P	112	18	0	0	112	18
Sta. RB 25+32 BL4 50P	65	10	0	0	65	10
Sta. LB 25+20 BL5 TOL	4	7	0	0	4	7
Sta. LB 25+20 BL5 75P	510	105	510	105	0	0

Loading Scenario	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Sta. LB 25+20 BL5 50P	0	0	0	0	0	0

6.7. Life Loss Model Inputs and Parameters

6.7.1. Structure Inventory

The structure inventory was developed using the USACE National Structure Inventory (NSI) version 2022 as the base dataset. The NSI is developed from estimated information regarding the locations, building types, population, values, and other relevant information for all residential, commercial, industrial, agricultural, public, and private structures across the nation. Data sources for this information include national level parcel data, Census data, building footprints, and other sources for employment and school information. The value of structures, contents, and vehicles were indexed to 2025 price levels. Additional calibration on the structure point placement was accomplished using aerial imagery and inundation maps for both high and low flow scenarios.

6.7.2. Road Network and Destination Points

Evacuation simulation was not performed.

6.7.3. Emergency Preparedness

Johnson County has an “All-Hazard Mitigation Plan” that was developed by the county’s Office of Emergency Management in 2024. Included in the plan is a section dedicated to Flood Hazards. The section on Flood Hazards discusses historical flood events in Johnson County as well as dam failure inundation. It contains a detailed list of high hazard potential dams (earthen dams) and pertinent details. Maps and flood areas and areas of dam inundation zones are included in an appendix. There is not data on levees or levee risk. The Flood Hazards section provides information about flooding and potential sources of flood risk and may be useful for citizens of Johnson County to review to help them understand their risk of being exposed to flooding.

The “All-Hazard Mitigation Plan” does discuss emergency response or operations during a flood event. The plan does not include meaningful discussion about actions that would be taken by the county’s Office of Emergency Management or the county’s Department of Public Works during a flood hazard event. The expected response during a flood hazard event is not clear, and no actionable thresholds based on flood levels are identified in the plan. Additionally, there have been no recent flood events that necessitated PAR taking protective action. Emergency response during a flood event is unknown, and generally, the PAR appears to be unaware of their risk of being exposed to flooding.

The Emergency Preparedness parameters are described in the sections below.

6.7.4. Relative Hazard Identification

The Hazard Identification time is the time at which a hazard is identified (i.e., breach) relative to when it occurs (the actual breach initiation time). Two different warning scenarios were simulated with different ranges of hazard identification time: minimal warning and ample warning.

For overtopping scenarios, warning is relative to the time that overtopping begins, not breach time. The Relative Hazard Identification Times used for this study are summarized below.

Table 39.
Relative hazard identification times.

Warning Scenario	Distribution Type	Minimum (hr)	Maximum (hr)
Minimal	Uniform	-3	0.5
Ample	None	-24	-24

6.7.5. Hazard Communication Delay

The Hazard Communication Delay is the time that it would take from when the hazard is identified to when the emergency planning zone (EPZ) representatives would be notified. For example, if a breach occurs when no one is observing the project then the emergency managers could be notified after the hazard is identified. The hazard communication delay ranged between 0.01 and 0.5 hour and assumed a uniform distribution.

6.7.6. Warning Issuance Delay

The Warning Issuance Delay is the time it takes from when the emergency managers receive the notification of the imminent hazard to when they issue the first evacuation order to the public. Although the range of possible warning issuance delay is randomly sampled from 0 and 6 hours (300 minutes) after officials are notified of the flood hazard using a Lindell distribution, it is positively skewed so that results from 0 to 1.5 hours (90 minutes) are more likely.

6.7.7. Warning Diffusion

The preset “Preparedness Unknown” warning diffusion curves were used. The curves utilize a uniform distribution, and the warning diffusion curves are sampled during each Monte Carlo iteration in LifeSim. The upper bound of the curve reaches 100% diffusion just after 1.5 hours (100 minutes), and the lower bound reaches 100% diffusion after 6 hours (360 minutes).

6.7.8. Protective Action Initiation

Protective Action Initiation (PAI) is the rate at which PAR takes action after receiving an evacuation order (warning). Unlike the warning diffusion curves, the PAI “Preparedness Unknown” curve includes a perception element as well. The perception element describes a PAR as being aware of their flood risk (Perception = Likely to Impact) or generally unaware that they are at risk of being flooded (Perception = Unlikely to Impact). The “Preparedness Unknown, Perception Unknown” curve was used for all areas.

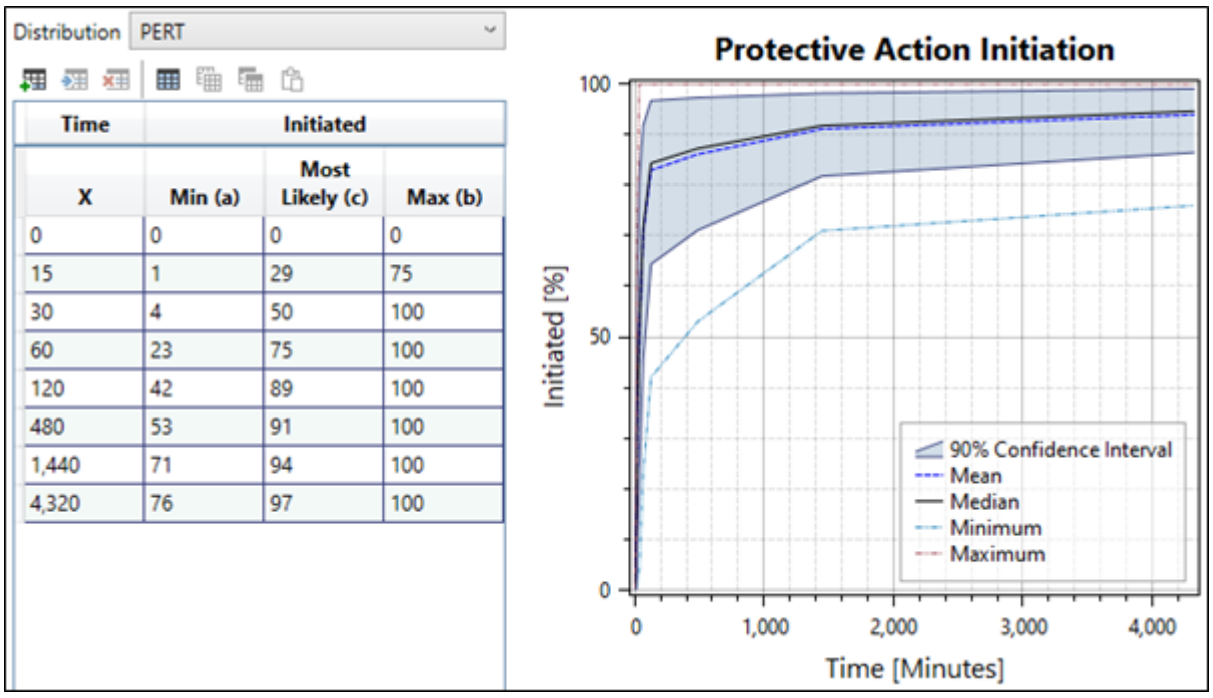


Figure 68. PAI curve for preparedness unknown, perception unknown.

6.8. Direct Life Loss

Estimates of direct life loss were generated using LifeSim for breach and non-breach inundation scenarios. LifeSim utilizes Monte Carlo uncertainty analysis, and 2,500 were performed for each scenario. The estimated direct life loss by breach location and loading scenario are summarized in the odd numbered tables Table 40 through Table 74 for minimal warning and in the even numbered tables Table 41 through Table 75 for ample warning. While the direct breach and non-breach results represent the outcome from model simulations, the direct incremental life loss results were obtained by subtracting the breach and non-breach results.

Table 40.
Estimated direct life loss for breach at Sta. RB 56+63 LCL 2-ft OT with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	9	4	8	3	1	1
95th Percentile	5	1	4	1	1	0
75th Percentile	3	0	2	0	1	0
Mean	2	0	1	0	1	0
Median	2	0	1	0	1	0
25th Percentile	1	0	0	0	1	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 41.
Estimated direct life loss for breach Sta. RB 56+63 LCL 2-ft OT with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	4	2	4	2	0	0
95th Percentile	1	0	1	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 42.
Estimated direct life loss for breach at Sta. RB 56+63 LCL 1-ft OT with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	1	1	1	1	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 43.
Estimated direct life loss for breach Sta. RB 56+63 LCL 1-ft OT with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	3	1	3	1	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 44.
Estimated direct life loss for breach at Sta. RB 56+63 LCL 0.5-ft OT with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 45.
Estimated direct life loss for breach Sta. RB 56+63 LCL 0.5-ft OT with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 46.
Estimated direct life loss for breach at Sta. RB 56+63 LCL TOL with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 47.
Estimated direct life loss for breach Sta. RB 56+63 LCL TOL with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 48.
Estimated direct life loss for breach at Sta. RB 56+63 LCL 75p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 49.
Estimated direct life loss for breach Sta. RB 56+63 LCL 75p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 50.
Estimated direct life loss for breach at Sta. RB 56+63 LCL 50p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 51.
Estimated direct life loss for breach Sta. RB 56+63 LCL 50p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 52.
Estimated direct life loss for breach at Sta. LB 77+43 BL2 TOL with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 53.
Estimated direct life loss for breach Sta. LB 77+43 BL2 TOL with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 54.
Estimated direct life loss for breach at Sta. LB 77+43 BL2 75p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 55.
Estimated direct life loss for breach Sta. LB 77+43 BL2 75p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 56.
Estimated direct life loss for breach at Sta. LB 77+43 BL2 50p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 57.
Estimated direct life loss for breach Sta. LB 77+43 BL2 50p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 58.
Estimated direct life loss for breach at Sta. RB 47+08 BL3 TOL with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 59.
Estimated direct life loss for breach Sta. RB 47+08 BL3 TOL with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 60.
Estimated direct life loss for breach at Sta. RB 47+08 BL3 75p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 61.
Estimated direct life loss for breach Sta. RB 47+08 BL3 75p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 62.
Estimated direct life loss for breach at Sta. RB 47+08 BL3 50p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 63.
Estimated direct life loss for breach Sta. RB 47+08 BL3 50p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 64.
Estimated direct life loss for breach at Sta. RB 25+32 BL4 TOL with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 65.
Estimated direct life loss for breach Sta. RB 25+32 BL4 TOL with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 66.
Estimated direct life loss for breach at Sta. RB 25+32 BL4 75p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 67.
Estimated direct life loss for breach Sta. RB 25+32 BL4 75p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 68.
Estimated direct life loss for breach at Sta. RB 25+32 BL4 50p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 69.
Estimated direct life loss for breach Sta. RB 25+32 BL4 50p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 70.
Estimated direct life loss for breach at Sta. LB 25+20 BL5 TOL with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 71.
Estimated direct life loss for breach Sta. LB 25+20 BL5 TOL with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 72.
Estimated direct life loss for breach at Sta. LB 25+20 BL5 75p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 73.
Estimated direct life loss for breach Sta. LB 25+20 BL5 75p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 74.
Estimated direct life loss for breach at Sta. LB 25+20 BL5 50p with minimal warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

Table 75.
Estimated direct life loss for breach Sta. LB 25+20 BL5 50p with ample warning.

Statistic	Daytime Breach	Nighttime Breach	Daytime Non-Breach	Nighttime Non-Breach	Daytime Incremental	Nighttime Incremental
Maximum	0	0	0	0	0	0
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Mean	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0
Minimum	0	0	0	0	0	0

These above statistical summaries only reflect the uncertainty in the LifeSim input parameters and do not include uncertainties for breach parameters or other hydrologic and hydraulic factors. Normally, direct life loss statistics for each run are shown using box-and-whisker plots but as most of the incremental life loss numbers are zero which cannot be plotted on a log scale, the box-and-whisker plots have been omitted.

6.8.1. Life Loss on Roads

With the very low life loss in structures, the small size of the project area and the fact that the majority of the structures are commercial and/or industrial evacuation simulation was not performed.

6.9. Indirect Life Loss

Indirect life loss is based on the Impacted Population at Risk (iPAR), which is the group of people impacted by the sub-optimal conditions created by a disaster. To determine iPAR values, direct life loss numbers are removed from the initial PAR estimates. This includes the PAR comprising the population within buildings that will be flooded and those outside of the inundation zone who might evacuate due to uncertainty or other conditions as well as those who might lose power for an extended period of time. The RMC has developed guidance for estimation of indirect life loss based on research of disaster case histories. These case histories show that a reasonable estimate of indirect life loss can be based on a mean

indirect fatality rate of 0.1% of the iPAR that evacuates and 0.5% of the iPAR that does not evacuate and is exposed to flooding. This can be used to develop a screening-level estimate of indirect life loss. The table below shows the potential indirect life loss based on the iPAR for in comparison with the direct life loss estimated from LifeSim

Table 76.
Comparison of potential daytime indirect and direct incremental life loss.

Loading Scenario	Mean Evacuee Incremental Population at Risk	Mean Non-Evacuee Incremental Population at Risk	Mean Evacuee Incremental Indirect Life Loss (0.1% of Evacuee iPAR)	Mean Non-Evacuee Incremental Indirect Life Loss (0.5% of Non-Evacuee iPAR)	Mean Direct Incremental Life Loss[1]
Sta. RB 56+63 LCL 2-ft OT	343	166	0.3	0.8	1
Sta. RB 56+63 LCL 1-ft OT	339	108	0.3	0.5	1
Sta. RB 56+63 LCL 0.5-ft OT	4	0	0.0	0	0
Sta. RB 56+63 LCL TOL	4	0	0.0	0	0

NOTE: [1]Mean direct incremental life loss is for minimal warning.

Based on the indirect incremental life loss screening, it is unlikely that a detailed estimate and inclusion of indirect incremental life loss would change the estimated average incremental life loss by a half order of magnitude or more. Therefore, a detailed evaluation of indirect life loss was not performed for this SQRA.

Indirect life loss is typically caused by cardiac events or cardiovascular failure among the elderly and people with pre-existing health conditions. Other causes of indirect life loss would be power related issues that lead to carbon monoxide poisoning or failure of medical devices and life loss during evacuation not caused by drowning.

6.10. Economic Losses

Economic considerations help inform risk management decisions. Direct economic losses considered in this SQRA included include direct property damages, and levee repair or replacement costs.

6.10.1. Direct Property Damage

An inventory of downstream property directly damaged by the inundation was developed using the structure inventory described in Section 6.7.1 which contains information for residential, commercial, industrial, agricultural, public and private structures as well as data for essential facilities such as hospitals, clinics, fire stations, police stations, emergency operation centers, schools and universities.

Direct property damage including structure, content, and vehicle damage was estimated in LifeSim using standard depth-percent damage curves specific to each structure occupancy type and structure stability curves which evaluate damage caused by combined depth and velocity.

The area near the project is predominantly commercial and light industrial businesses and these are the structures are getting damaged. The estimated mean direct property damage is summarized in Table 77.

Table 77.
Estimated mean direct property damage.

Loading Scenario	Breach	Non-Breach	Incremental
Sta. RB 56+63 LCL 2-ft OT	$49,260,000	$49,260,000	$0
Sta. RB 56+63 LCL 1-ft OT	$25,820,000	$25,820,000	$0
Sta. RB 56+63 LCL 0.5-ft OT	$45,800	$3,500	$42,300
Sta. RB 56+63 LCL TOL	$4,300	$0	$4,300
Sta. RB 56+63 LCL 75P	$0	$0	$0
Sta. RB 56+63 LCL 50P	$0	$0	$0
Sta. LB 77+43 BL2 TOL	$0	$0	$0
Sta. LB 77+43 BL2 75P	$0	$0	$0
Sta. LB 77+43 BL2 50P	$0	$0	$0
Sta. RB 47+08 BL3 TOL	$0	$0	$0
Sta. RB 47+08 BL3 75P	$0	$0	$0
Sta. RB 47+08 BL3 50P	$0	$0	$0
Sta. RB 25+32 BL4 TOL	$7,175,000	$0	$7,175,000
Sta. RB 25+32 BL4 75P	$2,761,000	$0	$2,761,000
Sta. RB 25+32 BL4 50P	$1,714,000	$0	$1,714,000
Sta. LB 25+20 BL5 TOL	$0	$0	$0
Sta. LB 25+20 BL5 75P	$0	$0	$0
Sta. LB 25+20 BL5 50P	$0	$0	$0

NOTE: FY 2025 price level

6.10.2. Levee Embankment and Floodwall Repair or Replacement Costs

As this is a design SQRA and the levee has not been through 65% design yet, the costs are not firm enough that it would make sense to estimate replacement costs at this point.

7. Risk Assessment

7.1. Potential Failure Mode Analysis

A potential failure mode (PFM) is a unique set of conditions and/or sequence of events that could result in failure. For dam and levee safety risk assessments, failure generally results in breach or the formation of a gap in the dam or levee system through which water may flow uncontrolled onto the downstream flood plain or adjacent leveed area. It is characterized by the "sudden, rapid, and uncontrolled release of impounded water" as defined in FEMA P-93 (FEMA 2004), and FEMA P-1025 (FEMA 2015) expanded the definition of failure to include "liquid-borne solids" like mine tailings and dredge spoils.

A potential failure mode analysis (PFMA) is a facilitated process of identifying and fully describing PFMs. It is based on a team's understanding of the project's vulnerabilities from a review of existing data and conditions. The facilitated PFMA and SQRA were performed on 08-11 Month 2025. Additional meetings were held virtually after the in-person meeting to discuss the risk drivers and elicit. The SQRA team and their roles are provided in Table 78.

Table 78.
PFMA team.

Name	Organization/Office	Discipline/Role
Alex Nelson, P.E.	Levee Safety Center	Hydraulic Engineer (Facilitator)
Axel Montalvo-Bartolomei, P.E.	Levee Safety Center	Geotechnical Engineer (Co-facilitator)
Courtney Forsythe	Kansas City District	Tech Lead / Civil Engineer
Conner Szarwinski	Kansas City District	H&H Engineer
Luke Augustine	Kansas City District	Geotechnical Engineer
Brandon Harmon	Kansas City District	Geologist
Paul Muller	Kansas City District	Structural Engineer
Devin Smith	Kansas City District	Economist
Kailey Jones	Kansas City District	Project Manager
Derek Jenson	Kansas City District	Geotechnical Engineer
Forrest Pritt	Kansas City District	Civil Engineer
Gregory Wachman	St. Paul District	Geotechnical Engineer

Advance preparation includes collecting and reviewing the background and performance data, site characterization, hydrologic and seismic hazards, hydraulic and consequence modeling, and performing any screening or limited supporting analysis. Prior to the PFMA, a site visit was performed to help the team look for clues as to how the dam or levee system and appurtenant structures may be vulnerable to failure.

A site visit and review of background data was initially conducted followed by brainstorming of PFMs by the SQRA team. Table 79 provides the list of PFMs brainstormed by the SQRA team. PFMs related to sabotage or terrorism were beyond the scope of this risk assessment. From the list of brainstormed PFMs resulting in breach and uncontrolled release, the SQRA team identified the PFMs judged to be risk drivers. PFMs that were judged to be non-risk drivers were excluded from further consideration, and the rationale is provided in Appendix A.

Table 79.
Potential failure modes.

Potential Failure Mode	Risk Driver?	Description
PFM 01	Yes	Overtopping breach of the levee
PFM 02	No	Overtopping breach of the floodwall
PFM 03	No	Concentrated leak erosion through the embankment between construction lifts
PFM 04	No	CLE along differential settlement
PFM 05	No	CLE along levee/wall/bridge interfaces
PFM 06	No	Flap gates stuck open resulting in backflow (for pipe >36", assuming one flap gate per pipe)
PFM 07	Yes	Global instability of the levee
PFM 08	No	Global instability of the I-wall
PFM 09	Yes	Global instability of the T-wall
PFM 10	No	Local instability of the I-wall
PFM 11	No	Local instability of the T-wall
PFM 12	No	Internal migration into the storm sewer
PFM 13	Yes	Internal migration into sanitary sewer
PFM 14	No	Internal erosion out of sanitary sewer
PFM 15	No	CLE along a pipe
PFM 16	No	CLE or BEP through the levee foundation
PFM 17	No	CLE or BEP through I-wall foundation
PFM 18	No	CLE or BEP through T-wall foundation
PFM 19	No	Instability of limestone wall leads to levee instability
PFM 20	No	Instability of limestone wall leads to I-wall instability
PFM 21	No	Instability of limestone wall leads to T-wall instability
PFM 22	No	Seismic instability of levee or wall
PFM 23	No	Riverine erosion of levee toe or levee face
PFM 24	No	Riverine erosion of T-wall foundation
PFM 25	No	Riverine erosion of I-wall foundation
PFM 26	No	Failure of Johnson Bridge due to uplift
PFM 27	No	Debris impact failure of wall/bridge
PFM 28	No	Pedestrian bridge fails and impacts wall/bridge/levee
PFM 29	No	Channel erosion undermines limestone wall and fails levee/wall
PFM 30	No	Structural failure of I-wall
PFM 31	No	Structural failure of T-wall
PMF 32	No	Vehicle impact failure of I-wall
PMF 33	No	Tree next to the floodwalls/levees topples and leads to instability
PMF 34	No	Wind-wave levee erosion
PMF 35	No	Internal erosion out of pressurized pipe
PMF 36	No	Bridge construction leads to increased water levels for existing conditions
PMF 37	No	Flap gate stuck closed leads to excessive ponding

7.2. Risk Assessment

A semi-quantitative risk assessment (SQRA) is a facilitated process of making order-of-magnitude estimates of likelihood of failure and average incremental consequences. It is based on limited numerical data and judgment. The facilitated SQRA was performed using the team listed in Table 78.

Risk drivers from the PFMA were fully developed and semi-quantitatively evaluated. This includes a complete description of the failure mechanism from initiation to failure, nature of the failure (e.g., breach characteristics) and consequences, event tree, pertinent background and performance data, likelihood factors, estimated likelihood of failure and associated consequences, intervention, plotting on a risk matrix, and any sources of significant uncertainty in the estimates. Lastly, any recommendations for risk-reduction actions, instrumentation and monitoring, additional data, or analysis were discussed. Table 80 summarizes the PFMs judged to be risk drivers.

Table 80.
Risk-driving potential failure modes.

Potential Failure Mode	Description
PFM 01	Overtopping breach of the levee
PFM 07	Global instability of the levee
PFM 13	Internal migration into sanitary sewer

The incremental risk (due to failure) considers likelihood of failure and incremental consequences. The likelihood of failure is a function of both the likelihood of the loading condition that could lead to failure (i.e., critical loading) and the likelihood of failure given the loading condition (i.e., conditional probability of failure or system response probability). During the SQRA, order-of-magnitude estimates were made for both likelihood of failure and incremental consequences for each risk-driving potential failures mode. The evaluation of each risk-driving PFM was documented as well as any sources of significant uncertainty.

7.3. PFM 01: Overtopping breach of the levee

7.3.1. Description

A flood occurs on Turkey Creek with a flow greater than 19,000 cfs, AEP of 7.65E-03 (1/130-year) event. Efforts to raise the levee are unsuccessful and the levee system begins to experience balanced overtopping. Overtopping of a large portion of both floodwalls and levee sections occurs almost immediately after overtopping starts at the levee control location (LCL). A 10-feet tall levee was evaluated as a representative section. As flows increase on Turkey Creek to 0.5-feet of overtopping AEP of 5.62E-03 (19,800 cfs, ~1/180 AEP), 1-foot of overtopping AEP of 4.02E-03 (21,300 cfs, ~1/250 AEP), and the 2-foot overtopping AEP of 2.11E-03 (25,800 cfs, ~1/470 AEP) the overtopping flow velocity is sufficient to erode and remove the landside grass cover. Knick points are created at the levee toe slope change, and erosion begins on the landside levee slope. The erosion channel deepens and widens, resulting in the formation of head-cuts on the landside slope, and rill development occurs. Overtopping is of sufficient duration such that progressive headcutting advances through the crest of the levee. This results in a rapid uncontrolled release of water into the leveed area with associated loss of life and economic consequences. An event tree of this failure description is shown in Figure 75 along with a location of a potential 10-foot tall levee overtopping in Figure 69, and details of the embankment section in Figure 70.

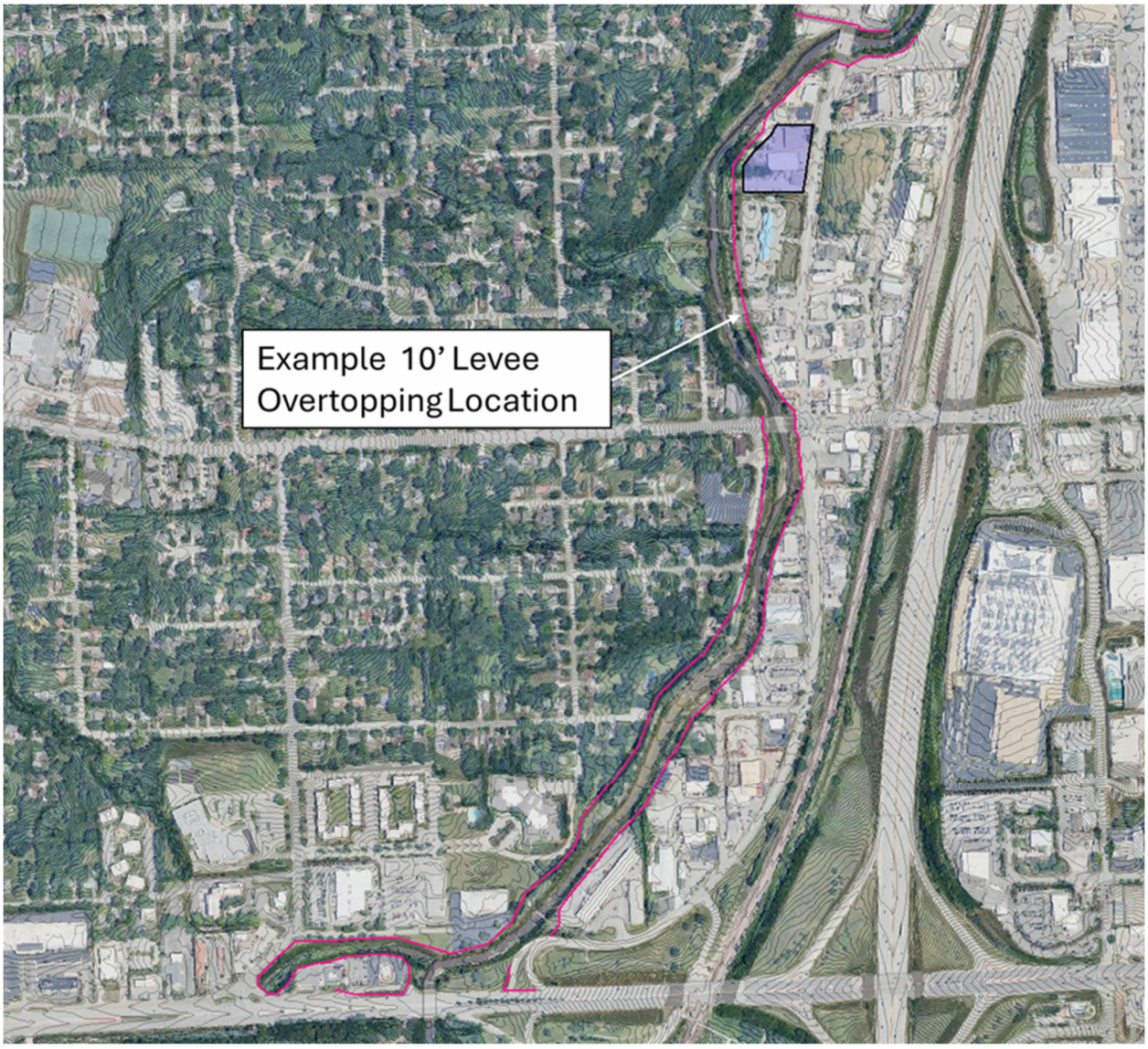


Figure 69. Example overtopping location of a 10-foot levee section.

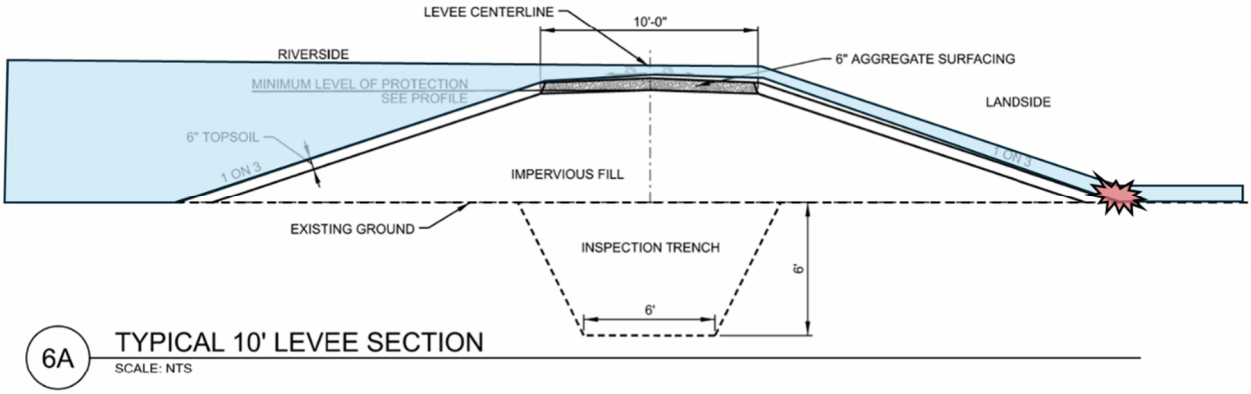


Figure 70. Overtopping at a 10-foot-tall levee section.

7.3.2. Background

The typical levee height is around 10 feet tall but can be as low as 4 feet or as tall as 14 feet in some sections. The levees are designed with a 10 to 14-foot top width. Some of the levee crowns will be paved with asphalt to replace existing paved trails. Riverside and landside slopes are to be 1V:3H and will have planted and established grass cover. Stability berms and sand drains are tentatively included in the design in some areas but are expected to be removed as part of the risk informed design changes, so they were not factored in to the elicitation of this PFM. The impervious fill is to consist of lean clays and/or low plasticity silt, which could be considered moderately resistant to erosion. A borrow area has not yet been identified, though the detention pond is a potential source pending HTRW.

Overtopping durations and velocities were analyzed for this failure mode for the three overtopping depths outlined below. Table 81 shows overtopping velocities at the levee section along with overtopping durations previously estimated in Chapter 4 for each loading scenario along with the peak elevations at the LCL. These durations were used for the evaluation of PFM 01 at an example 10-foot-tall levee section. The overtopping velocities were determined in a spreadsheet using weir equations, normal depth, and critical depth equations. The normal velocity is computed using the Manning's equation, after computing the normal depth by an iterative process. The velocities considered from the spreadsheet are the normal depth velocities. Overbank velocities in the channel can influence overtopping velocities and are fairly high in the channel, but are lower along the landside embankment, around 5-6 ft/sec.

Table 81. Overtopping velocities and durations at the 10-foot levee section.

MMC Loading Scenario	Overtopping Velocity (ft/s)*	Total Overtopping Duration	Duration Above Toe
0.5 ft OT	8.3	0.5 hrs	1 hr
1 ft OT	12.6	1 hr	1.5 hrs
2 ft OT	18.7	1.5 hrs	1.67 hrs

*Velocities are estimated with a typical 10-foot levee section since the LCL is located at a short floodwall section

Velocities on the 1V:3H levee slope likely exceed 8 ft/sec for overtopping depths greater than or equal to 0.5 feet. Though this can depend on the height of the levee section. Levee heights vary from 4- to 10-feet with the current design iteration. Figure 71 shows the estimated distance from the riverside crest to develop peak velocities. It should be noted that overtopping depths of 2-feet lead to widespread

overtopping of almost the entirety of the flood protection system. Interior depths would be sufficient that flow would enter back into the channel on the downstream end of the project area by way of landside overtopping. Stage-frequencies were estimated at the LCL and used for PFM 01 given that the levee experiences balanced overtopping throughout the levee system immediately after incipient overtopping. This is shown in Table 82.

Table 82. Stage-Frequency at LCL location (Channel STA 65+40)

Loading Scenario	AEP	Elevation (ft, NAVD88) at LCL (Channel STA 65+40)
1,000	1.00E-03	913.15
500	2.00E-03	912.19
470 (2ft OT)	2.11E-03	912.10
250 (1ft OT)	4.02E-03	911.10
200	5.00E-03	910.78
180 (0.5ft OT)	5.62E-03	910.60
130 (TOL)	7.65E-03	910.10
100	1.00E-02	909.55
50	2.00E-02	908.16
20	5.00E-02	905.99
10	1.00E-01	904.05
5	2.00E-01	902.32
2	5.00E-01	899.89

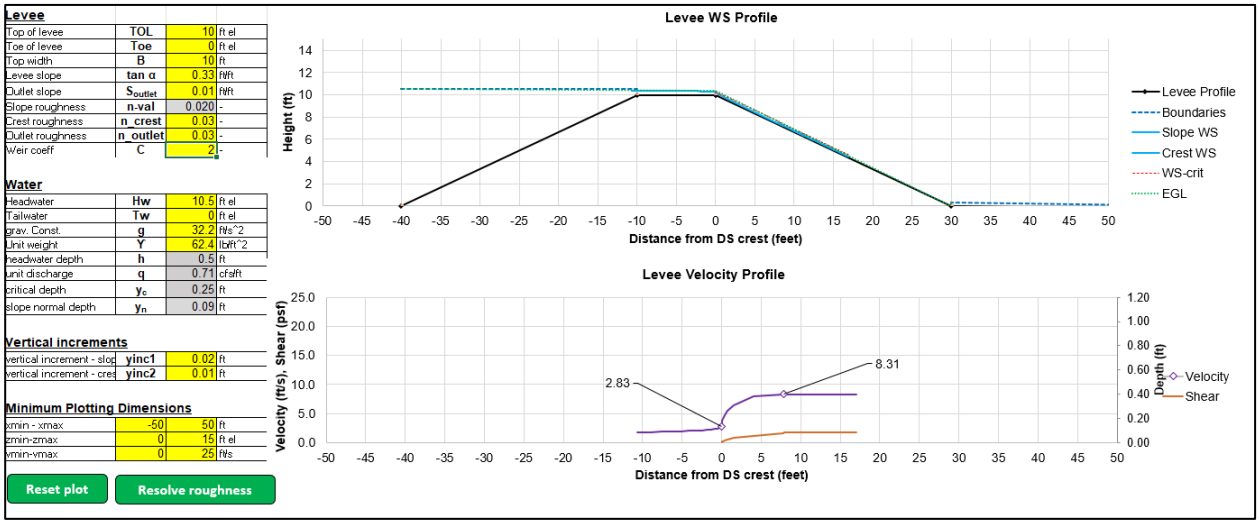


Figure 71. Estimated overtopping velocities for 0.5-ft OT depth at 10-ft levee section.

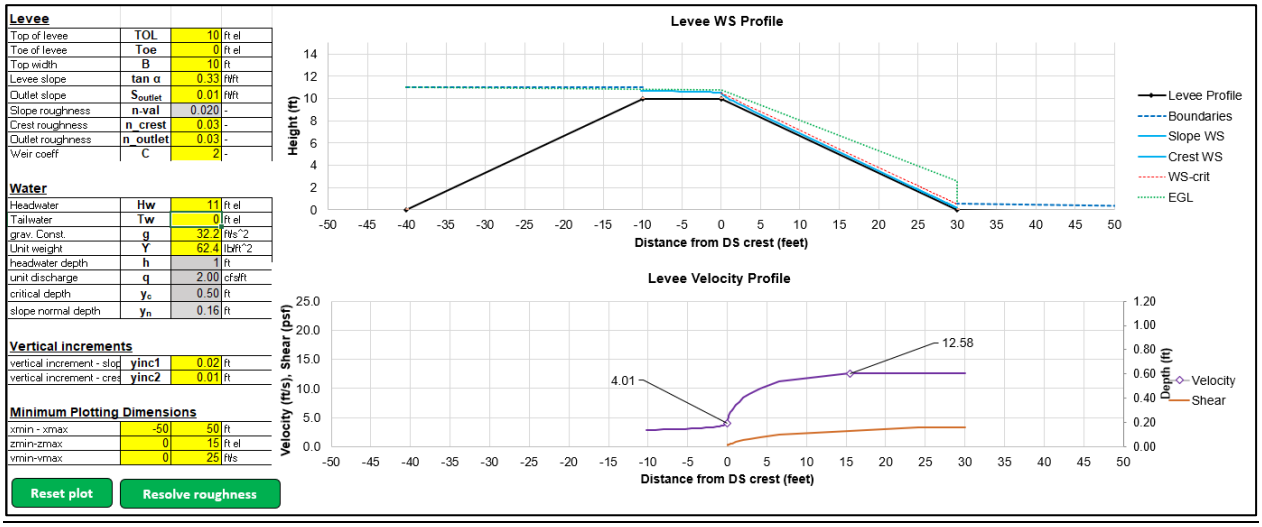


Figure 72. Estimated overtopping velocities for 1-ft OT depth at 10-ft levee section.

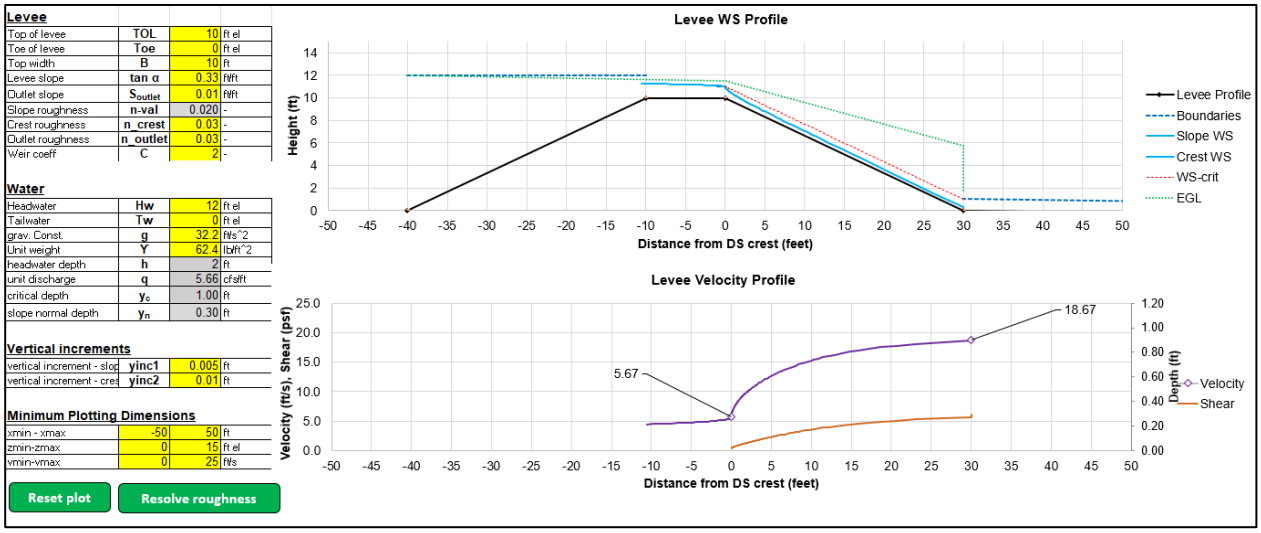


Figure 73. Estimated overtopping velocities for 2-ft OT depth at 10-ft levee section.

7.3.3. Performance

The levee system is in the design phase, and thusly, has no performance history.

7.3.4. Intervention

Intervention is unlikely to occur during an extreme flood event. The Turkey Creek watershed does not have flood forecasting capabilities such as those available through the National Weather Service. The upstream drainage area of Turkey Creek is approximately 12 square miles and heavily urbanized. Storm events of sufficient magnitude result in very flashy floods that leave little time to prepare or respond to. Rate of rise of the channel for overtopping events are likely to exceed 20 feet/hour With the total out of bank duration lasting only an estimated 3-4 hours. Drainage structures are proposed to be fully passive,

and no closure structures exist in the design because of the little warning time and flashiness of the system.

7.3.5. Likelihood of Failure

Table 83 summarizes the likelihood factors that make the PFM more likely or less likely to lead to failure.

Table 83.
Likelihood factors for PFM 1.

More Likely (Adverse) Factors	Less Likely (Favorable) Factors
• Sand drain exiting at the landside toe will be more vulnerable to erosion once the vegetation and top soil is removed. This is at the location of highest landside slope velocities and may lead to faster headcut progression.	• **Short duration of expected overtopping flows (less than 2 hours for up to 2 feet of peak overtopping depth) may be insufficient to initiate or progress erosion to failure.**
• Unsuccessful intervention is likely due to the flashy nature of flooding	• Lean clay and low plasticity silt may be unlikely to erode tens of feet to fully breach the levee in the short duration of overtopping.
• Consequence warning will be minimal as there are no forecasting capabilities for this creek.	• Levee is in a high visibility area so maintenance of vegetation or animal burrows should be dealt with regularly.
• Landside slope velocities of 8 or more ft/s are likely to erode clays and silts, if vegetation has been removed.	• Vegetation is likely in average or good condition and will likely withstand short durations of low overtopping depths without failure.
• Debris blockage at the bridges or debris/vegetation growth in the channel can result in increased overtopping depths further upstream on the project. If channel maintenance is deferred, further sedimentation and debris accumulation can continue to increase stages.	• For greater depths of overtopping (e.g., 2 feet), overtopping is occurring along most of each levee segment which would build tailwater making failure less likely and incremental consequences from a breach minimal.
• No features present to arrest the erosion in the levee.	•

NOTE: Key factors are shown in **bold**.

The annual probability of failure was estimated between 3E-04 and 3E-03 (with and without intervention)

7.3.5.1. Rationale

7.3.5.1.1. 0.5 ft of OT

Overtopping occurs in numerous locations at 0.5 feet of depth at the LCL, including levee and floodwall sections. The 10-foot levee section used for the risk assessment will experience an estimated overtopping velocity of 8.3 ft/s on the landside slope. Grass stripping is very unlikely to unlikely because of the short overtopping duration with the assumption of established and good vegetation cover. The total overtopping duration of 0.5 hours. Knickpoints are expected to be the slope change at the landside toe.. Lean clays and low plasticity silts along with potentially asphalt paved crests are expected to slow headcutting. Intervention is not expected to occur due to the flashiness of the system with no established flood forecasting for the watershed. Additionally, there are no known low spots with the levee being in design, so armoring or raising of the system would need to occur for the entire alignment, which includes 0.8 miles of levee between the left and right banks.

7.3.5.1.2. 1 ft of OT

A 1-foot overtopping depth would lead to significant portions of the levee system to be overtopping. Overtopping duration is 1 hour with a peak overtopping velocity of 12.6 ft/s. The higher velocities are unlikely to neutral to strip the grass cover, again because of the short overtopping durations. Knickpoints are expected to be the slope change at the landside toe. Lean clays and low plasticity silts along with potentially asphalt paved crests are expected to slow headcutting. Intervention is not expected to occur due to the flashiness of the system with no established flood forecasting for the watershed.

7.3.5.1.3. 2 ft of OT

At 2 feet of overtopping, the entire alignment is overtopping. Peak overtopping velocities are estimated to be 18.6 ft/s with a duration of 1.5 hours. Overtopping velocities are likely to erode grass cover and initiate headcutting at a knickpoint of the landside levee toe. Progression of headcut all the way to breach is unlikely due to the short duration and development of tailwater.

The annual probability of failure (APF) was estimated from a quantitative risk analysis (QRA) using an event tree approach. The overtopping scenarios discussed in Chapter 6 define a continuous loading relationship between peak water surface elevation and AEP. Z-variate interpolation was used to calculate the AEP for peak water surface elevations between those that define the relationship. The conditional probability of failure was estimated as a function of overtopping depth by team elicitation.

7.3.5.1.4. Nodal Estimates for 0.5 ft of OT

The 0.5-foot overtopping loading contained has an AEP of 5.56E-03 (1/180-year). The incipient overtopping location is a floodwall section at channel station 65+40 on the right bank. Given the balanced overtopping configuration of the levee profile, the 0.5-foot overtopping AEP was considered the same at the example section of 10-foot-tall levee. Unsuccessful intervention is virtually certain (1.0) as there is no flood forecasting capability for the Turkey Creek watershed, and the general flashiness of the system allows for little to no time to prepare for flood fighting. Flooding along Turkey Creek occurs within an hour of heavy rainfall given the small upstream drainage area, and flooding duration is less than 3 hours. Good grass cover on the levee slopes will be included in the specifications as part of the design which would offer some erosion resistance. The landside slopes will be designed to 1V:3H, producing peak velocities of 8.3ft/s. Slope protection failing was found to be very unlikely to unlikely (0.05) given the short duration of overtopping of 0.5 hours. The most likely headcut initiation location was determined to be at the landside levee toe at the slope change. Velocities are sufficient to initiate a headcut, but duration may not be sufficient to sustain a headcut all the way to the crest. A neutral value (0.5) was given to headcut initiation. No non-erodible features are present in the levee cross section to prevent or arrest erosion of the levee crest, so continuation was determined to be virtually certain (1.0). Breach was determined to be between very unlikely and unlikely (0.05) to progress fully through the embankment and crest with a 0.5-hour overtopping duration.

7.3.5.1.5. Nodal Estimates for 1 ft of OT

The 1-foot overtopping loading condition has an AEP of 4E-03 (1/250-year) at the LCL. Given the balanced overtopping configuration of the levee profile, the 1-foot overtopping AEP was considered the same at the example section of 10-foot-tall levee. Unsuccessful intervention is virtually certain (1.0) as there is no flood forecasting capability for the Turkey Creek watershed, and the general flashiness of the system allows for little to no time to prepare for flood fighting. Good grass cover on the levee slopes will be included in the specifications as part of the design which would offer some erosion resistance. The landside slopes will be designed to 1V:3H, producing peak velocities of 12.6 ft/s. Slope protection failing

was found to be unlikely to neutral (0.25) given the shorter duration of overtopping of 1 hour, but higher velocities than the 0.5 ft loading. The most likely headcut initiation location was determined to be at the landside levee toe at the slope change. Velocities are sufficient to initiate a headcut, but duration may not be long enough after failure of the slope protection. A neutral value (0.5) was given to headcut initiation. No non-erodible features are present in the levee cross section to prevent or arrest erosion of the levee crest, so continuation was determined to be virtually certain (1.0). Breach was determined to be between very unlikely to neutral (0.25) to progress fully through the embankment and crest with a 1-hour overtopping duration.

7.3.5.1.6. Nodal Estimates for 2 ft of OT

The 2-foot overtopping loading contained has an AEP of 2.13E-03 (1/470-year) at the LCL. Given the balanced overtopping configuration of the levee profile, the 2-foot overtopping AEP was considered the same at the example section of 10-foot-tall levee. Unsuccessful intervention is virtually certain (1.0) as there is no flood forecasting capability for the Turkey Creek watershed, and the general flashiness of the system allows for little to no time to prepare for flood fighting. Good grass cover on the levee slopes will be included in the specifications as part of the design which would offer some erosion resistance. The landside slopes will be designed to 1V:3H, producing peak velocities of 12.6 ft/s. Slope protection failing was determined to be likely (0.9) due to the significant overtopping velocities. The most likely headcut initiation location was determined to be at the landside levee toe at the slope change. Headcut initiation was given a value of likely (0.9), again because of the high velocities. No non-erodible features are present in the levee cross section to prevent or arrest erosion of the levee crest, so continuation was determined to be virtually certain (1.0). Breach was determined to be between very unlikely to neutral (0.3) to progress fully through the embankment and crest with a 1.5-hour overtopping duration.

The system response curve is tabulated in Table 84 and portrayed in Figure 74 with and without intervention. Linear interpolation was used to calculate the conditional probability of failure for peak water surface elevation between those that define the system response curve. The APF for overtopping with breach is estimated to be at most 5.1E-04 failures per year with and without intervention from the 2-foot overtopping scenario. Therefore, the estimated APF is between 3E-04 and 3E-05 failures per year with and without intervention. Calculations are provided in Appendix C.

Table 84. Overtopping system response curve for PFM 01.

Overtopping Depth (feet)	System Response Probability (with and without intervention)	APF (with and without intervention)
0.0	0	0
0.5	0.001	6.9E-06
1.0	0.031	1.3E-04
2.0	0.240	5.1E-04

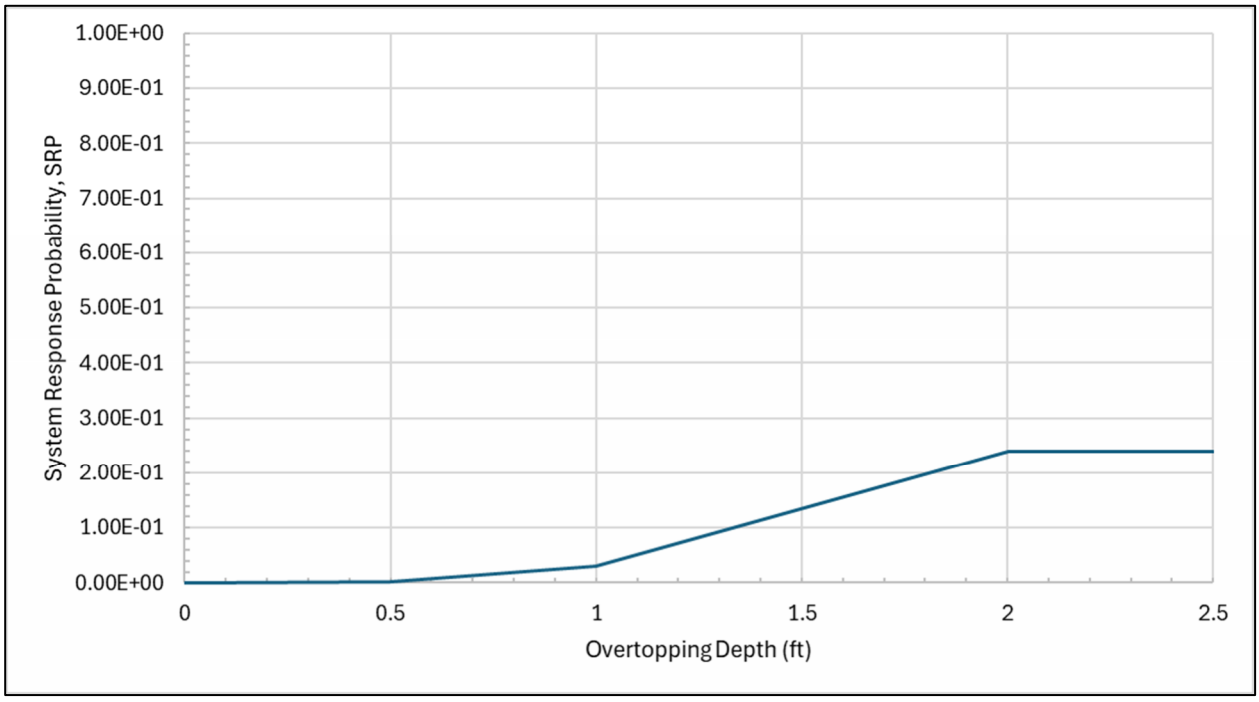


Figure 74. System response curve for PFM 01 (with and without intervention).

7.3.5.2. Uncertainty

There is major uncertainty in the hydrology due to a lack of consistent flow records anywhere along Turkey Creek, as previously discussed. The gage at Johnson Drive within the project area is not a USGS gage and does not record flow data, nor does it have an accompanying stage-discharge rating curve. Thus, an RMC-BestFit, flow frequency analysis is not possible to perform at this location. Instead, frequency flows (inflows) were developed using HEC-1, a hydrologic modeling software during the feasibility study, with Atlas 14 frequency precipitation depths for the 24-hour storm that would then be routed through the developed hydraulic model. Uncertainty is present in the duration and distribution of the frequency storm used in HEC-1 as this is often subjective and/or based on regional trends. Additionally, the Turkey Creek watershed is heavily urbanized and fully accounting for underground stormwater network routing and overland runoff is nearly impossible without significant additional data and time, which is out of the scope of this study.

There is major uncertainty in the hydraulic modeling because a lack of useable calibration data. Despite the base HEC-RAS model being set up for 2D rain-on-grid modeling, there are significant limitations to its ability to route storm network flow which could drastically affect routing timings and therefore affect peak flow magnitude, timing, and hydrograph shapes in the Turkey Creek channel. Using historical gridded precipitation to calibrate historic storms may lead to unrealistic calibration parameters by compensating for the stormwater network routing limitations. For this reason, best engineering judgement was used for selection of the calibration parameters while leaning towards more conservative modeling assumptions to account for hydraulic uncertainty in the design. Recent considerations were given to simulating 2017 storm events in the HEC-1 model, if possible, to compute inflows for the current set up of the HEC-RAS model and calibration efforts. But this was beyond the time allotted and scope of this SQRA. Highwater marks from the 2017 flood are also available for calibration purposes should these

historic inflows be computed in the future. But again, there is significant uncertainty in the historic flows and if they are representative of current conditions.

7.3.6. Incremental Life Loss

The average incremental life loss was estimated between 3 and 30.

7.3.6.1. Rationale

The mean incremental life loss estimates use the 0.5-foot OT (0 day/0 night), 1-foot OT (1 day/0 night), and 2-foot OT (1 day/0 night) life loss estimates for minimal warning. Minimal warning was used due to a lack of flood forecasting capability in the Turkey Creek watershed and the very flashy nature of the system. Evacuation routes are available in each section of the left and right bank levee systems both by vehicle and by foot due to the small size of the leveed area. The leveed areas are primarily commercial and industrial which leads to little nighttime population at risk. Historic floods show no recorded life loss along the upper portions of Turkey Creek. Generally, life loss has occurred in adjacent urban watersheds during the same storm events.

The incremental life safety risk was estimated from a QRA, building on the branches described for APR above. The overtopping scenarios discussed in Chapter 6 define a continuous consequence relationship between peak water surface elevation and incremental life loss. The breach life loss for 1-foot of overtopping was assumed for incipient overtopping (at top of levee) due to potential differences in breach parameters and warning assumptions associated with the breach prior to overtopping scenario at top of levee (100% levee height loading). Linear interpolation was used to calculate the incremental life loss for peak water surface elevations between those that define the relationship. Average incremental life loss estimates were weighted by time-of-day exposure: 0.45 for daytime and 0.55 for nighttime. The incremental flood limit, where breach and non-breach life loss are equal and increment life loss is zero, was estimated by the MMC to occur at an overtopping depth of 0.5 ft (5.62E-03 AEP).

The average annual incremental life loss (AALL) was estimated to be 1.8E-04 lives per year with and without intervention. The weighted average incremental life loss per failure ($\bar{N}$) was calculated by dividing the AALL by the APF resulting in 0.2 lives per failure with and without intervention. Therefore, the estimated average incremental life loss is between 0.1 and 1 lives per failure with and without intervention. Calculations are provided in Appendix C.

7.3.6.2. Uncertainty

Discuss any sources of significant uncertainty related to the average incremental life loss which includes the hydraulic modeling (e.g., breach parameters and flood characteristics) and consequence modeling (e.g., emergency preparedness, warning and evacuation, etc.).

7.3.7. Incremental Economic Cost

The average incremental economic cost was estimated between $300K and $3M.

7.3.7.1. Rationale

The incremental economic risk was estimated from a QRA, building on the branches described for APF above. The overtopping scenarios discussed in Chapter 6 define a continuous consequence relationship between peak water surface elevation and incremental economic cost. The breach economic cost for 1 foot of overtopping was assumed for incipient overtopping (at top of levee) due to potential differences in

breach parameters associated with the breach prior to overtopping scenario at top of levee (100% levee height loading). Linear interpolation was used to calculate the incremental economic cost for peak water surface elevations between those that define the relationship. The incremental flood limit, where breach and non-breach economic cost are equal and increment economic cost is zero, was estimated by the MMC to occur at an overtopping depth of 0.5 ft (5.62E-03 AEP).

The average annual incremental economic cost (AAEC) was estimated to be $1E03 per year with and without intervention. The weighted average incremental economic cost per failure ($\bar{\$}$) was calculated by dividing the AAEC by the APF resulting in $1.96E+06 per failure with intervention and without intervention. Therefore, the estimated average incremental economic cost is between $300K and $3M per failure with intervention and without intervention. Calculations are provided in Appendix C.

7.3.7.2. Uncertainty

Discuss any sources of significant uncertainty related to the average incremental economic cost which includes the hydraulic modeling (breach parameters and flood characteristics) and consequence modeling (limitations of structure inventory, repair costs, etc.).

7.4. PFM 7: Global instability of the levee

7.4.1. Description

A flood event with an AEP of 1/130 loads the levee embankment. The water in the Upper Turkey Creek reaches the top and loads the riverside slope. This condition leads to an increase in the shear stress of the embankment and the soils underneath. The increase in pore pressures leads to a decrease in the shear strength of the soils. An initial slope instability failure occurs on the landside slope with a slip surface that goes through the embankment and blanket. The severity of this initial slope instability could be sufficient to lower the crest and to breach the embankment. If the initial slope instability is not severe enough to lower the crest, the levee could still resist the loading. However, the embankment is not designed with a zone of higher strength that could arrest retrogressive instability. Intervention to prevent stability failures will not be possible. The load on the levee due to the flood causes stability failures to advance through the embankment, lowering the crest, and leading to uncontrolled release of water into the protected area.

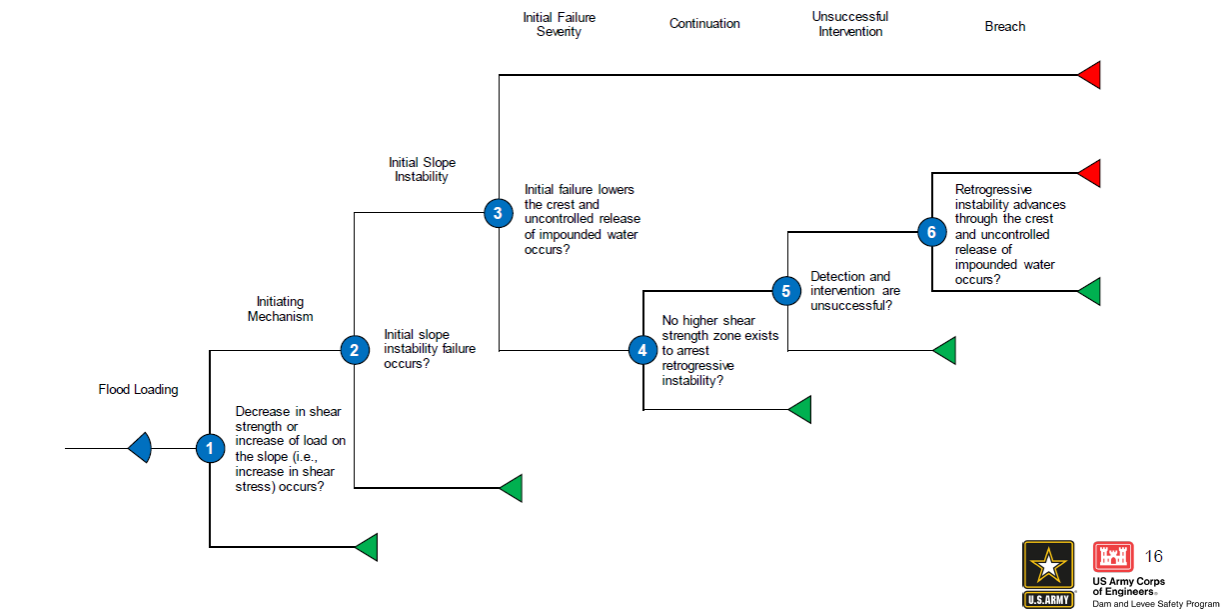


Figure 75. Event tree for PFM 07.

7.4.2. Background

The project has in its current design areas where earthen embankment levees will be used for flood control along both banks. Floodwalls are proposed for areas where there is no sufficient space for levees. The levees will consist of fine-grained materials such as lean clays (CL) or low plasticity silts (ML) in accordance with ASTM D2487. A borrow site has not been identified. The material will likely be acquired from a local commercial borrow source. Fine-grained soils with high plasticity and sandy soils will not be used. The areas where the levees will be constructed will be cleared, grubbed and stripped in preparation to place the fill. The levee will be compacted with equipment suitable for the type of material that will be used. The slopes of the levees will be 1V:3H for levees that exceed 5 feet in height, whereas levees that are less than 5 feet tall will use 1V:2H slopes. A 6-feet-wide inspection trench will be excavated along the levee alignment to find any pervious zones that have not been detected from borings. These trenches will be backfilled with the excavated material or filled with levee fill.

Figure 76 and Table 85 show the levee design reaches that were used for seepage and slope stability analyses. The levee was divided in these reaches that maintain similar site geometry and soil stratigraphy. The subsurface profiles along the creek show an increase of depth from levee to bedrock from upstream to downstream (left to right in Figure 76). The stationing of the right bank and left bank are based on a 2024 alignment.



Figure 76. Levee design reaches.

Table 85. Levee design reaches with stationing.

Reach	Left Bank Stationing	Right Bank Stationing
1	0+00 to 31+50	0+00 to 9+00
2	31+50 to 43+50	9+00 to 21+50
3	43+50 to 52+00	21+50 to 30+50
4	52+00 to 72+00	30+50 to 49+00
5	72+00 to 82+00	49+00 to 57+00

The design team performed slope stability analyses for 6 cross sections within these reaches. These analyses were conducted using GeoStudio SLOPE/W program. Representative levee cross sections within the reaches from Table 85 were used. In most cases, the tallest levee sections were analyzed. The levee height from two of these sections was 9 feet. The design of the levee crest has a width of 10 feet, but in some instances a crest of 14 feet was considered as well. The crest width did not have a negative impact on the results. The City of Merriam had expressed their desire of a 14-foot-wide crest to allow more room for pedestrians and bicyclists. Slope stability analyses were conducted to meet criteria per EM 1110-2-1913 (2000) and EM 1110-2-1902 (2003b). The water level was set at top of levee for the analyses.

The evaluated cases were end of construction with undrained soil strengths (Factor of Safety of 1.3) and steady-state with drained soil strengths (Factor of Safety of 1.4). The end-of-construction condition used undrained soil strengths from unconsolidated undrained (UU) triaxial tests. The drained soil strengths were based on effective stress plots of consolidated undrained (CU) triaxial tests with pore pressure measurements. There were difficulties obtaining soil samples for testing, which is why only 3 UU tests and 4 CU tests were performed. Due to the limited set of tests, assumptions needed to be made in the selection of the parameters and were based on values from literature and experience.

Table 86 shows the material properties used in the initial set of analyses. A total of 19 borings have been conducted in the project area, 2 of them were part of the feasibility phase in 2002 and the other 17 in 2023. The soil profile shows that the subsurface where the levees will be constructed is mainly clayey fill or clayey natural soil down to shallow bedrock with no consistent sand layer. While the soil stratigraphy does not show a continuous pervious stratum, the design team assumed for modeling purposes that there is a zone where seepage could flow through. This zone includes weathered rock, gravelly clay, and the voids and joints of bedrock.

Table 86. Material properties used in slope stability analyses.

Material	Unit Weight (pcf)	Cohesion (pcf)	Phi (degrees)	Hydraulic Conductivity (ft/s)
Embankment	120	0 (drained) 1000 (undrained)	2 (drained) 0 (undrained)	7.5E-06
Blanket	115	0 (drained) 1000 (undrained)	2 (drained) 0 (undrained)	7.5E-06
Pervious Stratum	115	0 (drained) 0 (undrained)	31 (drained) 31 (undrained)	3E-03

Table 87 shows the results of the initial slope stability analyses. Analyses were conducted for end of construction with undrained parameters and all results indicated that they meet the minimum factor of safety of 1.3. The lowest factor of safety was 4.5. However, some of the cases with steady state conditions with drained strengths did not result in minimum factors of safety of 1.4. Some of them were as low as 0.8. To be able to achieve the minimum of 1.4, some modifications to the levee design were incorporated wherever it was needed. These included stability berms, sand drains, or a combination of both as seen in Figure 77. The results of the analyses with the stability remediations are shown in the last column of Table 87.

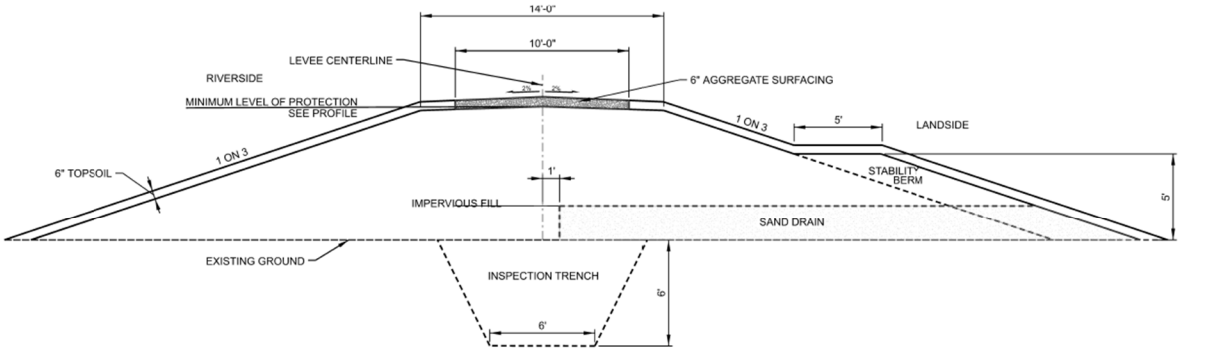


Figure 77. Typical levee section of Reach 2 LB: 40+80 showing sand drain and stability berm.

Table 87. Slope stability factors of safety for end of construction and steady state seepage conditions.

Reach	Levee Height (ft)	End of Construction Factor of Safety (Undrained)	Steady State Factor of Safety (Drained)	Steady State Factor of Safety (Drained) with Stability Remediation
Reach 1 LB: 28+00 Tallest Levee	4.0	10.4	1.4	-
Reach 1 RB: 2+00 Tallest Levee	2.5	20.8	2.1	-
Reach 2 LB: 40+80 Tallest Levee	7.0	4.5	0.8	1.6 w sand drain and 5' tall 5' wide berm or 1.6 w sand drain and 14' crest
Reach 2 RB: 18+20 Tallest Levee	9.0	4.5	0.8	1.4 with sand drain and 5' tall 10' wide berm or 1.4 with sand drain and 14' crest and 5' tall 5' wide berm
Reach 4 RB: 41+20 Tallest Levee	9.0	4.9	1.0	1.4 with sand drain
Reach 5 LB: 77+80 Tallest Levee	5.0	10.3	1.4	-

Although the remediations would increase the factor of safety of steady state, drained conditions to a minimum of 1.4, these added features require a wider footprint and higher costs. The design team proposed using the SQRA to evaluate if steady state could really be achieved for this project due to the flashy nature of the creek. If it is determined that steady state conditions are unlikely or not possible for the levees, this would allow recommending the removal of stability remediations such as the stability berms and sand drains from the levee design. Hydrographs show that water level remains shallow within the channel most of the time and that it rises and falls very quickly during floods. In addition, the fine-grained material of the levee and blanket would not have the capacity of allow sufficient flow through. Therefore, it is possible steady state conditions and drained strengths would not be applicable for the levees in this project. To verify how this assumption has an effect on the design and to inform the risk assessment, slope stability analyses with transient seepage were conducted. Transient seepage analyses are not considered reliable for design, which is why the USACE does not recommend them for initial design mostly due to the difficulty in the ability to measure the parameters for unsaturated soils. Nevertheless, transient seepage can be used in a risk assessment with sensitivity analyses to evaluate the development of pore pressures with time.

Slope stability models with transient seepage were conducted for the levee sections that did not meet the steady state factor of safety of 1.4. The models were run with these levee sections without the stability remediations to verify if they were necessary or not to meet the minimum factor of safety. While some adjustments were made to the hydraulic conductivities, the models used the same soil parameters for the most part. Volumetric water content and compressibility were estimated from consolidation tests and used for the blanket and embankment materials. For the pervious stratum, values from literature were used.

Soil–water characteristic curves (SWCC) were generated within GeoStudio. Table 88 shows the material properties used for transient seepage modeling.

Table 88. Material properties used for transient seepage analyses.

Material	Saturated Water Content	Hydraulic Conductivity (ft/s)		Compressibility (1/psf)			Rationale
		expected	high	low	expected	high	
Embankment	0.4	1.0E-06	-	3.0E-06	1.13E-05	1.96E-05	Assumed identical to blanket
Blanket	0.4	1.0E-06	1.0E-04	3.0E-06	1.13E-05	1.96E-05	Computed from 5 consolidation test results
Pervious Stratum	0.4	3.0E-03	-	-	1.13E-06	1.13E-05	Assumed medium-loose sands

Slope stability analyses with transient seepage and drained strengths combined with sensitivity analyses were performed for Reach 2 LB Sta. 40+80, Reach 2 RB Sta. 18+20 and Reach 4 RB Sta. 41+20. Adjustments were made to the hydraulic conductivity, compressibility, and initial water level in the analyses to verify how sensitive the results were to changes in these parameters. Hydrographs with the expected duration were incorporated in the models to make the analyses time dependent. Figure 78 shows an example of a hydrograph with expected duration used for Reach 2 LB Sta 40+80. All the hydrographs at the locations used for the analyses show that the rise and fall of water level between toe and levee crest takes less than 2.5 hours. The slope stability models were run with the water level at top of levee. The lowest factors of safety obtained from these slope stability models resulted in factors of safety of 1.5. In addition, the hydrographs were extended with durations that were two times and five times longer (2x and 5x) to exaggerate the duration of the water surface elevation above the waterside toe and study the influence of time in the development of pore pressures within the embankment and blanket. The risk assessment team decided to not rely on these extended hydrographs as they seemed to be unrealistic based on the available data and the nature of the watershed. Nevertheless, the cases that were run with a hydrograph that had an extended duration 2x resulted in similar factors of safety to the ones that used the expected duration. A flood duration up to twice as long as expected does not result in a significant increase in the total head at the landside toe for any cases of varying soil parameters.

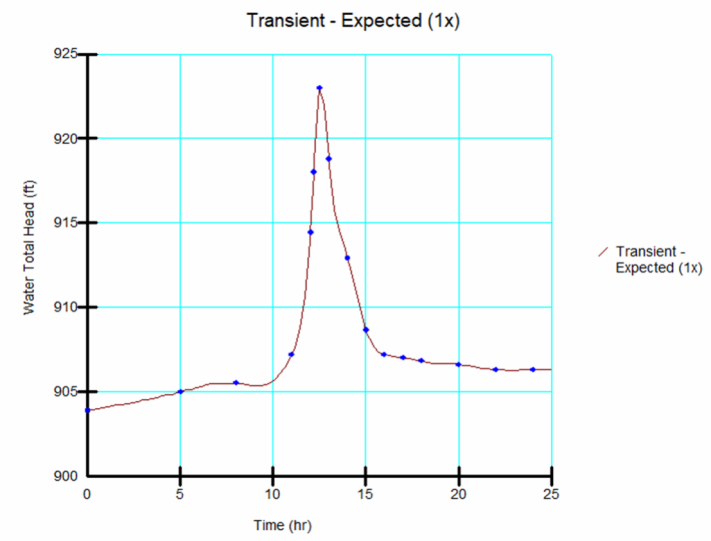


Figure 78. Hydrograph used for transient seepage in Reach 2 LB Sta 40+80

In the transient analyses, in no case did the embankment become saturated. The wetting front progressed no more than ~10% through the width of the embankment, and that was for the 5x the expected hydrograph duration. Figure 79 shows the results of the slope stability analysis at top of levee and it can be noticed from the phreatic surface line that seepage barely penetrates through the embankment. Pore water pressure development in the blanket was more significant than within the embankment. Pore water pressures in the blanket were tracked using nodal points 3 feet and 6 feet below the landside toe of the embankment, since this is the location most relevant to slope stability. The results from all the analyses show that steady state seepage conditions will not be possible in the levee. Rainfall producing the flood loading would also be present at the levee, which could lead to infiltration and affect pore water pressures. However, the lag time between the rainfall and peak flood levels is less than an hour, so it is unlikely that infiltration would be significant in the short duration before peak flooding occurs. Table 89 shows the results of the slope stability analyses with transient seepage.

In addition to evaluating whether or not steady state conditions could be achieved, the design team and SQRA team reassessed the soil strength parameters used in the original steady state analyses. Drained shear strengths were defined using the Mohr's circles obtained from consolidated undrained (CU) triaxial testing and a cohesion intercept of zero; assuming the cohesion intercept is zero is a conservative estimate for soil strengths as adding even a small amount into stability modeling can have a sizeable impact on making slopes calculate as stable. The selected strength plotted on the Mohr's circle envelope is shown in Figure 80. However, this plot was considered too conservative and underrepresented the available strength in the soils, particularly at higher stresses, given the difference between the tested effective stress envelope and the estimated effective stress envelope. In order to more accurately define the soil strength, while still maintaining a cohesion intercept of zero, a curvilinear drained strength function was used (Figure 81). Using this curvilinear drained strength function resulted in factors of safety above the required 1.4 for the steady state condition (Table 90).

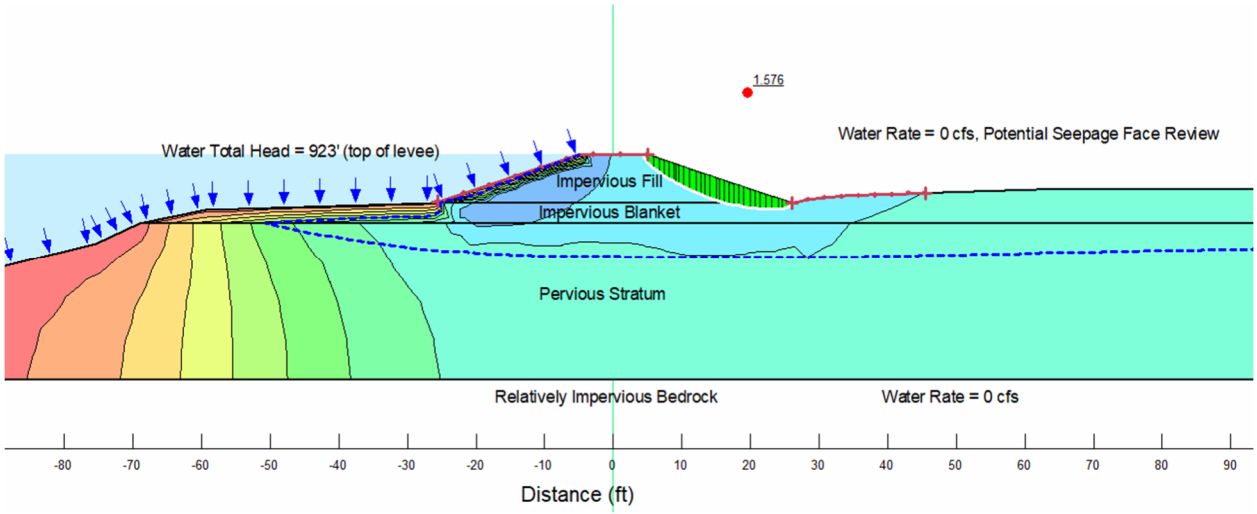


Figure 79. Slope stability analysis at top of levee in Reach 2 LB Sta 40+80

Table 89. Results of slope stability analyses with transient seepage.

Cross section	Case	Hydrograph	Factor of Safety
Reach 2 LB Sta. 40+80	impervious m_v - high initial condition = 904	Expected	1.57
		Extended (2x)	1.57
	impervious m_v - expected initial condition = 904	Expected	1.57
		Extended (2x)	1.57
	impervious m_v - low initial condition = 904	Expected	1.57
		Extended (2x)	1.57
	impervious m_v - low initial condition = 908	Expected	1.57
		Extended (2x)	1.57
	impervious m_v - low initial condition = 908 impervious k - high	Expected	1.58
		Extended (2x)	1.58
	impervious m_v - low initial condition = 908 impervious k – high pervious m_v - high	Expected	1.60
		Extended (2x)	1.60
Reach 2 RB Sta. 18+20	impervious m_v - high initial condition = 905	Expected	1.54
		Extended (2x)	1.54
	impervious m_v - expected initial condition = 905	Expected	1.50
		Extended (2x)	1.50
	impervious m_v - low initial condition = 905	Expected	1.50
		Extended (2x)	1.50
	impervious m_v - low initial condition = 908	Expected	1.50
		Extended (2x)	1.50
Reach 4 RB Sta. 41+20	impervious m_v - high initial condition = 894	Expected	1.50
		Extended (2x)	1.50
	impervious m_v - expected initial condition = 894	Expected	1.50
		Extended (2x)	1.50
	impervious m_v - low initial condition = 894	Expected	1.55
		Extended (2x)	1.55
	impervious m_v - low initial condition = 896	Expected	1.55
		Extended (2x)	1.55

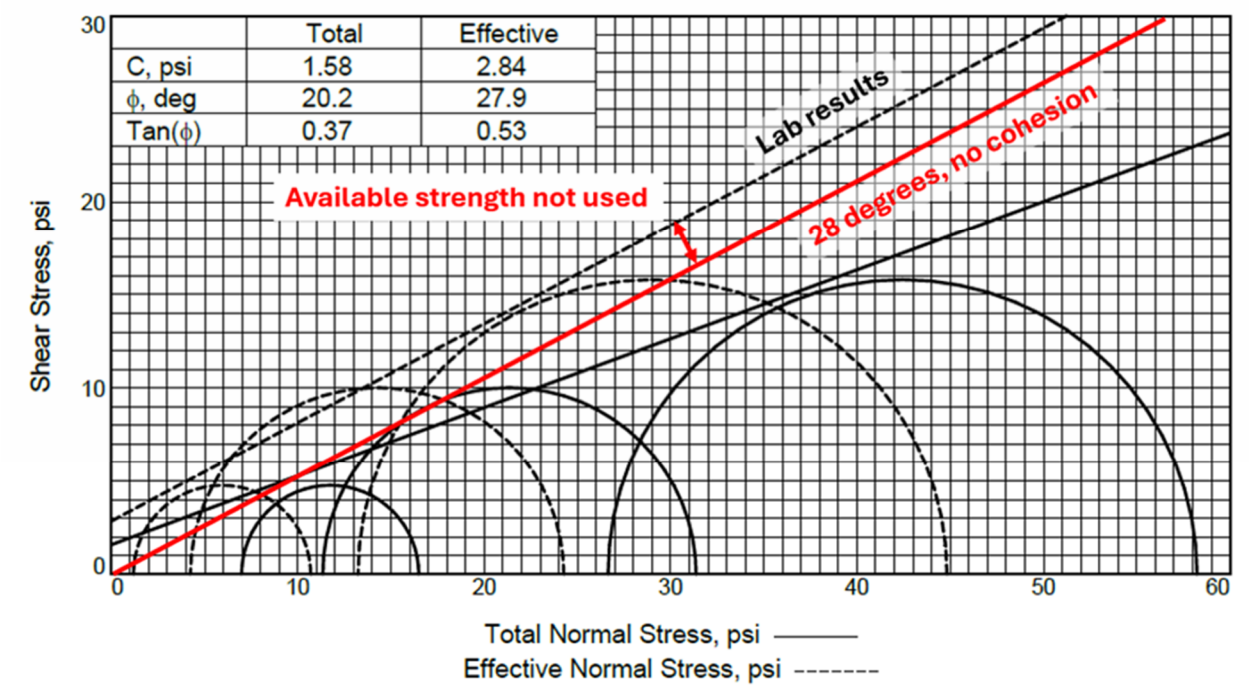


Figure 80. Drained strength function with no cohesion.

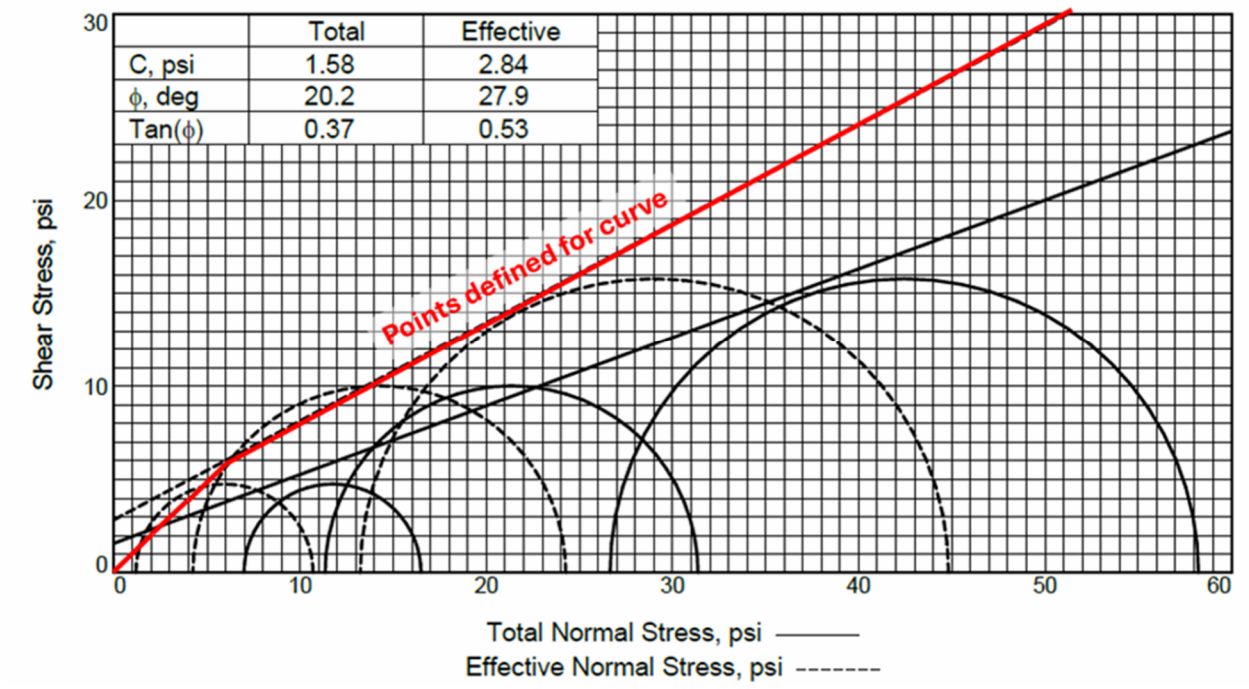


Figure 81. Curvilinear drained strength function with no cohesion.

Table 90. Revised steady state stability results.

Reach	Levee/Wall Height (feet)	Steady State (Drained) Stability Factor of Safety (1.4 Required)		
		Setback		
		1V:3H	1V:2H	1V:1.5H
Left Bank				
R1 F 13+20/UTC 0+50	11.1	NS	NS	NS
R1 L 26+80/UTC 14+50	4.8	2.7	2.7	2.4
R3 L 44+40/UTC 31+80	7.4	1.9	1.9	1.8
Right Bank				
R2 L 18+20/UTC 26+80	7.9	1.4	1.4	1.3
R4 F 33+80/UTC 41+75	10.1	NS	NS	NS
R4 L 41+40/UTC 49+50	10.1	1.9	1.9	1.8

7.4.3. Performance

The levee is in its design phase. Therefore, there is no information about its performance.

7.4.4. Intervention

Intervention is unlikely to occur during a short duration, extreme flood event. The Turkey Creek watershed does not have flood forecasting capabilities such as those available through the National Weather Service. The upstream drainage area of Turkey Creek is approximately 12 square miles and heavily urbanized. Storm events of sufficient magnitude result in very flashy floods that leave little time to prepare or respond to.

7.4.5. Likelihood of Failure

Table 91 summarizes the likelihood factors that make the PFM more likely or less likely to lead to failure.

Table 91.
Likelihood factors for PFM 07.

More Likely (Adverse) Factors	Less Likely (Favorable) Factors
• Foundation soils are relatively weak, they are not compacted/controlled fill.	• **Duration of flood loading is expected to be less than 24 hours with peak elevations lasting less than 3 hours.**
• Intervention is not possible with the short duration of flood loading and associated intense storm event.	• Majority of borings show a consistent impervious zone between base of levee and bedrock (seepage modeling assumes some amount of pervious transmission for modeling purposes).
• There is no higher strength zone within the embankment (uniform fill) that would arrest continuation of multiple slope instabilities.	• **A flood duration up to twice as long as expected does not result in a significant increase in the total head at the landside toe for any cases of varying soil parameters.**

More Likely (Adverse) Factors	Less Likely (Favorable) Factors
•	• **Flood loading is insufficient in duration to result in pore water pressure development within the fine-grained embankment beneath the crest and landside slope.**
	• Successive failures may occur as flood level is dropping which may result in crest not lowering below the current water level in quick enough of a duration.
	• Small, unregulated watershed upstream of the levee. There are no cases that could lead to extended durations of moderate to high flows.

NOTE: Key factors are shown in **bold**.

The annual probability of failure was estimated between **3E-7** and **3E-6** with and without intervention.

7.4.5.1. Rationale

Flood loading: 1/130 AEP

The initiating mechanism of slope stability failure could be a combination of an increase of load on the waterside slope with a decrease in strength of the levee due to pore pressures. While a flood could load the levee, its short duration and the low hydraulic conductivity of the embankment fill make it unlikely for the development of pore water pressures that could weaken the levee.

When the waterside slope is loaded, the probability of initiating a slope stability failure is very unlikely.

An initial slope stability failure could lower the crest and there are two potential scenarios: the instability is so severe that it lowers the crest and leads to uncontrolled release of water into the protected area (breach) or the crest is lowered but not enough for an imminent breach. The team determined that there is an equal probability that the initial slope stability failure would be severe enough to cause breach or not (neutral).

It is virtually certain that there are no higher shear strength zones in the levee design to arrest the continuation of slope instability, based on the information provided to the team. There are no plans to add a zone with a different material within the levee.

There will be no activities during a flood to intervene and prevent a slope stability failure.

Since it is certain that intervention will be unsuccessful, at this point the retrogressive instability would advance through the crest leading to uncontrolled release of water. A full breach was estimated to be unlikely due to short duration of peak loading and seepage through embankment not reaching steady state, as it was made evident from the slope stability analyses with transient seepage.

7.4.5.2. Uncertainty

While it is known that floods will be of short duration, it is uncertain if consecutive floods could saturate the levee. Also, there are some uncertainties related to the soil properties. Assumptions had to be made for modeling related to shear strength, hydraulic conductivity, compressibility, etc. due to limited set of tests. In addition, the levee has not been constructed, and a burrow source location has not been identified yet.

7.4.6. Incremental Life Loss

The average incremental life loss was estimated between 0.3 and 3.

7.4.6.1. Rationale

It was estimate that the average life loss is 1 if a breach of the levee occurs in this scenario based on the consequences models.

7.4.6.2. Uncertainty

Discuss

7.4.7. Incremental Economic Cost

The average incremental economic cost was estimated between $3M and $30M.

7.4.7.1. Rationale

Synthesize

7.4.7.2. Uncertainty

Discuss.

7.5. PFM 13: Internal migration into sanitary sewer

7.5.1. Description

Turkey Creek flows through the City of Merriam, crossing many public and private utilities. One of the most prominent of these utilities is a 42” sanitary sewer main. The sanitary sewer crosses the creek bottom 3 times, the levee/floodwalls in 6 locations, and parallels the levee within the influence zone between stations 11+84 to 65+83 (Turkey Creek centerline stationing). The sanitary sewer has manholes present on the landside and riverside of the levee.

Failure of the sanitary sewer and infiltration through riverside manholes or lines located within the riverside area can move a significant amount of soil in a short time. The soil removal erodes away at the levee/floodwall footprint and causes instability. This results in a failure of the levee embankment/floodwall with associated loss of life and economic consequences. An event tree of this

failure description is shown in

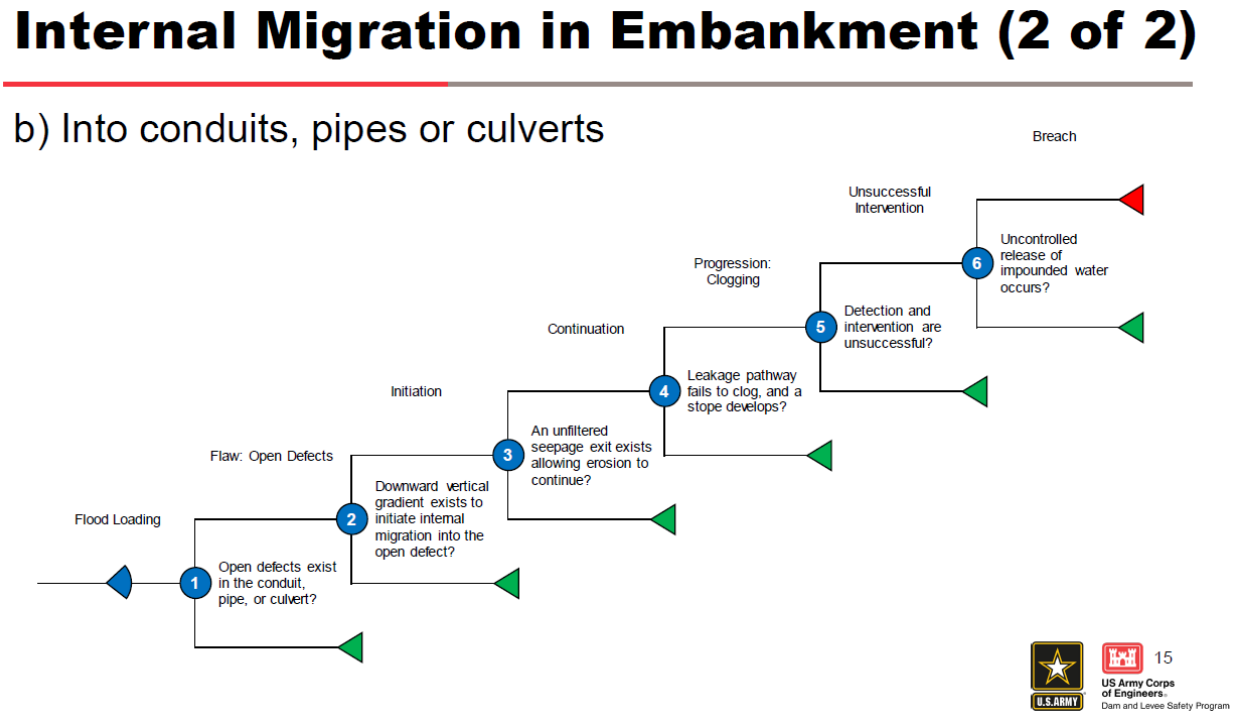


Figure 82.

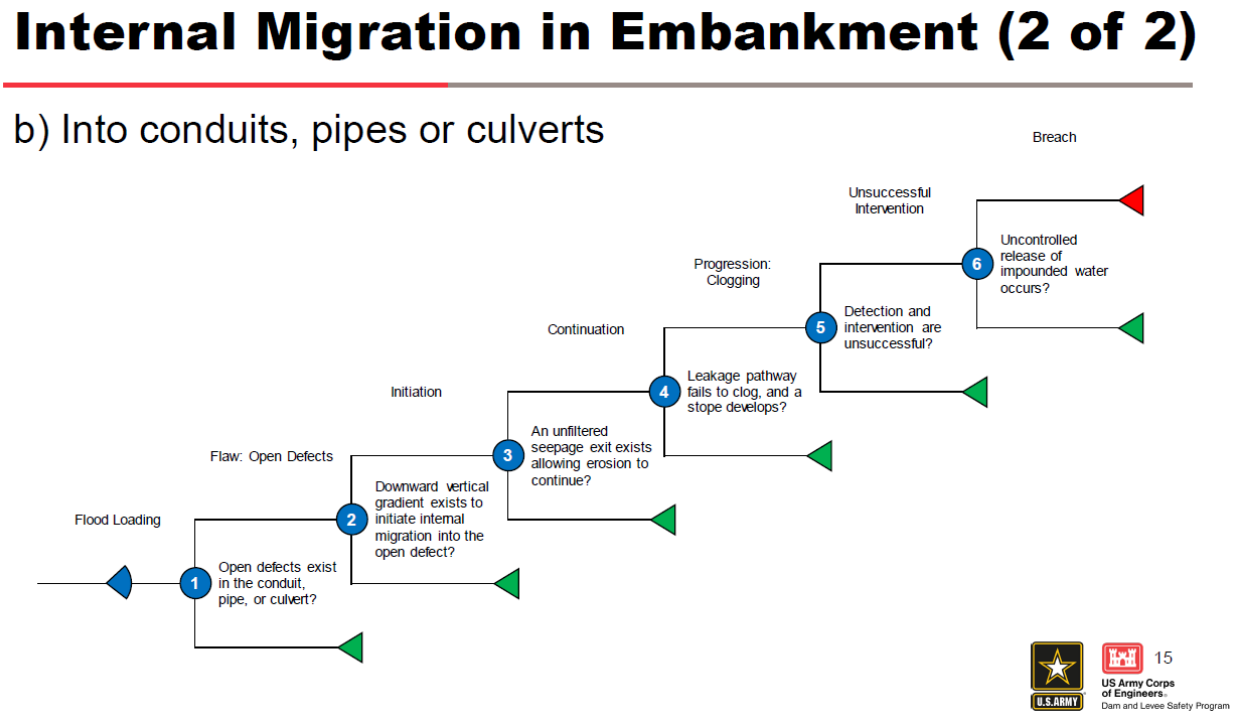


Figure 82. Event tree for PFM 13.

7.5.2. Background

In total, there are approximately 110 utilities that may impact the construction and operation of the Upper Turkey Creek levee system. The utilities include underground electric, communication/fiber, natural gas, water supply, storm sewer, and sanitary sewer. Some of the utilities have a simple crossing within the levee footprint and others run parallel in and out of the levee influence zone. The most notable of the utilities that have a PFM is a 42" sanitary sewer main.

The sanitary sewer system in this area was constructed in the 1950s and video inspected by the utility owner every 10 years with the last inspection occurring in 2014. Results from the video inspection deemed the pipe to be in good condition and not recommended for replacement.

The 42" sanitary sewer crosses creek bottom in 3 locations, the levee in 6 locations, and runs parallel to the levee for approximately 5,400 feet. The sanitary sewer is laid on or within bedrock for most of the run, as seen in the as-built in Figure 83. A crossing of the design levee and the sanitary sewer can be found below in Figure 84 with the profile in Figure 85. The floodside manhole is located 175' from the toe of the design levee. The top of design levee is approximately 927' and the top of the 42" sanitary sewer is approximately 915', leaving a potential of 12' of water loading during the maximum design flood. The natural ground surface in this area is approximately 925.5' so the construction of the levee in this area would only add approximately 1.5' of water loading to the existing pipe.

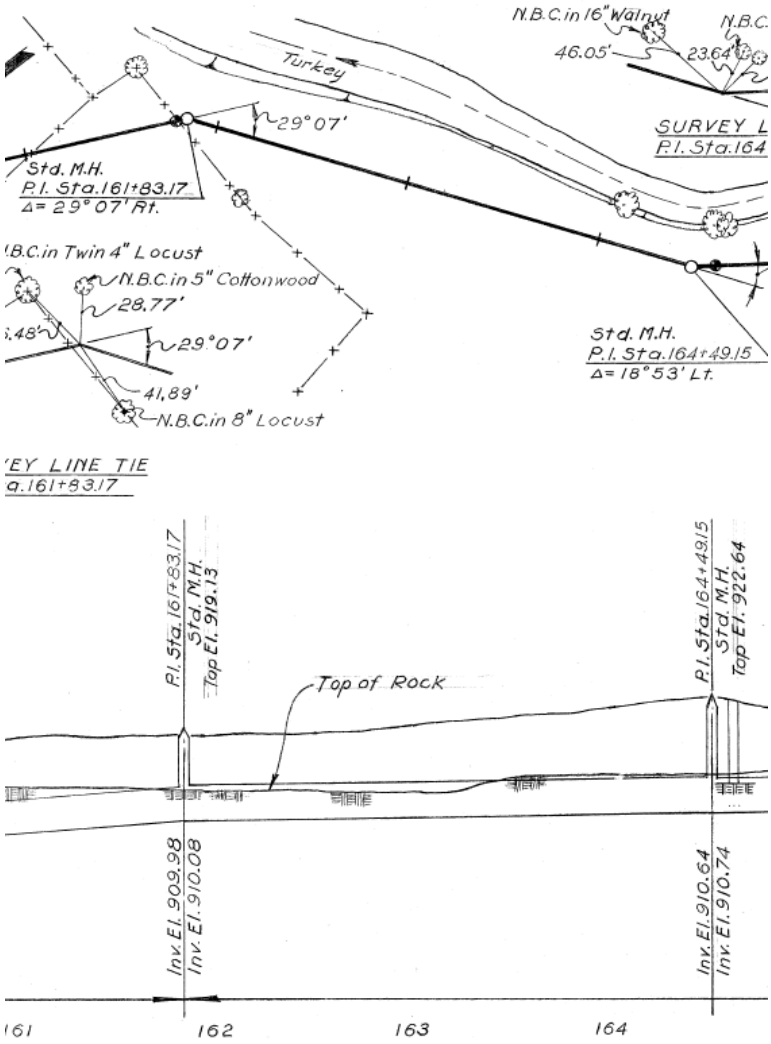


Figure 83. Sanitary Sewer As-built (1959).

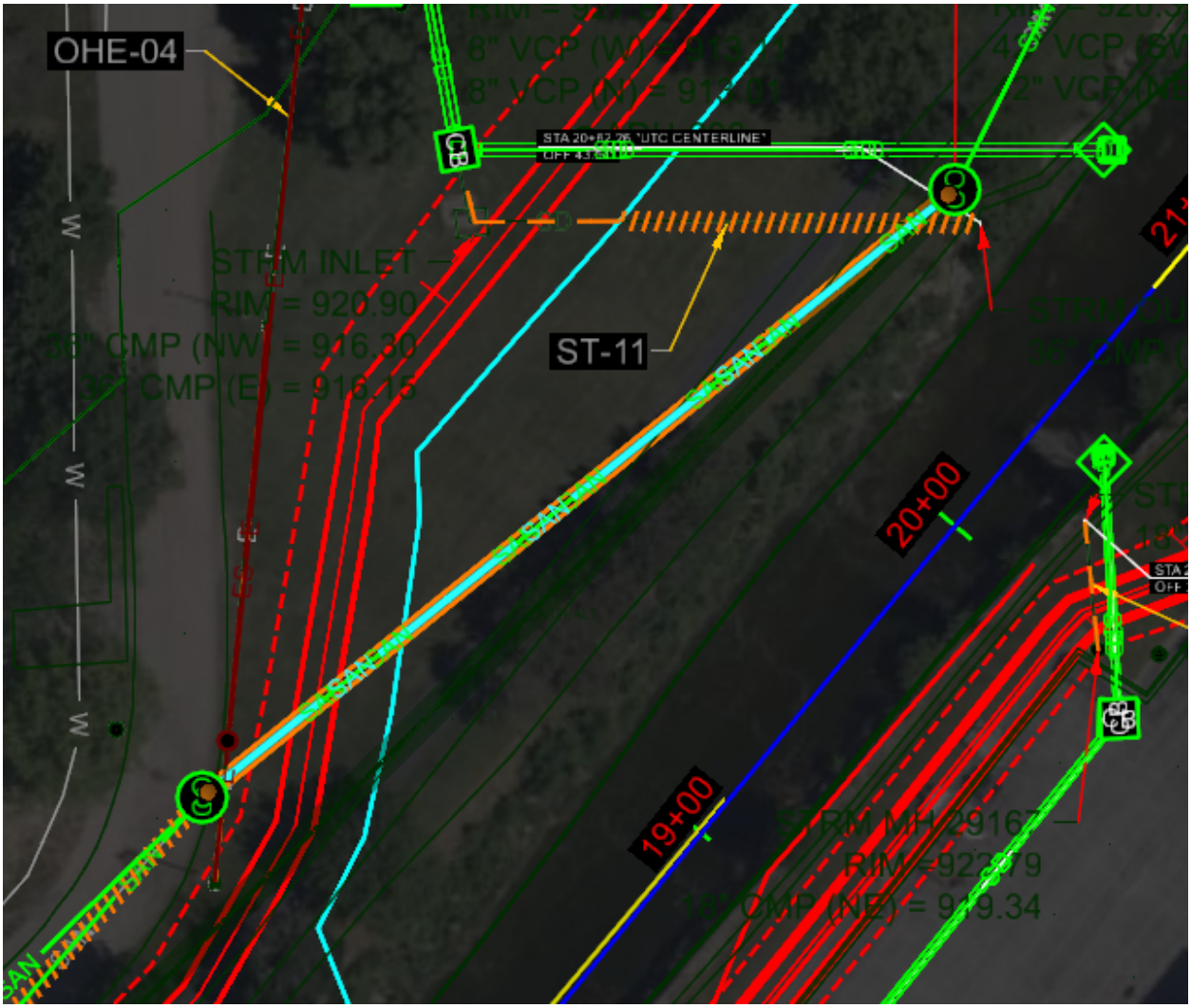


Figure 84. Example location of 42” Sanitary Sewer crossing.

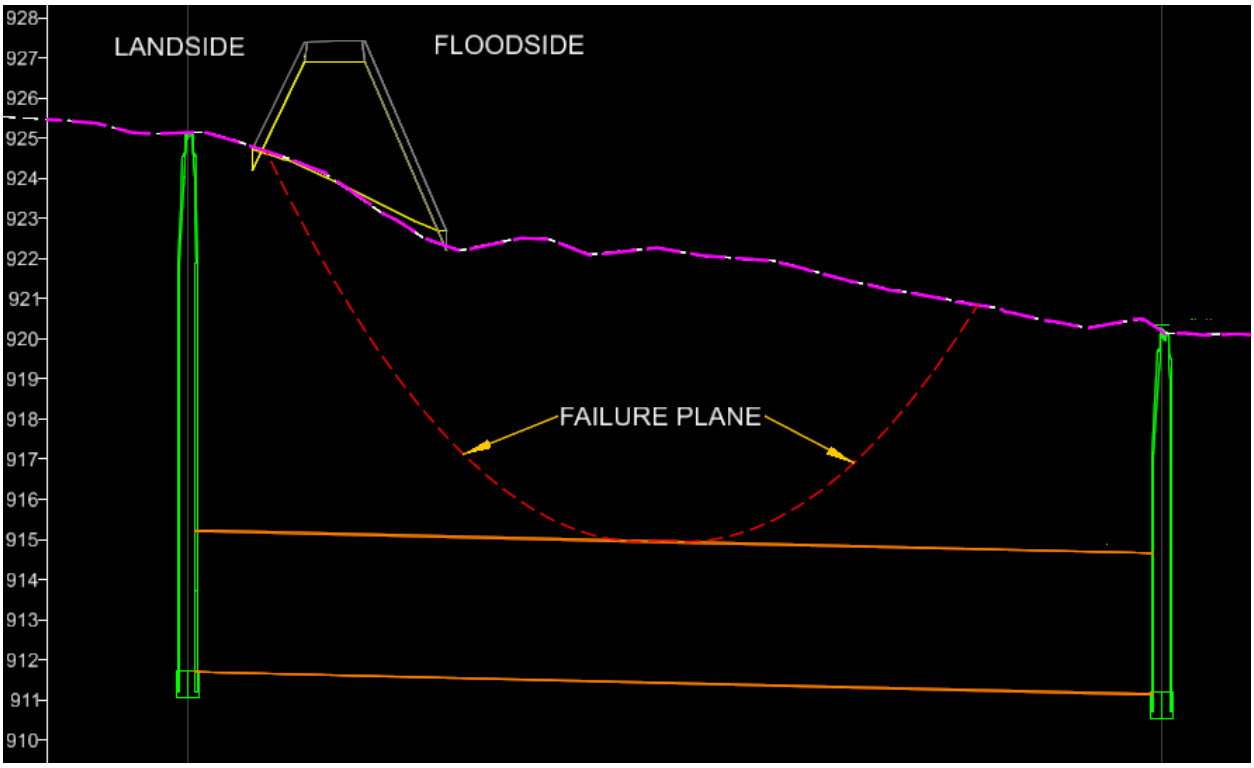


Figure 85. Failure Mode of PFM 13.

7.5.3. Performance

The levee system is in the design phase, and thusly, has no performance history. The sanitary sewers have no known history of internal migration.

7.5.4. Intervention

The first course of action is to relocate all utilities directly impacted by the construction of the levee/floodwalls. This includes shallow utilities such as underground electric, communication/fiber, natural gas, water lines, storm sewer, and sanitary sewer service lines. In some instances, these utilities will be replaced during construction instead of relocated outside of the levee influence zone. Storm sewer will be replaced with RCP and in many cases, located deeper to avoid the floodwall. All other utilities besides sanitary sewer will be relocated up and over the levee/floodwall.

Sanitary sewer lines not directly impacted by construction of the levee/floodwall will be inspected on condition of the pipe and manholes. In some instances, the sanitary sewer may need to be replaced in-kind or internally lined to mitigate the risk of failure. If the sanitary sewer is deemed to be in good condition, it may be left in-place under the levee/floodwall construction. In addition, all manholes within the floodside of the levee will have the rim raised above the 1/25 AEP. A summary of utility decisions can be found below in Table 92.

Table 92: Utility Relocation/Abandonment/Replacement Decision Matrix.

Type	Concern	Associated PFMs	Recommended measure

Existing manhole on floodside	Floodside not preferable because of access, floodside could be problem if manhole allows infiltration of material, overtopping into manhole can cause sewer backups	Internal migration into, overtopping into sewer (backflow)	Inspect all floodside manholes, line to ensure no internal migration where needed (cracks, holes, etc.), raise to above 1/25 AEP flood level, Some cases may need full manhole replacement (e.g., brick). Consider short duration of peak water levels.
Existing manhole on L/W alignment	Not desirable, but may be similar to a gatewell structure in a levee, Difficult to compact, could allow internal migration	Internal migration into	Relocate
Existing manhole within 15-feet of toe	Could allow internal migration	Internal migration into	Inspect, line to ensure no internal migration where needed
Channel-crossing pipe	Internal migration into pipe could occur on floodside of L/W, although DIP has 100-year design life	Internal migration into	Televise inspect all crossing pipes, verify age and material, line if any defects are found. There are 6 sanitary pipes crossing the channel.
Through-pipe under levee	Internal migration could occur due to additional weight of levee of levee	Collapse, internal migration into	Inspect per EM 1110-2-2902 Section 6.4.5, Similar to the channel crossing pipe but with additional soil load
Through-pipe under floodwall	Internal migration could occur due to additional weight of floodwall	Collapse, internal migration into	Inspect per EM 1110-2-2902 Section 6.4.5. Similar to the channel crossing pipe but with additional soil load and/or sheet pile driving
Parallel pipe under levee	Internal migration could occur due to additional weight of levee, RCP pipe could crack (100-year design life)	Internal migration into, collapse	Controlled by manhole relocation, Inspect per EM 1110-2-2902 Section 6.4.6. Line if in poor condition. Try to avoid this alignment if possible.

Parallel pipe under floodwall	Internal migration could occur due to additional weight of floodwall, RCP pipe could crack (100-year design life)	Internal migration into, collapse	Inspect per EM 1110-2-2902 Section 6.4.6. Line if in poor condition. Try to avoid this alignment if possible
Parallel pipe within 15-feet of levee toe	Internal migration into	Internal migration into	Inspect per EM 1110-2-2902 Section 6.4.6. Line if in poor condition.
Parallel pipe within 15-feet of floodwall toe	Internal migration into	Internal migration into	Inspect per EM 1110-2-2902 Section 6.4.6. Line if in poor condition.
Parallel pipes outside 15 feet	No concern	No PFM	No action

*Pressurized pipes that are parallel (water, gas, etc.) need to be relocated outside the levee footprint (PFM 35)

7.5.5. Likelihood of Failure

Table 93 summarizes the likelihood factors that make the PFM more likely or less likely to lead to failure.

Table 93.
Likelihood factors for PFM 13.

More Likely (Adverse) Factors	Less Likely (Favorable) Factors
• Pipes installed are more than 50 years old, so may have some deterioration leading to flaws.	• Short duration of flood loading (< 24 hours, <3 hour peak loading) may not allow for progression to failure
• Pipes are not frequently expected and have not been inspected since 2014. Pipes are third-party pipes not maintained by the sponsor.	• A flaw at the channel crossing pipe would be further from the alignment, so not as likely to directly collapse the levee/wall
• Intervention during a single event is very unlikely given the short duration of the storm.	• Pipe material ranges from ductile iron pipe (DIP). Clay, concrete, and PVC with design life ranging from 75-150 years, and may still be in good condition (~50 year old pipes).
• A 1/10 AEP water surface would have approximately 13.5 feet of head over the pipe. A 1/100 AEP water surface would have 17 feet of head. The vertical gradient may be sufficient to move material.	• Last inspection (2014) showed pipes in good condition and pipes will be televised prior to adopting them into the project. Any major defects identified will be mitigated (e.g., slip line).
• Exit is unfiltered as sanitary line can continue to move material away from the site with gravity.	• Could be possible to identify initiation and progression of this type of PFM between successive floods. Requires two ore more events greater than 1/5 AEP (out of bank) to 1/10 AEP (toe elevation).
• Pipes are up to 42” diameter, so may not clog with material	• Clay material above the pipe is less likely to migrate than other materials.

NOTE: Key factors that drove the likelihood of failure are shown in **bold**.

The annual probability of failure was estimated between 1E-05 and 1E-04 (with and without intervention).

7.5.5.1. Rationale

Flaw – assumes that the pipe has been inspected and shows no obvious defects. Over the levee design life, these pipes may be reaching the end of their design life and may not be inspected as frequently as required (i.e., every five years). It is unlikely there is a flaw, but possible.

Initiation – short duration of flood may not lead to the flaw experiencing the full hydraulic head due to the clay material, but still have some amount of head that make initiation neutral.

Continuation – the pipe can carry material along the pipe with gravity with no real limiting point that would stop material from moving, so estimated to be likely.

Progression: clogging – The flaw in the pipe, the size of the pipe, and the type of material may not efficiently move into the flaw at a fast enough rate given the flood duration, so estimated to be neutral.

Unsuccessful intervention – repairs between floods are possible but intervention is not possible during a single event. The duration and peak duration are very short and would be occurring along with intense rainfall.

Breach – flood peak may have receded below the toe by the time of failure.

Loading	1/130
Flaw	0.1
Initiation	0.5
Continuation	0.9
Progression	0.5
Unsuccessful Intervention	0.999
Breach	0.1
Total APF	2E-05

7.5.5.2. Uncertainty

Discuss any sources of significant uncertainty related to the likelihood of failure, which includes both the hazard (loading) and performance (conditional probability of failure). Reference Chapters 4 and 5 for sources of significant uncertainty with the hydrologic and seismic hazards, respectively.

7.5.6. Incremental Life Loss

The average incremental life loss was estimated between 0.3 and 3.

7.5.6.1. Rationale

Synthesize the consequence estimates, mapping products, hydraulic and consequence modeling assumptions to build the case for the average incremental life loss. These include breach dimensions and timing, depths, incremental flows and depths at consequences areas, and arrival times. Discuss their appropriateness for the identified PFMs and site-specific conditions which make the life loss estimate better or worse than the modeled assumptions and estimated life loss.

If the average incremental life loss is very similar or the same as a previously evaluated PFM, do not repeat the information. Reference the previously evaluated PFM (e.g., the breach characteristics and thus the rationale are similar to PFM ##) or discuss what is different for the evaluated PFM.

7.5.6.2. Uncertainty

Discuss any sources of significant uncertainty related to the average incremental life loss which includes the hydraulic modeling (e.g., breach parameters and flood characteristics) and consequence modeling (e.g., emergency preparedness, warning and evacuation, etc.).

7.5.7. Incremental Economic Cost

The average incremental economic cost was estimated between _____ and _____.

7.5.7.1. Rationale

Synthesize the consequence estimates, mapping products, hydraulic and consequence modeling assumptions to build the case for the average incremental economic cost. Discuss their appropriateness for the identified PFMs and site-specific conditions which make the economic cost estimate better or worse than the modeled assumptions and estimated economic cost. Include repair or replacement costs if appropriate and the resources used to estimate them. Include any benefits foregone with the direct property damages and describe the period during which the project is considered out of service for the lost benefits estimate.

If the average incremental economic cost is very similar or the same as a previously evaluated PFM, do not repeat the information. Reference the previously evaluated PFM (e.g., the breach characteristics and thus the rationale are similar to PFM ##) or discuss what is different for the evaluated PFM.

Provide the loss of service economic cost separately because DSAC only considers risks associated with breach and uncontrolled release.

7.5.7.2. Uncertainty

Discuss any sources of significant uncertainty related to the average incremental economic cost which includes the hydraulic modeling (breach parameters and flood characteristics) and consequence modeling (limitations of structure inventory, repair costs, etc.).

8. References

Electric Power Research Institute (EPRI), U.S. Department of Energy (DOE), and U.S. Nuclear Regulatory Commission (NRC). (2012). *Central and eastern United States seismic source characterization for nuclear facilities*. http://www.ceus-ssc.com/.

Larkin. (2005). *DRAFT Northeast Johnson County Watershed Study.* Johnson Count, Kansas Stormwater Management Program. Larkin Group Inc. Kansas City, MO. May 2005.

McPhillips, D.F., Herrick, J.A., Ahdi, S., Yong, A.K., and Haefner, S. (2020), *Updated compilation of VS30 data for the United States*. U.S. Geological Survey data release. https://doi.org/10.5066/P9H5QEAC.

NOAA. *NGS Coordinate Conversion and Transformation Tool (NCAT)*. National Geodetic Survey. https://www.ngs.noaa.gov/NCAT/

Petersen, M.D., Mueller, C.S., Moschetti, M.P., Hoover, S.M., Llenos, A.L., Ellsworth, W.L., Michael, A.J., Rubinstein, J.L., McGarr, A.F., and Rukstales, K.S. (2016). U.S. Geological Survey Open-File Report 2016-1035. *2016 one-year seismic hazard forecast for the central and eastern United States from induced and natural earthquakes* (ver. 1.1, June 2016). https://pubs.usgs.gov/of/2016/1035/ofr20161035ver1_1.pdf.

Petersen, M.D., Mueller, C.S., Moschetti, M.P., Hoover, S.M., Shumway, A.M., McNamara, D.E., Williams, R.A., Llenos, A.L., Ellsworth, W.L., Michael, A.J., Rubinstein, J.L., McGarr, A.F., and Rukstales, K.S. (2017). 2017 one-year seismic hazard forecast for the central and eastern United States from induced and natural earthquakes. *Seismological Research Letters*. 88(3), 772-783. https://doi.org/10.1785/0220170005.

Petersen, M.D., Mueller, C.S., Moschetti, M.P., Hoover, S.M., Rukstales, K.S., McNamara, D.E., Williams, R.A., Shumway, A.M., Powers, P.M., Earle, P.S., Llenos, A.L., Michael, A.J., Rubinstein, J.L., Norbeck, J.H., and Cochran, E.S. (2018). 2018 one-year seismic hazard forecast for the central and eastern United States from induced and natural earthquakes. *Seismological Research Letters*. 89(3), 1049-1061. https://doi.org/10.1785/0220180005.

Petersen, M.D., Shumway, A.M., Powers, P.M., Mueller, C.S., Moschetti, M.P., Frankel, A.D., Rezaeian, S., McNamara, D.E., Luco, N., Boyd, O.S., Rukstales, K.S., Jaiswal, K.S., Thompson, E.M., Hoover, S.M., Clayton, B.S., Field, E.H., and Zeng, Y. (2020). The 2018 update of the U.S. National Seismic Hazard Model: Overview of model and implications. *Earthquake Spectra*. Earthquake Engineering Research Institute. 36(1), 5-41. https://doi.org/10.1177/8755293019878199.

Stantec. (2024). *Turkey Creek and Brush Creek 2D Watershed Hydrologic and Hydraulics Report.* Stantec. Overland Park, KS. July 2024.

StormWatch. *County Wide Flood Warning System*. TU08 Lamar Avenue at Turkey Creek. StormWatch. https://www.stormwatch.com/site/?site_id=109&site=b031e79e-f6dd-4909-a39f-ae6d69f96200

StormWatch. *County Wide Flood Warning System*. TU08 Johnson Drive at Turkey Creek. StormWatch. https://www.stormwatch.com/site/?site_id=111&site=bd978dbb-c74b-4bb9-a5f5-626a128371ec

StormWatch. (2025). Observed Instantaneous Period of Record Stage, StormWatch Gage Johnson Drive at Turkey Creek TU04. (1998 to 2025). Johnson County, KS.

StormWatch. (2025). Observed Instantaneous Period of Record Stage, StormWatch Gage Lamar Avenue at Turkey Creek TU08. (1998 to 2025). Johnson County, KS.

U.S. Bureau of Reclamation and U.S. Army Corps of Engineers (USACE). *Best practices in dam and levee safety risk analysis*. https://publibrary.planusace.us/#/series/Best%20Practices-Manual.

USACE. (1968). Turkey Creek Merriam, Kansas, Design Memorandum No.1 Project Economics. U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. May 1968.

USACE. (1996) Engineering Manual No. 1110-2-1619. Risk-Based Analysis for Flood Damage Reduction Studies. U.S. Army Corps of Engineers, Headquarters. Washington, DC. August 1996.

USACE. (1998). Feasibility Report and Environmental Assessment. U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. December 1998.

USACE. (2000). *Design and Construction of Levees* (Engineer Manual [EM] 1110-2-1913).

USACE. (2003a). Turkey Creek Basin General Reevaluation Report and Environmental Assessment. U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. January 2003.

USACE. (2003b). *Slope Stability* (Engineer Manual [EM] 1110-2-1902).

USACE. (2005). *Engineering and Design: Stability Analysis of Concrete Structures* (Engineer Manual [EM] 1110-2-2100).

USACE. (2015). Upper Turkey Creek Feasibility Report with Integrated Environmental Assessment. U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. March 2015.

USACE. (2018). *SQRA calculation methodology* (Risk Management Center [RMC] Technical Note [TN] 2018-01). https://www.iwrlibrary.us/document/4e148d75-116e-4129-a9d3-2025191f3b09.

USACE. (2018). Turkey Creek 2018 Sediment Analysis. U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. November 2018.

USACE. (2018). Turkey Creek Summer 2017 Flood Events. U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. December 2018.

USACE. (2020). *Engineering and Design: Conduits, Pipes, and Culverts Associated with Dams and Levee* (Engineer Manual [EM] 1110-2-2902).

USACE. (2021). *USACE levee safety program* (Engineer Circular [EC] 1165-2-218). https://www.publications.usace.army.mil/Portals/76/Users/182/86/2486/EC%201165-2-218.pdf

USACE (2022). Appendix 3.1.3: Application of simplified physical breach method. *FY2023 standard operating procedure for dams.* Modeling, Mapping and Consequences (MMC) Production Center. https://usace.dps.mil/:b:/r/sites/KMP-MMC/Shared%20Documents/MMC%20Standard%20Operating%20Procedures/FY2023/Appendices/3.0-

Modeling/3.1.4_MMC_Application_of_Simplified_Physical_Breach_Method.pdf?csf=1&web=1&e=d7V45W.

USACE. (2022). *Engineering and Design: Floodwalls and Other Hydraulic and Retaining Walls* (Engineer Manual [EM] 1110-2-2502).

USACE. (2022) Modeling, Mapping, and Consequences Technical Manual. Technical Manual for Levees. U.S. Army Corps of Engineers, Headquarters. Washington, DC. June 2022.

USACE. (2023a). *Interim methodology for individual risk.* IWR. RMC. pw:\\140.194.161.13:RMC01\Documents\Technical Library\Templates and Examples\Periodic Assessment\Interim Methodology for Individual Risk 20230209.

USACE. (2023b). *Summary of costs associated with levee-related activities*. Levee Safety Center. https://mmc.sec.usace.army.mil/NLSP_website/NLSP_LeveeCostBrochure_FINAL_NOV2023.pdf.

USACE. (2023c). DLS-SOP-2023-03. *PFMA and SQRA methodology*. RMC.

USACE. (2023d). DLS-SOP-2023-04. *Periodic assessments*. RMC.

USACE. (2024). *Engineering and Design: Strength Design for Reinforced Concrete Hydraulic Structures* (Engineer Manual [EM] 1110-2-2104).

USACE. (2024) *Engineering Regulation No. 1110-2-1806*. Earthquake Design and Evaluation for Civil Works Projects. U.S. Army Corps of Engineers, Headquarters. Washington, DC. June 2024

USACE. (2024) *FY2025 Standard Operating Procedure for Levees.* Risk-informed Design Levees. U.S. Army Corps of Engineers, Headquarters. Washington, DC. March 2024.

USACE. (2024). *DRAFT Upper Turkey Creek USACE and Stantec Models Comparison.* U.S. Army Corps of Engineers, Kanas City District (NWK). Kansas City, MO. May 2024.

USGS. *Earthquake catalog*. https://earthquake.usgs.gov/earthquakes/search/.

USGS. *Earthquake hazard toolbox*. https://earthquake.usgs.gov/nshmp/.

USGS. *Quaternary faults*. https://doi.org/10.5066/F7S75FJM.

USGS. *A compilation of VS30 values in the United States*. https://earthquake.usgs.gov/data/vs30/us/.

USGS. *National Water Information System: Web Interface*. USGS 06892800 Turkey Creek at Merriam, KS. United States Geological Survey. https://waterdata.usgs.gov/nwis/inventory/?site_no=06892800

USGS. *National Water Information System: Web Interface*. USGS 06892900 Turkey Creek at Mission, KS. United States Geological Survey. https://waterdata.usgs.gov/nwis/uv?site_no=06892900&legacy=1

USGS. *National Water Information System: Web Interface*. USGS 06892940 Turkey Creek at Kansas City, KS. United States Geological Survey. https://waterdata.usgs.gov/nwis/inventory/?site_no=06892940

USGS. (2025). Peak Annual Period of Record Flow, USGS Gage 06892800. (1974 to 1986, 2006-2025). USGS Kansas Water Science Center.

USGS. (2025). Peak Annual Period of Record Flow, USGS Gage 06892900. (2022 to 2025). USGS Kansas Water Science Center.

USGS. (2025). Peak Annual Period of Record Flow, USGS Gage 06892940. (1974 to 1987). USGS Kansas Water Science Center.

Appendix A: Excluded Potential Failure Modes

The following sections summarize the potential failure modes that were excluded from further consideration because they were deemed non-credible or credible but non-risk drivers.

A.1. PFM 1: Overtopping breach of the levee

The team determined that this PFM is a risk driver, see Chapter 7.

A.2. PFM 2: Overtopping breach of the floodwall (I-wall)

PFM 2 requires erosion on the landside toe of the floodwall due to overtopping for sufficient duration. Plunging flow from overtopping would scour erodible material on the landside toe causing the I-wall to become unstable and collapse suddenly, allowing uncontrolled flow into the leveed area.

This failure mode was determined to be a non-risk driver based on design recommendations. Currently most of the locations being considered for I-walls (less than 7-feet tall) have asphalt or concrete paving on the landside toe, which can better withstand short durations of overtopping plunging. Whether that be parking lots, sidewalks, or roads. Because I-walls are only going to be used in locations where the floodwall would need to less than 7-feet tall, plunging forces are not anticipated to be significant. Additionally, the duration of overtopping flows is expected to be less than 2 hours, as shown in Section 4.4.5, which would limit the possibility of extended scour progression. Tailwater is also expected to develop to at least 1.5-feet through most of the levee for a 2-foot overtopping scenario which would further inhibit toe scour. Figure 86 provides an example of the tailwater depth during peak overtopping at a 6.5-foot-tall section of floodwall.

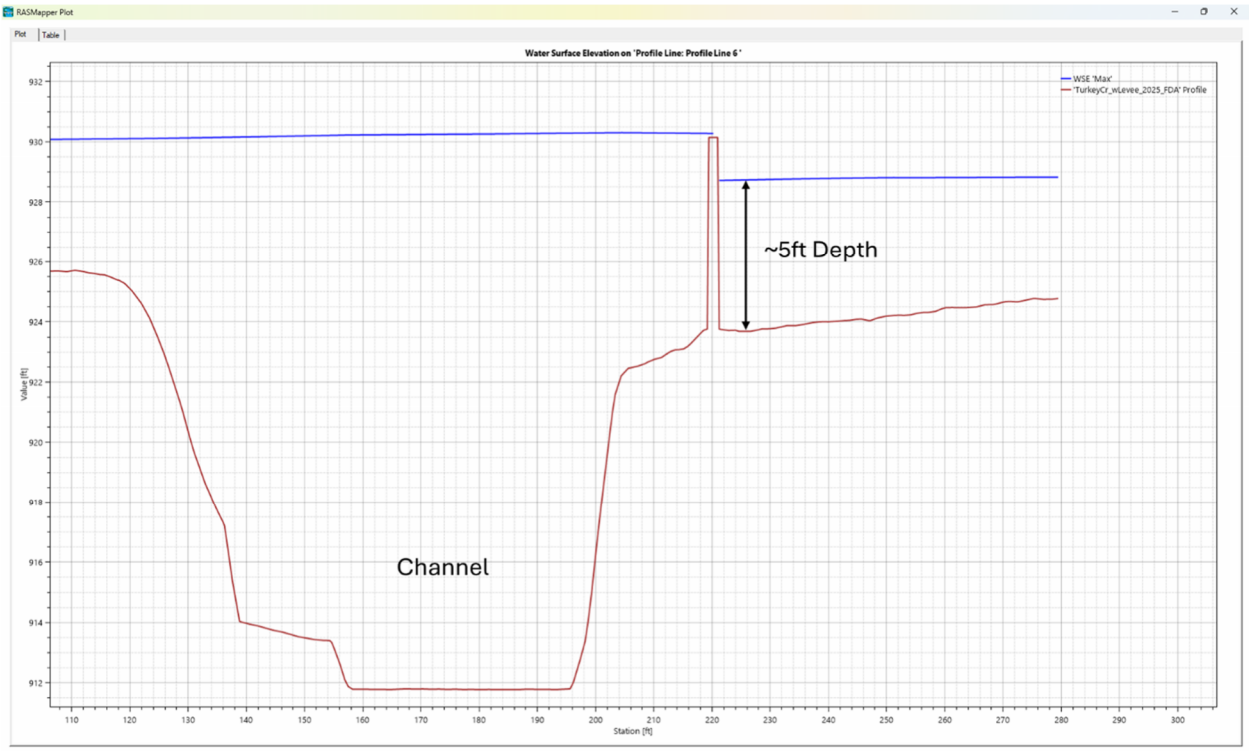


Figure 86. Tailwater development at floodwall with 2-foot overtopping scenario.

Locations where grass is present on the landside toe and I-walls are being used instead of T-walls, it is recommended that riprap be used for armoring. With this information in mind, risk of an overtopping breach for I-walls is expected to be no greater than overtopping breach of the levee (PFM1).

A.3. PFM 3: Concentrated leak erosion (CLE) through the embankment between construction lifts

This potential failure mode considers the potential for CLE to occur through a continuous flaw in the embankment between construction lifts. For the flaw in this potential failure mode to exist and allow CLE to initiate and progress, it is dependent on a number of factors, including poor compaction of the embankment, inadequate embankment material controls, and extended duration for a high loading event that maintains a high differential head.

Specifications have not yet been drafted for this project. Specification 35 41 00 Levee Construction will be the primary specification for construction of the earthen embankment. This specification contains adequate detail for the requirements of materials, placement, compaction, moisture control, scarification, and more pertaining to the construction of the levee. These requirements are expected to eliminate the concern for a continuous flaw through the embankment that would be susceptible to CLE initiation and progression.

The levee loading duration is expected to be less than 24 hours in total for a top of levee loading event. This is not considered a long enough time to allow for CLE to progress in any significant manner, let alone to failure, regardless of if CLE is even able to initiate due to poor construction practices. For these reasons, this potential failure mode was excluded.

A.4. PFM 4: CLE along differential settlement

This potential failure mode considers the potential for CLE to occur through a continuous flaw in the embankment that is created through differential settlement of the embankment. The development of a continuous flaw in this potential failure mode is dependent on several possible factors, including a relatively sudden change in earthen embankment fill height, inconsistent subsurface conditions (localized low stress zones, karstic bedrock features, sudden undulations in bedrock), and/or poor construction practices. For this failure mode to initiate and progress to failure, an extended loading duration near the top of the levee is required to maintain a high differential head.

Based on the 2023 site investigation bore logs and a desktop survey of available historical data, bedrock appears to dip from upstream to downstream in a relatively consistent manner and is not expected to contain sudden undulations. Additionally, the bedrock geology is not known for having karstic or other features that could accept or pass along any meaningful quantity of foundation material that would result in unexpected settlement.

Construction practices are expected to be adequately defined in Specification 35 41 00, which has not yet been drafted, but will be the primary specification for construction of the earthen embankment. This specification contains adequate detail for the requirements of materials, placement, compaction, moisture control, scarification, and more pertaining to the construction of the levee.

A settlement analysis has not yet been conducted but will be completed as part of the design requirements for this project. Any concerns that arise from this analysis about the compressibility of the fill materials and native clays will be addressed during design. To further ensure satisfactory conditions in embankment's foundation, an inspection trench will be dug beneath all earthen levee segments during construction and evaluated for any localized anomalies.

The levee loading duration is expected to be less than 24 hours in total for a top of levee loading event. This is not considered a long enough time to allow for CLE to progress in any significant manner, let alone to failure, regardless of if CLE is even able to initiate due to poor construction practices. For all of these reasons, this potential failure mode was excluded.

A.5. PFM 5: CLE along levee/wall/bridge interfaces

This potential failure mode considers the potential for CLE to occur along the interface of the earthen embankment and structure (floodwall, bridge abutment). Factors that may influence the existence of a continuous flaw at such an interface include poor construction practices and/or differential settlement. For this failure mode to initiate and progress to failure, an extended loading duration near the top of the levee is required to maintain a high differential head.

As stated in PFM 4 and 5, construction practices are expected to be adequately defined in Specification 35 41 00, which has not yet been drafted. In addition to requirements for construction of the earthen embankment, this specification includes additional controls for placement and compaction of embankment materials adjacent to structures to ensure conditions conducive to CLE are not present.

Also as stated in PFM 4, a settlement analysis has not yet been conducted but will be completed as part of the design requirements for this project. Any concerns that arise from this analysis about the compressibility of the fill materials and native clays will be addressed during design. The floodwall is being designed as soil-founded, not pile-founded, and is expected to experience at least a share of the settlement experienced by the earthen embankment.

The levee loading duration is expected to be less than 24 hours in total for a top of levee loading event. This is not considered a long enough time to allow for CLE to progress in any significant manner, let alone to failure, regardless of if CLE is even able to initiate due to poor construction practices. For all of these reasons, this potential failure mode was excluded.

A.6. PFM 6: Flap gate stuck open resulting in backflow into leveed area

PFM 6 occurs if debris were to pin open a flap gate during high channel stages and allow uncontrolled backflow into the leveed interior. Channel water surface elevations higher than the landside drainage structure inlet inverts would produce hydraulic head necessary to backflow into the interior area. Flow could back up through curb inlets, the detention pond drop inlet, or daylighted conduits depending on which flap gate gets pinned open.

This failure mode was determined to be a non-risk driver for a number of reasons. Most of the drainage structures are outlets to underground stormwater systems, and thusly, limits the size of any debris that could pin open a flap gate from the inside as it would likely need to pass through a curb inlet or similar structure. Additionally, the pipes have slopes ranging from 1% to 5% slopes and are likely to be self-cleaning which reduces the likelihood of sediment buildup that could prevent flap gate closure. Large sources of debris from the river, such as logs or branches are unlikely to prevent closure of the flap gates because the river would still need to be at a low/moderate stage for flap gates to be partially open and draining interior flow. Which would not have the velocities and energy to sweep debris downstream. During higher stages that do have the capability to sweep trees downstream, the flap gates would already be well submerged and pinned closed due to the water pressure.

It is recommended that the sponsor inspect the flap gates visually multiple times a year during routine inspections and/or after large storms or flood events which exceed the monitoring stage of 14-feet at the Turkey Creek at Johnson Drive StormWatch gage (TU04). These visual inspections should be conducted to ensure the flap gates are free of debris and functioning properly.

It is recommended to proceed with only one passive closure for each drainage structure, including outfalls greater than 36-inches in diameter. Active closures are not viable alternatives given the flashy nature of the system and the lack of flood forecasting capabilities for the watershed. There also runs a risk of closing a gate well and impounding water on the interior longer than it would have otherwise remained under full passive conditions. The risk to life safety from backflow through an open pipe is zero, and the economic damages related to backflow are minimal as backflow would only lead to small incremental differences in the interior ponding depths in isolated areas. Two examples are shown below. The first is drainage structure ST-15a flap gate on the left bank being stuck fully open for the duration of the top of levee loading scenario. As shown in Figure 87 depths of less than 2 feet make up the majority of inundation on the interior due to backflow, with only 2 structures seeing minor water. The second is the ST-20a flap gate being fully open which allows backflow into downtown Merriam. Backflow into the interior is mostly contained to the streets or parking lots, but some buildings do see inundation of 2 feet or less. The only locations where depths exceed 2 feet are ditches or along the streets as seen in Figure 88.

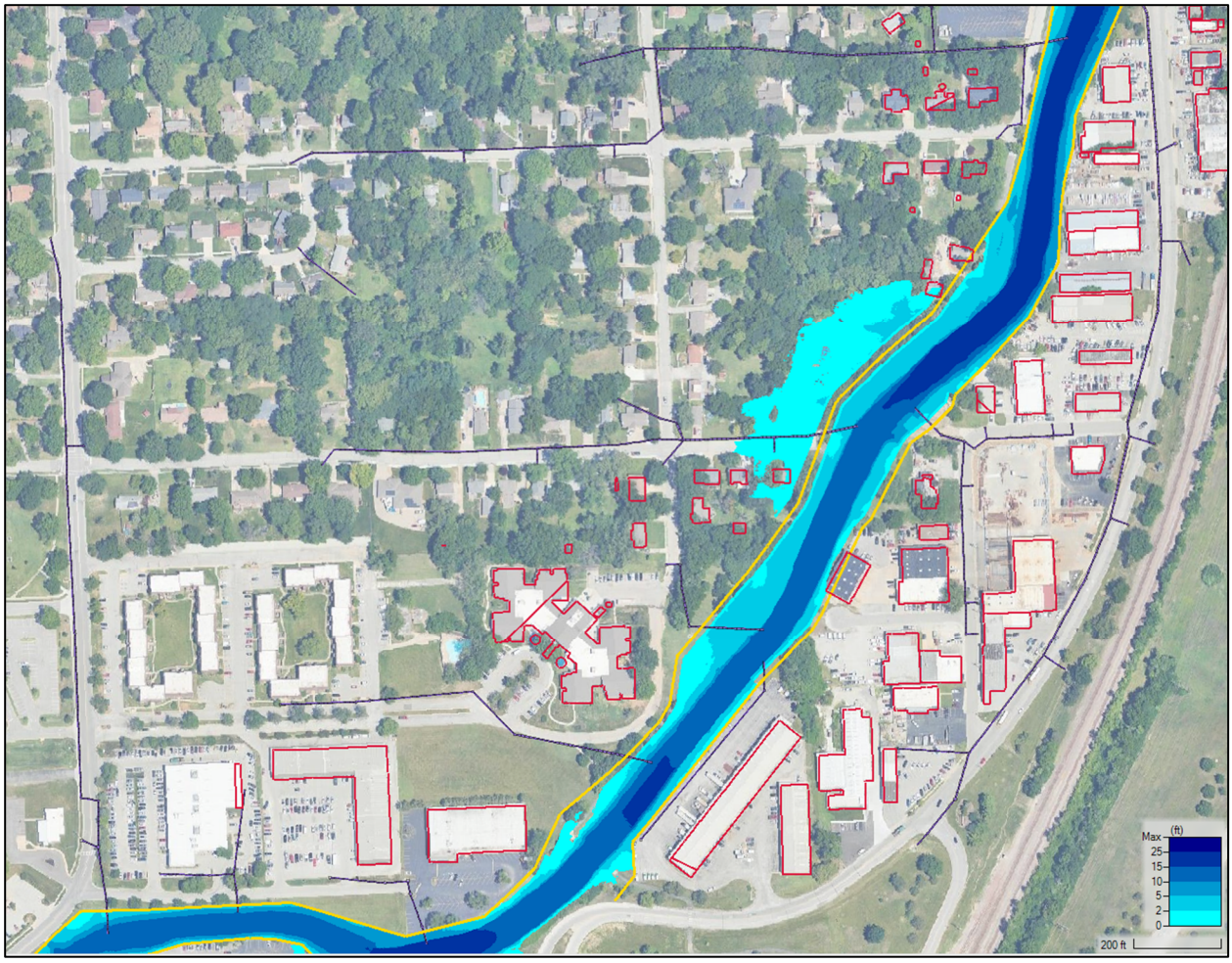


Figure 87. Interior flooding with drainage structure ST-15a flap gate stuck fully open.

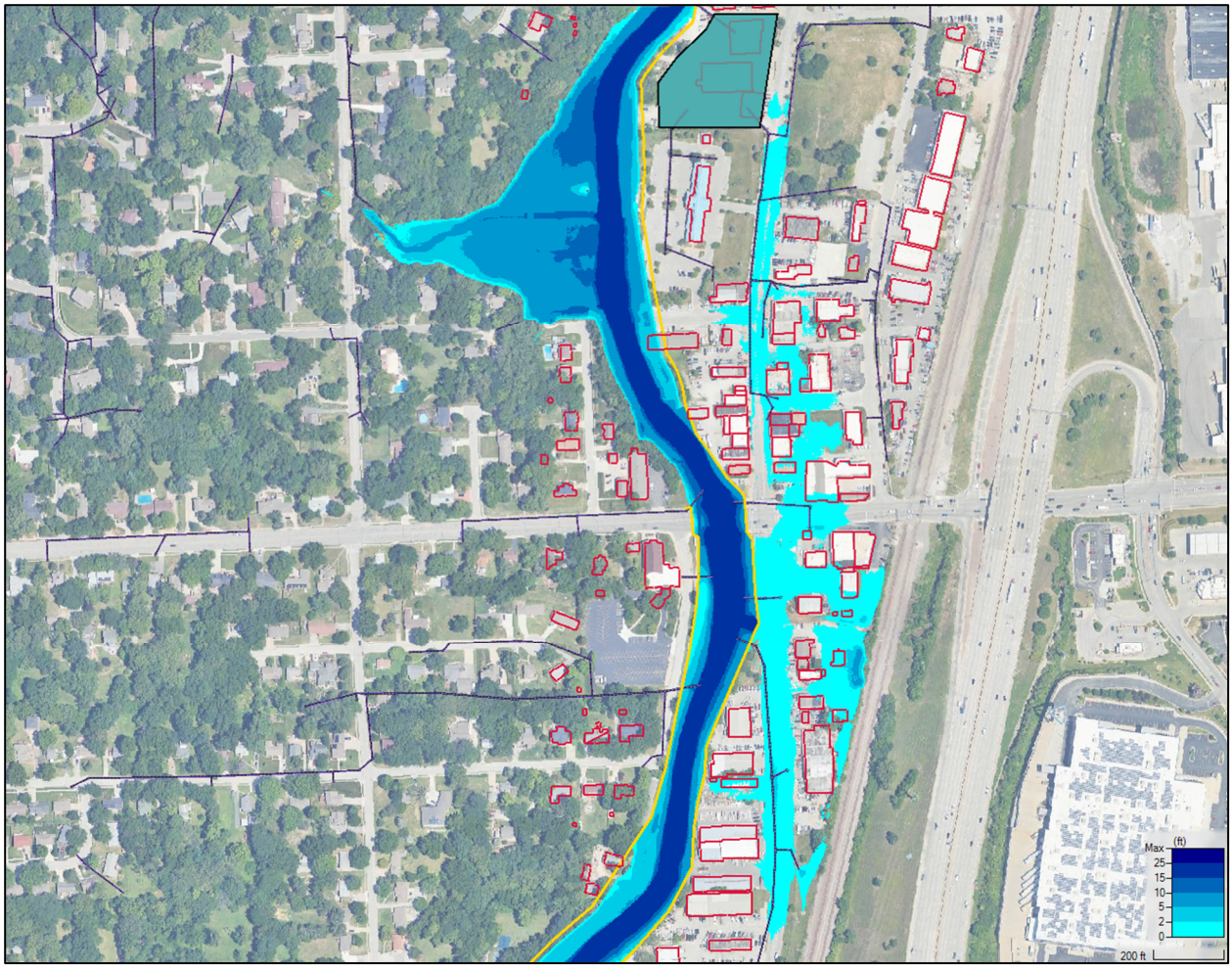


Figure 88. Interior flooding with drainage structure ST-20a flap gate stuck fully open.

A.7. PFM 8: Global instability of the I-wall

This potential failure mode is dependent on the instability of the I-wall. This PFM does not include effects of the destabilization of the limestone wall which is covered in PFM 20.

Requirements for global stability of floodwalls are defined in EM 1110-2-2502 and will be adhered to for the floodwall design. Stability analyses with relevant parameters for end of construction, steady state, and rapid drawdown will be finalized for floodwall following the 35% design milestone. For this reason, this potential failure mode was excluded.

A.8. PFM 9: Global instability of the T-wall

This potential failure mode is dependent on the instability of the T-wall. This PFM does not include effects of the destabilization of the limestone wall which is covered in PFM 21.

Requirements for global stability of floodwalls are defined in EM 1110-2-2502 and will be adhered to for the floodwall design. Stability analyses with relevant parameters for end of construction, steady state, and rapid drawdown will be finalized for floodwall following the 35% design milestone. While more

expensive than I-wall, there does not appear to be physical limitations that would keep T-wall from meeting the design requirements of EM 1110-2-2502. For these reasons, this potential failure mode was excluded.

A.9. PFM 10: Local instability of the I-wall

This potential failure mode is regarding the potential of breaching due to sliding, exceeding the bearing capacity or overturning of the I-wall. The three failures are individual PFMs and may or may not happen at the same time. For sliding, the wall may fail due to excessive horizontal deformations at the very bottom of the sheet pile due to lateral earth pressure and/or hydrostatic pressure, especially during a flood. For bearing capacity, the wall may fail when the pressures that can be permitted against the foundation soils exceed the allowable bearing capacity. The bearing capacity will depend on the loading conditions, compressibility of the soil and geometric considerations. Lastly, overturning of the wall may happen. However, the wall will always fail first due to sliding, exceeding bearing capacity or structural damage before the structure overturns.

The I-walls will be founded on mainly clayey fill or clayey natural soil down to shallow bedrock with no consistent sand layer. Assessment of these potential failure modes must follow the equations and criteria presented in the Engineer Manual 1110-2-2100 Stability Analysis of Concrete Structures (USACE, 2005) and Engineer Manual 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls (USACE, 2022). The team determined that this PFM is not a risk driver if the follows the appropriate design criteria.

A.10. PFM 11: Local instability of the T-wall

This potential failure mode is regarding the potential of breaching due to sliding, exceeding the bearing capacity or overturning of the T-wall. These three failures are individual PFMs and may or may not happen at the same time. For sliding, the wall may fail due to excessive horizontal deformations. The potential for a sliding failure can be assessed by comparing the resisting shear forces against the acting applied shear forces on the wall. Friction of the base and sides of the T-wall provide resistance against sliding. For bearing capacity, the wall may fail when the pressures that can be permitted against the foundation soils exceed the allowable bearing capacity. The bearing capacity will depend on the loading conditions, compressibility of the soil and geometric considerations. Lastly, overturning of the wall may happen. However, the wall will always fail first due to sliding, exceeding bearing capacity or structural damage before the structure overturns.

T-walls will be used in locations where there is no sufficient space for levees. I-walls may also be considered if design deviation is approved. The T-walls will be founded on mainly clayey fill or clayey natural soil down to shallow bedrock with no consistent sand layer. Assessment of these potential failure modes must follow the equations and criteria presented in the Engineer Manual 1110-2-2100 Stability Analysis of Concrete Structures (USACE, 2005) and Engineer Manual 1110-2-2502 Floodwalls and Other Hydraulic Retaining Walls (USACE, 2022). The T-wall sizes were calculated using CTWALL and geometry of the wall was adjusted to reduce overturning and sliding and to decrease the bearing pressure exerted on the soil. The team determined that this PFM is not a risk driver.

A.11. PFM 12: Internal migration into the storm sewer

The focus of this PFM is on the many individual/private storm sewer outlets into the creek. All corrugated pipes will be replaced with concrete pipes during construction and will be recommended to be video inspected every 5 years. Modern construction practices and the short duration of high-water events would

not likely have sufficient duration to initiate and progress a flaw to failure. The team determined this PFM is not a risk driver and was excluded.

A.12.PFM 13: Internal migration into sanitary sewer

The team determined that this PFM is a risk driver, see Chapter 7.

A.13.PFM 14: Internal erosion out of sanitary sewer

This PFM has similar background to PFM 13 but was excluded due to the low likelihood of failure without intervention by means of impacts to water quality as a treatment plant is downstream of the creek. Additionally, the pipe is inspected every 10 years with the results of the last one shown to be in good condition. As such, this PFM is not a risk driver and was excluded.

A.14.PFM 15: CLE along a Pipe

PFM 15 requires a flaw along the length of a pipe crossing under the levee. This deficiency leads to CLE of the material along the pipe, an unfiltered exit on the landside of the levee, hydraulic shear stresses exceed the soil strengths and CLE progresses along the conduit. Material is lost through the unfiltered exit, lowering the level of protection and leading to breach of the levee.

This failure mode was determined to be a non-risk driver based on the construction methods and details that will be required in the construction documents. The construction documents will meet the requirements of and follow the guidance in EM 1110-2-2902. As stated in the EM, 2.3.4.2 PFM-1 Internal Erosion along a Pipe, CLSM will be used along the pipe with a landside filter on the landside end. The CLSM eliminate the common issue with compaction in the haunches and significantly reduce the likelihood of a continues flaw. The landside filter will allow for a filtered exit in the event the CLSM has shrinkage cracks that do allow water to migrate along the pipe. Due to all these considerations, the team determined that PFM is not a risk driver.

A.15.PFM 16: Internal erosion (CLE or BEP) through the levee foundation

As the water surface elevation rises during a flood event, seepage occurs through a continuous (waterside to landside) zone of uniform sand beneath the embankment levee (backward erosion piping or BEP) or through a defect or cracks in the foundation material (concentrated leak erosion or CLE). Levees will be used for risk reduction in place of a floodwall where there is sufficient room for the landside and riverside slopes. For BEP, an unfiltered exit must exist on the landside area. Sufficient seepage forces to detach soil particles in the uniform sand layer at the seepage exit, initiating backward erosion piping. The overlying materials support a continuous and stable roof above the developing pipe. Sufficient horizontal gradients exist for progression of a pipe toward the creek. If detection and intervention are unsuccessful, BEP continues until the pipe connects the creek to the protected area. Gross enlargement of the pipe occurs leading to instability, collapse of the embankment and uncontrolled release of water into the leveed area with associated consequences. For CLE, erosion starts at some defect or crack through the foundation through voids or low density zones. These defects could also be at the foundation/bedrock interface, due to roots of the existing trees, or animal burrows. Erosion of the foundation material could lead to large flows coming from the creek. Enlargement of the seepage path can lead to breach of the structure.

For BEP, a continuous layer of poorly-graded sand needs to exist. The soil profile shows that the subsurface where the levees will be constructed is mainly clayey fill or clayey natural soil down to shallow bedrock with no consistent sand layer. A total of 19 borings have been conducted in the project area, 2 of them were part of the feasibility phase in 2002 and the other 17 in 2023. Only AD-201A shows a layer with poorly graded sand that has a thickness of about 1 foot. The site is generally comprised of undocumented fills underlain by alluvial lean clays. Bedrock was classified as weathered to competent shale and occasional limestone, which ranged from soft to moderately hard. Figure 89 and Figure 90 show the location of a boring and its log, respectively, where a levee will be used for flood control. This is just a small representation of the rest of soil conditions in the project area. The boring log shows mostly clays on top bedrock. The alluvial soils also include silty lean clays, gravelly lean clays, and occasional clays with sand. Seepage analyses were conducted assuming some permeability of the gravel or through cracks in the bedrock. The plan is to excavate a 6-feet-deep inspection trench (Figure 91) below the base of the levee and add a cutoff wall if a continuous pervious zone of sand is found. No seepage berms or relief wells have been recommended for this project. Due to the lack of presence of a continuous sand layer and the construction of a cutoff if one is found, the team has determined that BEP is not likely to occur.



Figure 89. Location of boring ADU-208 where a levee will be used.

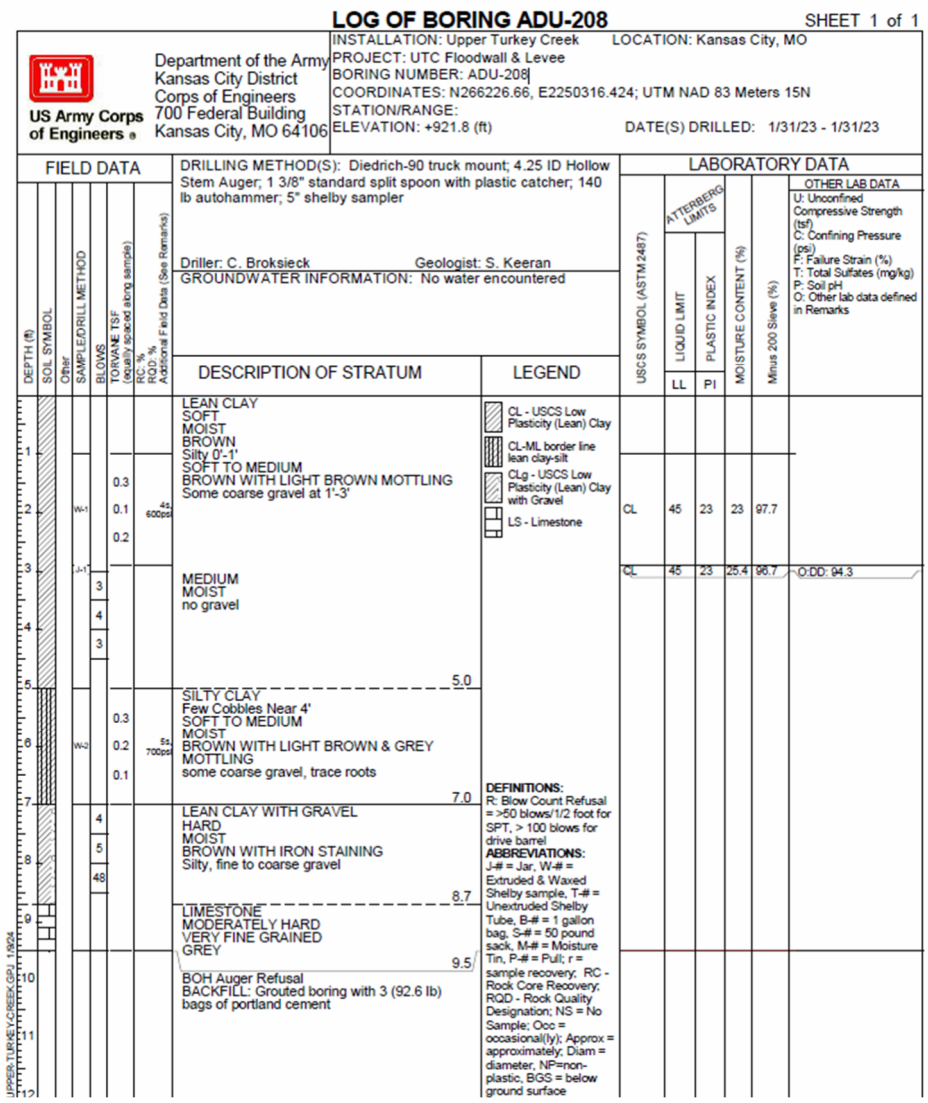


Figure 90. Boring log of ADU-208.

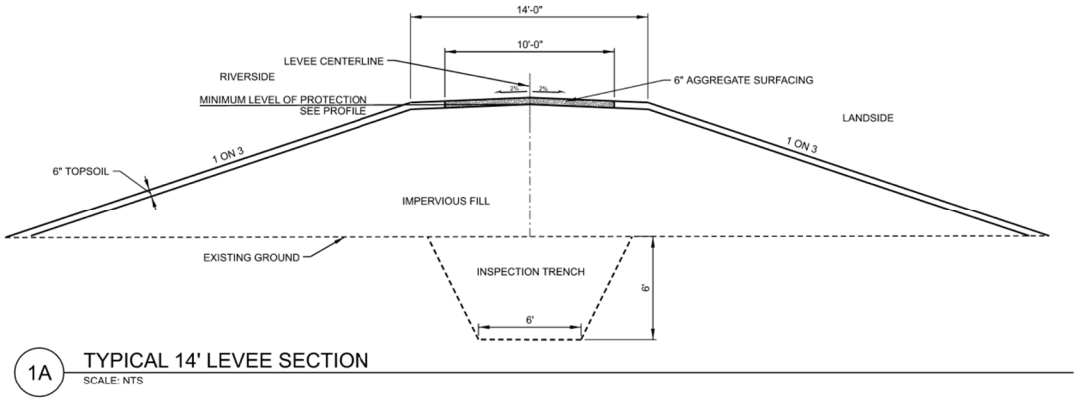


Figure 91. Typical levee section with inspection trench.

For CLE, a void or crack could exist where erosion could start. The fines could maintain the sidewalls of an eroding pipe/crack. There are burrowing animals in the area and the risk assessment team were able to see a groundhog or woodchuck during the site visit on the limestone wall. Extensive burrowing would likely be observed as this area is used daily by the people who use the park or that walk around the area. There are also some trees which roots could provide preferential paths for seepage. While there is a possibility of flaws or defect in the foundation, the short duration would make it unlikely for CLE to progress or continue anywhere between the levee base and the shallow bedrock. Based on these considerations, both CLE and BEP have been excluded as driving the risk in this project.

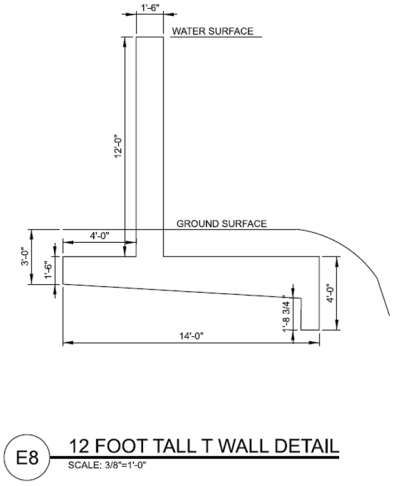
A.16.PFM 17: Internal erosion (CLE or BEP) through I-wall foundation

This PFM is similar to PFM 16, in which internal erosion, whether it is CLE or BEP, occurs beneath within the foundation underneath the flood control structure. Floodwalls will be used for flood risk reduction where there are no sufficient room for a levee. I-walls will be installed in lieu of T-walls wherever possible as long as they meet criteria and have sufficient embedment depth. Sufficient seepage forces detach soil particles and the overlying materials support a continuous and stable roof above the developing pipe/crack. Sufficient horizontal gradients exist for progression toward from the water side to the land side. If detection and intervention are unsuccessful, erosion continues leading to instability and collapse of the I-Wall. Breach of the floodwall causes uncontrolled release of water into the leveed area with associated consequences.

As mentioned in PFM 16, the soil profile in this area is mainly clayey fill or clayey natural soil down to shallow bedrock with no consistent sand layer. BEP would not be possible unless a sand layer is found when excavating. It is possible that CLE could initiate due to voids, joints, cracks, tree roots or animal burrows. I-walls will be embedded into the soil above bedrock if there is sufficient depth that provides resistance against rotational stability. Embedment depth varies in the current design from 12 ft to 20 ft and it could cut off seepage. The short duration of floods would decrease the probability of progression and breach. The team has determined that internal erosion beneath the I-wall is not a risk driver.

A.17.PFM 18: Internal erosion (CLE or BEP) through T-wall foundation

This PFM is similar to PFM 16, in which internal erosion, whether it is CLE or BEP, occurs beneath within the foundation underneath the flood control structure. Sufficient seepage forces detach soil particles and the overlying materials support a continuous and stable roof above the developing pipe/crack. Sufficient horizontal gradients exist for progression toward from the water side to the land side. If detection and intervention are unsuccessful, erosion continues leading to instability and collapse of the T-Wall. Breach of the floodwall causes uncontrolled release of water into the leveed area with associated consequences.



As mentioned in PFM 16, the soil profile in this area is mainly clayey fill or clayey natural soil down to shallow bedrock with no consistent sand layer. BEP would not be possible unless a sand layer is found when excavating. It is possible that CLE could initiate due to voids, joints, cracks, tree roots or animal burrows. However, the short duration of floods would not lead to progression and breach. As part of the design, there was a plan of adding a sheet pile under the T-wall to meet stability criteria that would have also provided a cutoff for seepage. The risk assessment team concluded that adding the sheet pile would not have made the T-wall more stable and that seepage cutoffs were not necessary for the T-wall in the project location due to the soils in the area. The team made the recommendation to remove the sheet pile from the design. The team has determined that internal erosion beneath the T-wall is not a risk driver.

A.18.PFM 19: Instability of limestone wall leads to levee instability

This potential failure mode is dependent on the instability of the limestone wall destabilizing a segment of the earthen embankment. The limestone wall, found lining both sides of the channel, is a gravity block wall built between the 1960s and 1980s. The blocks are approximately 6 feet long, 2 feet deep, and 14 to 18 inches tall and are stacked with approximately 2 inches of offset from the block below (sloping away

from the channel). The wall is maintained by the Merriam Drainage District who estimates 10 percent of the wall has been replaced since its original construction.

The earthen embankment segments of the levee are being designed with a setback from the channel as far as feasibly possible to eliminate instability concerns with the levee that may arise if the block wall collapses. In areas where a sizeable setback is not achievable, the levee alignment will be such that a projected line from the riverside levee toe, with a 1V:3H slope, will not intersect with the face of the block wall. The slope stability analyses will be modeled without the presence of the limestone block wall and will instead show the 1V:3H projected line from the base of the block wall to the ground surface riverward of the levee. The conservatism in choosing to not model the limestone block wall should eliminate concern that a collapse of the wall could lead to instability of the earthen embankment. For this reason, this potential failure mode was excluded.

A.19.PFM 20: Instability of limestone wall leads to I-wall instability

This potential failure mode is dependent on the instability of the limestone wall destabilizing a segment of the I-wall. The limestone wall, found lining both sides of the channel, is a gravity block wall built between the 1960s and 1980s. The blocks are approximately 6 feet long, 2 feet deep, and 14 to 18 inches tall and are stacked with approximately 2 inches of offset from the block below (sloping away from the channel). The wall is maintained by the Merriam Drainage District who estimates 10 percent of the wall has been replaced since its original construction.

The I-wall segments of the levee will be designed with a minimum setback from the channel such that a projected line from the riverside edge of the base of the floodwall, with a 1V:2H slope, will not intersect with the face of the block wall. In the stability modeling, the analyses will be developed without the presence of the limestone block wall and will instead show the projected line from the base of the block wall to the ground surface. The conservatism in choosing not to model the limestone block wall should eliminate concern that a collapse of the wall could lead to instability of the I-wall. For this reason, this potential failure mode was excluded.

A.20.PFM 21: Instability of limestone wall leads to T-wall instability

This potential failure mode is dependent on the instability of the limestone wall destabilizing a segment of the T-wall. The limestone wall, found lining both sides of the channel, is a gravity block wall built between the 1960s and 1980s. The blocks are approximately 6 feet long, 2 feet deep, and 14 to 18 inches tall and are stacked with approximately 2 inches of offset from the block below (sloping away from the channel). The wall is maintained by the Merriam Drainage District who estimates 10 percent of the wall has been replaced since its original construction.

The T-wall segments of the levee are being designed with a minimum setback from the channel such that a projected line from the riverside edge of the floodwall's footing, with a 1V:2H slope, will not intersect with the face of the block wall. In the stability modeling, the analyses will be developed without the presence of the limestone block wall and will instead show the projected line from the base of the block wall to the ground surface. The conservatism in choosing not to model the limestone block wall should eliminate concern that a collapse of the wall could lead to instability of the T-wall. For this reason, this potential failure mode was excluded.

A.21.PFM 22: Seismic instability of levee or wall

The project area is in a region of low seismic activity but has some potential sources at over 100km from the project site. Due to the shallow nature of the overburden above bedrock and the likelihood of the floodwalls being founded on piles resting on bedrock, the likelihood of an event with a great enough magnitude to cause disturbance, the lack of liquefiable sands, and the low probability of a high water event and a co-seismic event, this potential failure mode was excluded.

A.22.PFM 23: Riverine erosion of levee toe/face leads to breach

The slope and conveyance of the creek during flood flows is expected to result in high channel velocities of greater than 16 ft/s and still relatively high velocities in the overbank of greater than 9 ft/s. These overbank velocities are sufficient to remove vegetative cover and initiate erosion in all embankment material types. However, the flow-depth and shear stresses are relatively low and are unlikely to cause significant damage, given the short duration of flooding above the banks (less than 2 hours). The current plan calls for riprap armoring between the limestone block wall and the levee which would mitigate erosion in this zone. The face of the levee itself may have some damage from a flood, but progressing to a full breach is unlikely. Figure 92 shows estimated velocities at one location in relation to the levee.

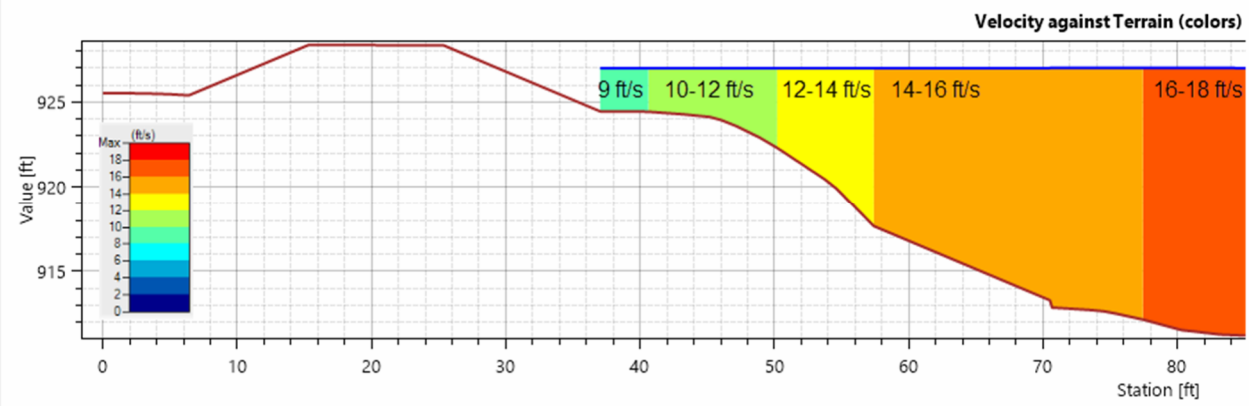


Figure 92. LB Sta 30+30 velocity estimates.

A.23.PFM 24: Riverine erosion of T-wall foundation and breach

Similar to the previous PFM, velocities are estimated to be high in the overbanks, but depths, shear stress, and duration are limited during peak flood conditions. The overbank area between the limestone block wall and the floodwall is planned to be armored with riprap. Erosion in this area is not expected if sufficiently armored. The wall is also designed to be stable if the block wall fails and some amount of creek-side material is eroded. A full failure of the wall from erosion is unlikely to progress during the short duration of peak flood conditions. Figure 93 shows velocity estimates at one location in relation to the floodwall.

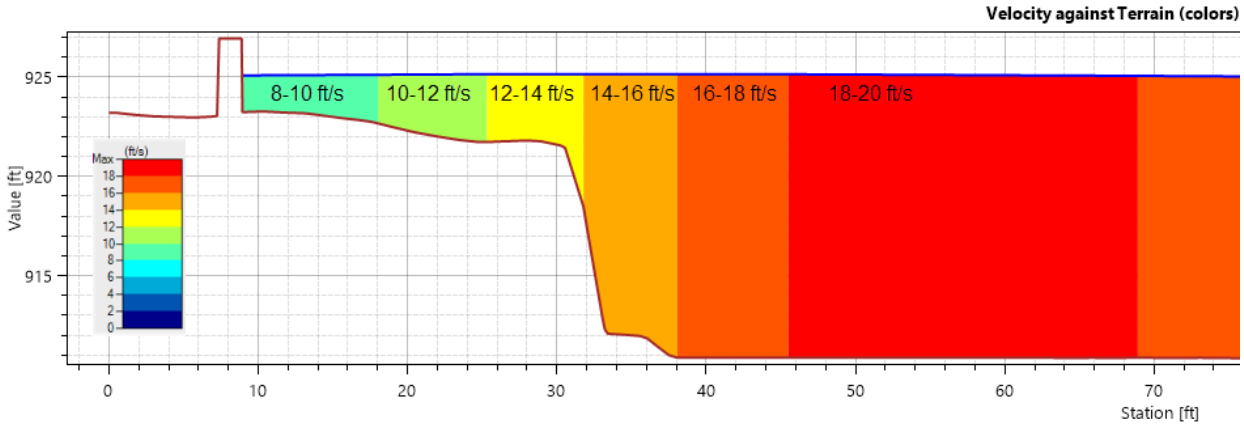


Figure 93. RB Sta 11+20 velocity estimates.

A.24.PFM 25: Riverine erosion of I-wall foundation and breach

Similar to the previous PFM, erosion of the foundation is planned to be mitigated through riprap armoring that withstands the estimated velocities. It is unlikely that the limestone block wall and a significant amount of the foundation are eroded during the short duration of peak flooding.

A.25.PFM 26: Failure of Johnson Drive Bridge Due to Uplift

Replacement of the Johnson Drive bridge is planned as part of the project. The bridge will be designed to AASHTO and city requirements and for USACE stability requirements. The design will be required to meet the stability requirements of EM 1110-2-2100 for the top of protection loading and the likelihood of failure due to uplift will be negligible and this failure mode is considered a non-risk driver.

A.26.PFM 27: Debris impact failure of a floodwall or bridge acting as a flood risk feature

Debris may be present in the channel and from overland sources before and during a flood event. This could be in the form of small vegetation, large woody debris, and anthropogenic sources. While the team expected debris to be present and to get hung up on features that protrude into the flow (e.g., pedestrian bridges), it is estimated that debris is unlikely to cause significant forces on levees or walls to a point of leading to breach. Floodwalls are designed with some debris loading forces incorporated and levees are largely not affected by debris due to the large size, sloping surface, and lower velocities away from the center of the channel. Debris is more likely to be carried in the center of the channel where velocities are highest and are unlikely to move away to lower velocity zones along the FRM features with any significant force associated.

A.27.PFM 28: Pedestrian bridge fails and impacts wall/bridge/levee resulting in breach

The pedestrian bridges are expected to be submerged and/or overtopped during large flood events. The high channel velocities of greater than 18 ft/s and any sources of debris are expected to put large forces on the bridges to a point that could lead to failure. If a bridge fails and is removed from its foundation, the bridge structure could be carried downstream and impact one of the project features, resulting in a breach. These bridges were not designed to support flood and debris flows associated with this FRM project and the team estimated that they may not withstand a major flood. Figure 94 shows an example of what this type of loading may look like for a bridge in Minot, ND during the 2011 flood event.

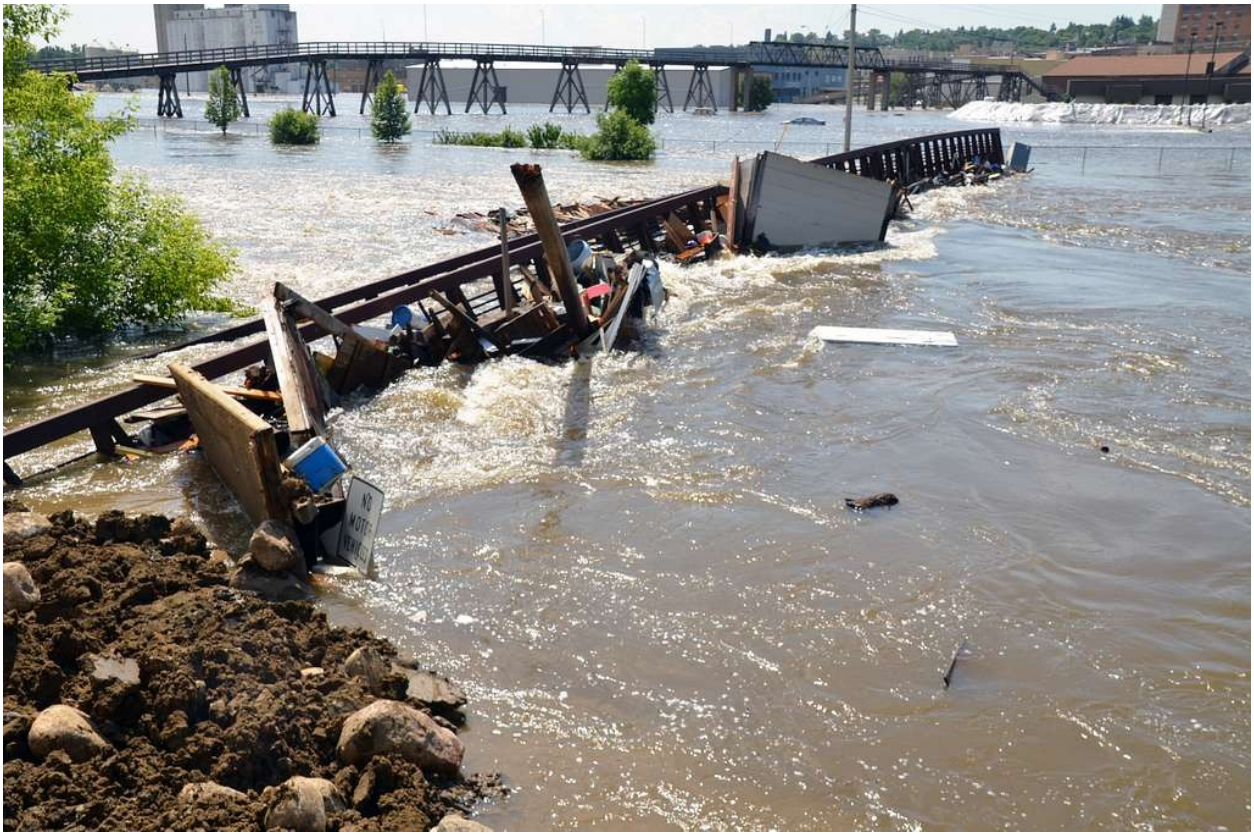


Figure 94. Photo of a pedestrian bridge in Minot, North Dakota during a flood in 2011 (https://nara.getarchive.net/media/a-pedestrian-bridge-that-fell-into-the-souris-river-c1e0cb?zoom=true)

In the case of the Minot flood, the bridge was expected to fail from the debris. Emergency management teams considered and tried to remove the bridge in anticipation of a failure but were unsuccessful. Ultimately, the bridge withstood the forces and did not fail. While the bridges at this project may not withstand similar forces, the team estimated that a failure of the bridge is unlikely to be redirected into a significant impact force against a levee or floodwall. It is likely that the bridge would be carried in the highest velocity zone at the center of the channel, if at all. A bridge failure and associated debris would likely be more of a concern of blocking downstream bridge openings, resulting in increased stages.

A.28.PFM 29: Channel erosion undermines limestone wall and fails the levee or wall

Channel erosion is a natural process for the creek that can be observed by the presence of small waterfalls within this reach, where layers of limestone and other rock of varying integrity will eventually experience differential erosion of softer layers leading to the vertical drop in the channel. Deposition of fine to medium grain material is present during low flows but is likely to be mobilized during peak flood flows. Long term flows and moderate to high flows can cause continued erosion of channel bedrock material and undermining of the limestone block walls, which is of greater concern to the levee and floodwall as a failure of the limestone block wall could progress to a point that could compromise the stability of those features. Figure 95 shows an example of depth-averaged velocities from the two-dimensional model and the relation to the limestone block wall and proposed floodwall. The block wall has experienced some undermining erosion in the past which has been responded to through repair projects by the drainage district in replacing blocks and sometimes placing concrete. These are likely related to long term flows and moderate flows. The concern for levee risk is that a single event undermines the block wall and erodes a significant amount of material. It is unlikely, given the short duration of storm events, that enough material could be removed to compromise the levee or floodwall stability. The design accounts for this through setting back the features so that some amount of erosion can be tolerated. If damage does occur to the block wall during a flood, the local sponsor should be inspecting and maintaining the blocks to ensure stability prior to the next flood event.

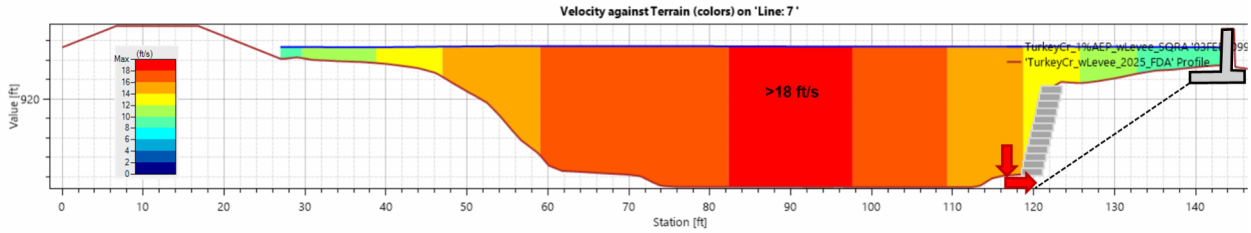


Figure 95. LB Sta 51+00 and RB Sta 10+00

A.29.PFM 30: Structural Failure of I-wall

This PFM is a concrete or sheet pile failure of an I-wall. This requires a major failure and near collapse which drops the top of the protection. As required and as stated in chapter 9 of the DDR, the I-walls will be designed for all applicable load cases in accordance with EM 1110-2-2502 and EM 1110-2-2104. By designing to meet the structural criteria, the likelihood of a structural failure is negligible, and the PFM is considered a non-risk driver.

A.30.PFM 31: Structural Failure of T-wall

The structural failure modes of a T-wall are similar to an I-wall and will be mitigated by following appropriate criteria. Refer to PFM 30.

A.31.PFM 32: Vehicle impact failure of I-wall

The likelihood of this failure mode to coincide with a high-water event is low. Structures are designed to withstand forces from barges and will be designed to consider vehicle impacts. This PFM is considered a non-risk driver since impacts will be a consideration of the design.



A.32.PFM 33: Tree next to the floodwalls/levees topples and leads to instability

This PFM is regarding a tree toppling and damaging any of the levees or floodwalls. If there is significant damage to the levee or floodwall and a high water event happens before any repairs, water could erode the exposed area or the elevation of protection could decrease if the floodwall is pushed. These damages could lead to breach of the levees or floodwalls during a flood event. The USACE policy has been to keep levees “vegetation free” on both slopes, crest and 15 feet away from the landside and waterside toes. The recommended vegetation cover for the levees has always been mowed grass. This is due to the uncertainty of how trees, shrubs and other vegetation could interact with the levees. Also, vegetation makes it harder for inspections and intervention. For floodwalls, policy has been to keep a “vegetation free” zone of 15 feet away from the wall (minimum) on the land side and water side. In addition, there is an 8 feet horizontal clearance from the wall foundation. Lastly, there is a vertical clearance of 8 feet above the top of the floodwalls. There are existing trees where the levees and floodwalls will be constructed both toward the land side and water side. Also, there is vegetation and trees that have grown right on top of the limestone wall as seen in Figure 96.



Figure 96. Vegetation along the Upper Turkey Creek.

The USACE has recently adopted risk-informed design process that could allow vegetation other than grass on levee, mainly on the waterside slope and waterside toe. This process requires evaluation of what type of vegetation will be planted and its associated risks. There have been no requests for leaving existing trees in place where the levees and floodwalls will be at the moment when this risk assessment was conducted. It could be possible that people that live near the park could want to leave some of the trees in place where levees will be constructed. A waiver would be needed allowing for vegetation, which depend on the results of another risk assessment. While a waiver could be requested for levees after conducting a risk assessment, best practices for floodwalls is to retain the minimum vertical and horizontal clearance distances in which there should be no trees or woody vegetation. Only grass or other small herbaceous vegetation could be allowed as long as the roots do not cause damage to the walls. This PFM was determined to be a non-risk driver if existing trees are removed and vegetation is cleared within the vegetation free zones or if they are outside of the required distances.

A.33.PFM 34: Wind-wave erosion of the levee

Wind-waves can develop over bodies of water with appreciable wind fetches, which then can break on the slope of a levee and cause significant erosion. If this erosion occurs for a sufficient duration and location, it could progress through the levee and result in a breach. The narrow, winding creek channel does not allow for long wind fetches to become apparent and would require specific wind directions for any fetches longer than the floodplain width of 100-500 feet. Also, high-velocity, turbulent flows may disrupt wave growth in the transverse direction to the flow. While a storm that generates peak flood flows may be associated with stronger winds, it is unlikely that peak winds coincide with peak water levels that would erode the narrowest part of the levee. Given that the duration of loading above the levee toe is expected to be less than a few hours, it is unlikely that the duration of any impact forces from small wind waves would cause enough erosion to progress to failure. Given the very short duration and low probability of coincident, adverse wind conditions during the short peak, it is unlikely that any wind-wave damage would occur to the levee and extremely remote that it would result in a full breach.

A.34.PFM 35: Internal erosion out of pressurized pipe

For this PFM, pressurized pipe ruptures within or under the levee or floodwall. Escaping fluid causes internal erosion sufficiently that the levee crest or top of floodwall is impacted and eventually lowered below the flood water level. Overtopping occurs and leads to breach.

The risk assessment team was informed that there are water and gas pipes in the ground where the levees and floodwalls will be constructed. Size and lengths of these pipes can vary. To cause major issues to the levee that could lead to its breach, a rupture of one of these pipes would have to go unnoticed. Loss of pressure due to rupture is independent from low water or flood events. As these pipes operate constantly and are heavily monitored, the likelihood of any of them experiencing a major rupture that could cause erosion issues to the levee are considered remote. The loss of pressure would be identified by the public agencies and would have to address the issue quickly. Some of these utility pipes are planned to be relocated. Others will be modified to replace their material. The team recommends valves on both sides of the levees for pressurized pipes that go through the levee embankment or its foundation. The team has determined this PFM is not a risk driver.

A.35.PFM 36: Bridge construction leads to increased water levels for the existing conditions

If care is not taken during construction of the bridge replacements, a condition could be created which increase the existing condition flood risk above current levels. For example, a full-height cofferdam that blocks half of the channel could result in increased water level impacts upstream of the bridges and, for certain flood events, could result in life safety or economic consequences that would not have been experienced with the existing bridge having full conveyance. The current plan is that bridge replacements will occur before the construction of the rest of the project including levees and floodwalls, so the flood risk management features will not be in place to mitigate some of the effects of bridge construction during a major flood. The primary factor that reduces the risk of this PFM is that normal water levels in the channel are typically only a few feet in depth, so the dewatering features associated with bridge construction are likely to have minimal impact on conveyance. While a flood could occur at any time and impact the dewatering and construction efforts, it is likely that the impact to upstream water levels is minimal.

A.36.PFM 37: Flap Gate stuck closed leads to excessive ponding

PFM 37 occurs if debris or sediment settled on the outside of a flap gate which does not allow drainage of the flap gate and leads to excessive ponding during an extreme storm event on the interior. This failure mode is a non-risk driver because it does not lead to uncontrolled flow into the levee system, only prevents drainage. Additionally, the sponsor should be ensuring there is clearance for the flap gates to open during flood events as part of routine operations and maintenance. Having two forms of closure does not prevent riverside flap gates from being pinned closed causing ponding of the interior and so this PFM does not factor into the single closure recommendation.

As also stated in PFM 6, it is recommended that during routine visual inspections of the levee system, that the sponsor clear any sediment or debris that would prevent opening of the flap gates.

A.37. PFM 38: Global Instability of Shawnee Mission Parkway roadway embankment east of Turkey Creek.

PFM 38 occurs if loading of the upstream side of the Shawnee Mission Parkway embankment is sufficient in both loading height and duration to reduce shear strengths of the embankment soils, causing a slide and lowering of the embankment height which would allow an uncontrolled release of flow into the leveed area.

The portion of Shawnee Mission Parkway to the east of Turkey Creek acts as a non-project feature in the flood protect system. The roadway embankment prevents flow from entering the leveed area after the levee and floodwall has already tied into the downstream side of the roadway embankment as shown below in Figure 97. This PFM was excluded due to the significant freeboard of 11 feet for the top of levee loading scenario between the upstream depths and roadway crest. Additionally, the roadway embankment is over 200 feet wide which would require significantly longer and higher loadings to saturate soils and reduce sheer strength for any slide to occur.

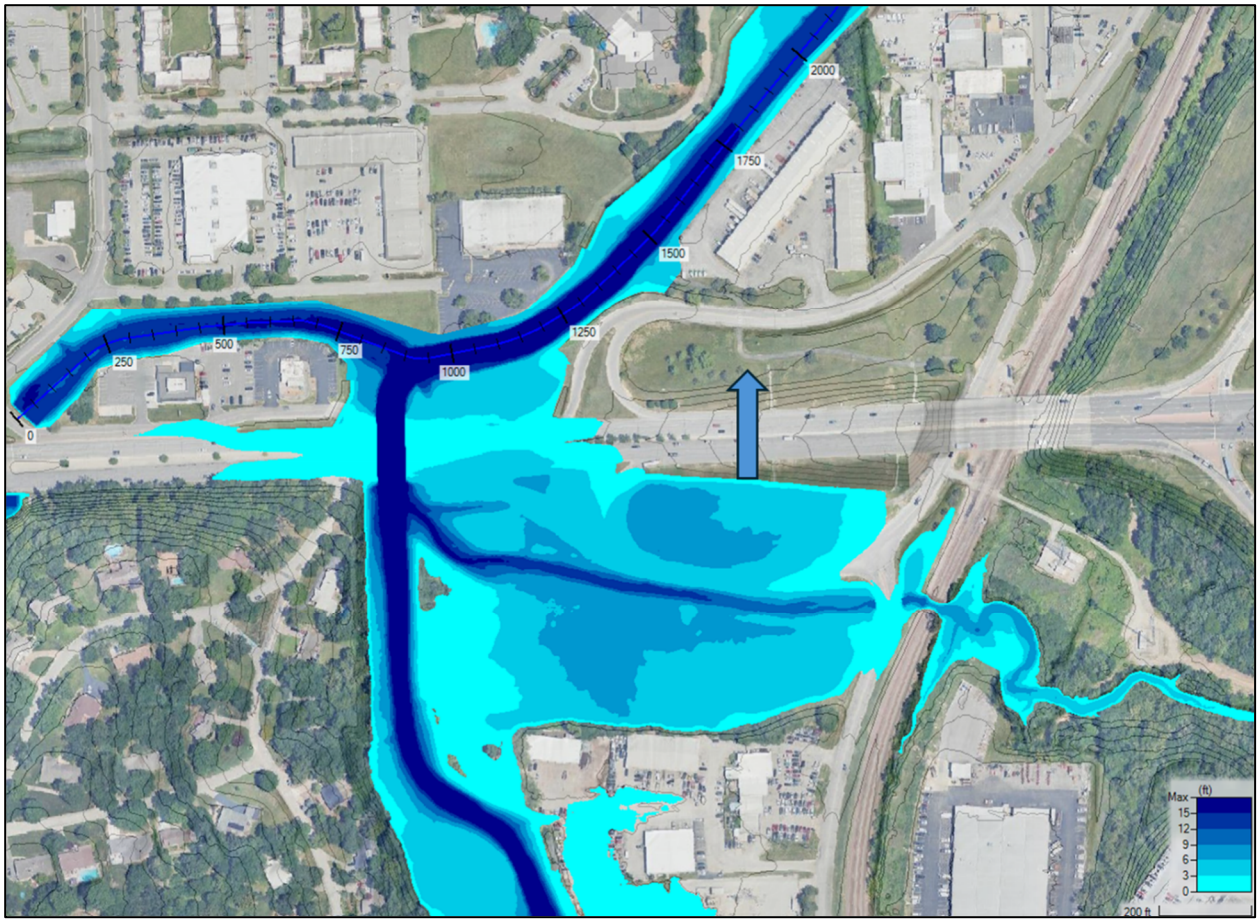


Figure 97. Non-project feature flood protection of Shawnee Mission Parkway roadway embankment.

Appendix B: Pertinent Plates and Drawings

- See UTC 35% PLAN SET.pdf dated 07 JAN 2025

Appendix C: Calculations

- SQRA Risk Calculations
- Levee Transient Seepage Sensitivity Analysis

Appendix D: Review Documentation

This appendix must be last in sequence.

Provide the signed Risk Assessment Report Concurrence and Approval sheet, DQC review comments (e.g., comment summary report from DrChecks or Excel worksheet), advanced H&H ATR comments, and ATR comments (comment summary report from DrChecks), in that order.

CENWK-ED 01 February 2026

MEMORANDUM THRU Commander, U.S. Army Corps of Engineer, Northwest Division, (CENWD-RBE/Ms. Rebecca Vandine), 631 Saint Anne St, Suite 100, Rapid City, SD 57701

Chair of SOG (CEIWR-RMC/Mr. David Serafini), 132 E Street, Suite 130, Davis, CA 95616

HQUSACE (CECW-EC/Mr. Rich Varuso), 7400 Leake Ave, New Orleans, LA 70118

FOR HQSACE (CECW-EC/Mr. Travis Tutka), 441 G Street NW, Washington, D.C. 20314

SUBJECT: Design Guidance Deviation for Secondary Closure for Storm Drainage Pipes – Upper Turkey Creek, Merriam, Kansas

1. PURPOSE: This memorandum requests approval for a design guidance deviation requiring secondary closures for storm drainage pipes 36 inches and larger within the Upper Turkey Creek project limits. The closure type requirements for storm drainage pipes set forth in EM 1110-2-2902, Chapter 9, 9.4.1 Gates, states that drainage pipes 36-inches and larger for fast rising floods requires one passive and one active closure. Kansas City District requests a variance to require one passive closure for pipes 36-inches and larger.

2. REFERENCES:

a. EM 1110-2-1413 Engineering and Division, Hydrologic Analysis of Interior Areas, 24 August 2018

b. EM 1110-2-1913 Engineering and Division, Design and Construction of Levees, 30 April 2000

c. EM 1110-2-2902 Engineering and Division, Conduits, Pipes, and Culverts Associated with Dams and Levee Systems, 31 December 2020.

3. BACKGROUND: The Upper Turkey Creek FRM project is located in the City of Merriam, in Johnson County, Kansas. Turkey Creek is a right-bank tributary of the Lower Kansas River located in Johnson and Wyandotte counties, Kansas, between approximate river miles (RMs) 2.7 to 3.7.

There are approximately 17 existing gravity drainage crossings under the proposed levee/floodwall. The portion of the existing drainage pipe crossing the levee or floodwall

limits will be replaced with reinforced concrete pipe (RCP) and any existing pipes smaller than 24-inch will be upsized to 24-inch with this project.

EM 1110-2-2902, Conduits Pipes and Culverts Associated with Dams and Levee Systems lists the closure type requirements for different flood rise conditions. The flood rise condition for our project is considered fast since there is less than 12 hours of prediction time. The Project Delivery Team (PDT) recommends a deviation from Table 9-1 of EM 1110-2-2902 to use one passive closure at the outlet for all pipes including those greater than or equal to 36 inches. This recommendation affects 8 of the 17 drainage crossings.

4. IMPACTS AND BENEFITS: This deviation would present an opportunity to reduce the operation and maintenance impacts for the Sponsor without increasing incremental project risk. This deviation also reduces the risk of the Sponsor closing an active gate and impounding water on the interior longer than it would have otherwise remained under full passive conditions. The risk to life safety from backflow through an open pipe is zero, and the economic damages related to backflow are minimal as backflow would only lead to small incremental differences in the interior ponding depths in isolated areas.

5. RISK ASSESSMENT: The design guidance deviation was evaluated as part of the Semi-Qualitative Risk Assessment (SQRA) and documented in the SQRA Report (2025). There were 37 potential failure modes (PFMs) identified by the team. Below, Table 1 summarizes the list of PFMs with a short description. The risk assessment concluded that the design deviation does not increase the incremental life safety risk and can be adopted without negatively impacting the risk characterization associated with the project. The risk matric for the project including the proposed deviation is shown in Figure 2.

Table 1. Potential failure modes.

Potential Failure Mode	Risk Driver?	Description
PFM 01	Yes	Overtopping breach of the levee
PFM 02	No	Overtopping breach of the floodwall
PFM 03	No	Concentrated leak erosion through the embankment between construction lifts
PFM 04	No	CLE along differential settlement
PFM 05	No	CLE along levee/wall/bridge interfaces
PFM 06	No	Flap gates stuck open resulting in backflow (for pipe >36", assuming one flap gate per pipe)
PFM 07	Yes	Global instability of the levee

Potential Failure Mode	Risk Driver?	Description
PFM 08		Global instability of the I-wall
PFM 09		Global instability of the T-wall
PFM 10	No	Local instability of the I-wall
PFM 11	No	Local instability of the T-wall
PFM 12	No	Internal migration into the storm sewer
PFM 13	Yes	Internal migration into sanitary sewer
PFM 14	No	Internal erosion out of sanitary sewer
PFM 15	No	CLE along a pipe
PFM 16	No	CLE or BEP through the levee foundation
PFM 17	No	CLE or BEP through I-wall foundation
PFM 18	No	CLE or BEP through T-wall foundation
PFM 19	No	Instability of limestone wall leads to levee instability
PFM 20		Instability of limestone wall leads to I-wall instability
PFM 21	No	Instability of limestone wall leads to T-wall instability
PFM 22	No	Seismic instability of levee or wall
PFM 23	No	Riverine erosion of levee toe or levee face
PFM 24	No	Riverine erosion of T-wall foundation
PFM 25	No	Riverine erosion of I-wall foundation
PFM 26	No	Failure of Johnson Bridge due to uplift
PFM 27	No	Debris impact failure of wall/bridge
PFM 28	No	Pedestrian bridge fails and impacts wall/bridge/levee
PFM 29	No	Channel erosion undermines limestone wall and fails levee/wall
PFM 30	No	Structural failure of I-wall
PFM 31	No	Structural failure of T-wall
PMF 32	No	Vehicle impact failure of I-wall
PMF 33	No	Tree next to the floodwalls/levees topples and leads to instability
PMF 34	No	Wind-wave levee erosion
PMF 35	No	Internal erosion out of pressurized pipe
PMF 36	No	Bridge construction leads to increased water levels for existing conditions
PMF 37	No	Flap gate stuck closed leads to excessive ponding

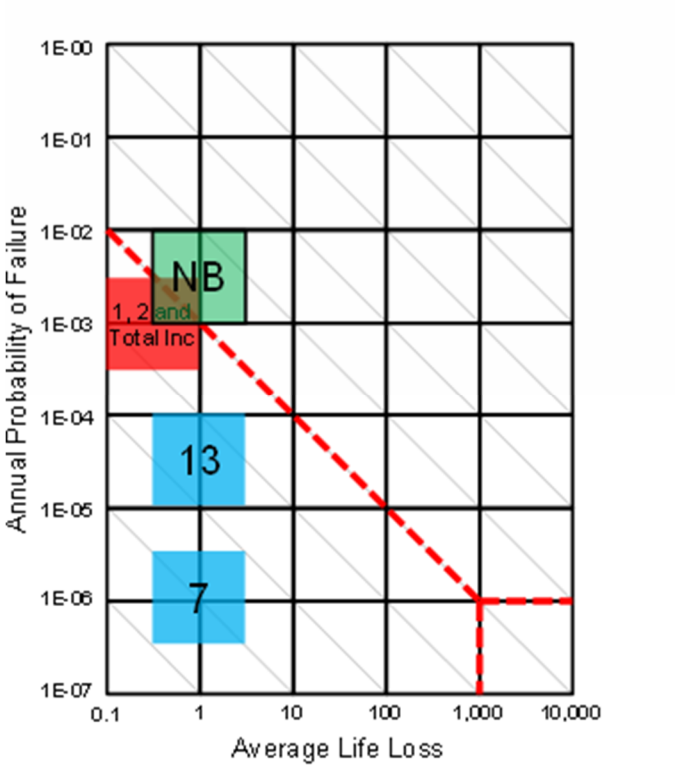


Figure 2. Societal Incremental Life Safety Risk Matrix.

The team identified one potential failure mode (PFM 6) in which one passive closure was directly involved by initiating or progressing failure. The PFM discussed below was evaluated and characterized as a non-risk driving PFM.

a. PFM 6: FLAP GATE STUCK OPEN RESULTING IN BACKFLOW INTO LEVEED AREA: PFM 6 occurs if debris were to pin open a flap gate during high channel stages and allow uncontrolled backflow into the leveed interior. Channel water surface elevations higher than the landside drainage structure inlet inverts would produce hydraulic head necessary to backflow into the interior area. Flow could back up through curb inlets, the detention pond drop inlet, or daylighted conduits depending on which flap gate gets pinned open.

   This failure mode was determined to be a non-risk driver for a number of reasons. Most of the drainage structures are outlets to underground stormwater systems, and thusly, limits the size of any debris that could pin open a flap gate from the inside as it would likely need to pass through a curb inlet or similar structure. Additionally, the pipes have slopes ranging from 0.66% to 5.45% slopes and are likely to be self-cleaning which reduces the likelihood of sediment buildup that could prevent flap gate

closure. Large sources of debris from the river, such as logs or branches are unlikely to prevent closure of the flap gates because the river would still need to be at a low/moderate stage for flap gates to be partially open and draining interior flow which would not have the velocities and energy to sweep debris downstream. During higher stages that do have the capability to sweep trees downstream, the flap gates would already be well submerged and pinned closed due to the water pressure.

It is recommended that the sponsor inspect the flap gates visually multiple times a year during routine inspections and/or after large storms or flood events which exceed the monitoring stage of 14-feet at the Turkey Creek at Johnson Drive StormWatch gage (TU04). These visual inspections should be conducted to ensure the flap gates are free of debris and functioning properly.

It is recommended to proceed with one passive closure for each drainage structure, including outfalls greater than 36-inches in diameter. Active closures are not viable alternatives given the flashy nature of the system and the lack of flood forecasting capabilities for the watershed. There also runs a risk of closing a gate well and impounding water on the interior longer than it would have otherwise remained under full passive conditions. The risk to life safety from backflow through an open pipe is zero, and the economic damages related to backflow are minimal as backflow would only lead to small incremental differences in the interior ponding depths in isolated areas. Two specific drainage structures along the project were analyzed. The first is a drainage structure (St-15) on the left bank with the flap gate fully stuck open for the duration of the top of levee loading. A majority of the inundation on the interior is less than 2 feet with only two structures seeing a minor impact. The second is drainage structure (ST-20) on the right bank with the flap gate being fully open which allows backflow into downtown Merriam. Backflow into the interior is mostly contained to the streets or parking lots, but some buildings do see inundation of 2 feet or less.

6. SUMMARY: Based on the SQRA, the proposed design guidance deviation does not impact the incremental project risk and the project risk will remain below tolerable risk guidelines. The design guidance deviation was presented, and findings were briefed to the RID Group on May 20, 2025. Based on the SQRA findings, Kansas City District recommends approval of the design deviation described in this memorandum.

7. POINT OF CONTACT. Please contact Zack Young, NWK LSPM, at (816) 389-3042 or zackary.j.young@usace.army.mil regarding this matter.

Encl

ERIC D. SHUMATE, P.E.
Chief, Engineering Division
NWK Levee Safety Officer
Kansas City District
U.S. Army Corps of Engineers

Rebecca M. Vandine, P.E.
NWD Levee Safety Officer
Northwest Division
U.S. Army Corps of Engineers

David C. Serafini, P.E.
Chair of SOG
CEIWR-RMC
U.S. Army Corps of Engineers

Rich J. Varuso, P.E.
Levee Safety Program Manager
H.Q, U.S. Army Corps of Engineers

Travis C. Tutka, P.E.
Chief of Dam and Levee Safety Branch
H.Q, U.S. Army Corps of Engineers

This page was intentionally left blank for duplex printing.

APPENDIX J – DESIGN CHARRETTE 2023 & 2024

This page was intentionally left blank for duplex printing.

CENWK-PMC-J 19 November 2024

MEMORANDUM FOR RECORD
SUBJECT: *Upper Turkey Creek Design Charrette Summary*

1. The purpose of this memorandum is to document the activities, discussions and conclusions from the Design Charrette that was performed for this project early in design. The Design Charrette was focused on the downtown reach of the project, between Johnson Drive and where the creek crosses Merriam drive to the north.

2. **2023 Efforts.** During some early meetings with the City of Merriam, it was decided that additional alternatives to the feasibility study recommended plan needed to be considered in order to align with the city's goals for the downtown reach of the project. The Design Charrette initial meeting was held 23-24 August 2023. The agenda and list of participants can be found in the attachments. The objective was to develop concepts that meet the city and USACE objectives and to select a preferred concept. During this meeting the city and USACE presented the concepts developed by the city in recent years before this project started and USACE presented the recommended plan. Then the group developed city and USACE priorities. These priorities are as follows:

- CITY PRIORITIES
    1. Incorporates city vision for downtown – not in FEMA regulated floodplain – developable.
    2. Creek as an amenity / visual and physical access / connectivity
    3. Green space and trails on east side of creek
    4. Optimize project value
        - » Coordinate with Merriam Drive current redevelopment plans (construction planned for 2024)
    5. Preserve Market Place (in floodplain as-is is ok or relocate if needed)
    6. Consider preserving South end of downtown buildings (LOW)
- USACE PRIORITIES
    1. Reduce flood risk for the whole area identified during feasibility study
    2. Minimize or eliminate USACE processes to re-evaluate (time and money)
        - » PACR already likely (cost)
        - » LRR at most (scope)
    3. Work with nonfederal partner – rework the original plan as needed to coordinate with development
    4. Minimize project cost

Other important considerations were to maintain creek access for maintenance and access for the sanitary sewer and possibly other utilities for maintenance.

Then the team split into groups and developed some concepts to align with the city and USACE priorities. Ten concepts were developed and they are included in this report as an attachment. Additionally, each concept was presented and reviewed as a group and the results of those

discussions and conclusions are also included in a table and other notes at the end of the concepts in the attachment. The group concluded to further develop alternatives 1b, 2b, 6, 7a and 7b. The H&H modeling of these 6 concepts that progress thru a sequence in order to see how the model responds with the incremental changes. The alternative that reaches the level needed for levee certification is the city's preferred alternative. The following was the 6 concepts, #3-#8 modeled, refer to the attachments for more details of each alternative.



Six Concepts modeled for the Design Charrette

3. 2024 Efforts. The alternative concepts were modeled in HEC-RAS the fall of 2023. There were some intentional delays as we waited for the Johnson County modeling to be completed, but due to delays in those efforts, we decided to proceed with developing these with the USACE hydrology in late 2023. Upon completion of the H&H modeling, the group reconvened on 5 March 2024 to review the findings and with the objective to make a decision going forward for design.

The results of the eight concepts were not giving the results that met the project priorities. Plan views and cross sections of H&H results for the eight concepts are in the attachment. Benching was somewhat limited in order to ensure sedimentation doesn't become a maintenance issue. Channel benching or channel widening causes flow to slow down and material drops out and deposits in the creek bed. Then eventually vegetation grows and becomes harder to displace by the water in a flood event. For these analyses, benching was limited to an estimated 2-year elevation.

Benching alone, concepts #3-#6, was not sufficient to provide the required level of protection. Concept #7 included benching with a levee along Merriam Drive, resulted in a taller levee than was preferred. Concept #8, which added some additional benching downstream of the Merriam Drive Bridge on the right bank, required shorter levees than #7.

All the alternatives required significant real estate acquisition along the west side of Merriam Drive and did not result in significantly lower or no levees along this reach. As a result, the group concluded to maximize available real estate out of the floodplain and to minimize negative impacts to the benefit-cost ratio, keeping the levee along the feasibility alignment was preferred. Additional considerations for the team were to maximize the levee extents in the downtown reach versus floodwall and to further investigate benching downstream of the Merriam Drive Bridge on the right bank and possibly channel widening north of Johnson Drive to further reduce the height of the levees and floodwalls in the downtown reach.

4. Conclusion. The project will remain on the alignment established in the feasibility study recommended plan since the concepts' impacts to real estate are substantial without much impact to the levee and floodwall heights through the downtown reach of this project. The project will likely have levees and floodwalls taller than the 3-foot objective, however the team will consider

benching downstream of the Merriam Drive Bridge on the right bank and possibly channel widening north of Johnson Drive during design to further reduce the height of the levees and floodwalls in the downtown reach.

Attachments

1. 2023 Agenda
2. 2023 Charrette Attendees
3. 2023 Concepts
4. 2024 Concept Modeling Results

Katrina Marx,
Project Manager,
Civil Works (PMC-J)

2023 AGENDA

CHARRETTE AGENDA Upper Turkey Creek - Merriam, KS		
FOCUS AREA: Johnson Drive North to the Merriam Drive creek crossing		
Charrette Objectives: PRIMARY: The purpose of the charrette is to develop concepts that meet project objectives and select a preferred concept that provides enough information to make decisions to proceed with into the design phase. SECONDARY: (1) Delineate USACE cost shared project from city street and development projects (2) Include conceptual construction phasing AND impacts to local activities (farmers market, car show, etc)		
Time	**Wednesday, August 23** USACE - 601 E 12th St, KCMO Room G41	**Thursday, August 24** USACE - 601 E 12th St, KCMO Room G41
0830	Building entry and Assemble	Building entry and Assemble
0900	**INTRODUCTIONS**	**Review Hybrid Concepts Developed on Day 1 - likes and dislikes - select 1-2 to develop further**
0930	**OVERVIEW PROJECT / CHARRETTE PROCESS** Katrina Marx / Gary Harden	
1000		
1030	**Review City Concepts & USACE Recommended Plan** Bryan / Katrina	**Further Develop Concept A** 1 TEAM
1100		
1130	**OBJECTIVES EXPECTATIONS** USACE/City	
1200	**Lunch Break**	**Lunch Break**
1230		
1300		
1330	**Develop Hybrid Concepts (2-4)** 2 teams	**Further Develop Concept B** 1 TEAM
1400		
1430		
1500		
1530		**Review Concept(s) changes / likes & dislikes**
1600		
1630		**Action Items / Next Steps / Schedule**
1700	Adjourn	Adjourn
EXPECTATIONS: (1) There will be additional hydraulic modeling and possibly other engineering efforts required outside of this 2 day effort to solidify any concept(s) developed. A short report or presentation documenting the charrette work and findings will be developed. A follow on meeting in X weeks (date TBD) will occur to verify that the concept(s) are valid. (2) It may not be possible to implement all ideas under the USACE authorized project. (3) Other technical experts (geotech, structures, cost, etc) are available as needed in an on-call basis.		

2023

CHARRETTE ATTENDEES

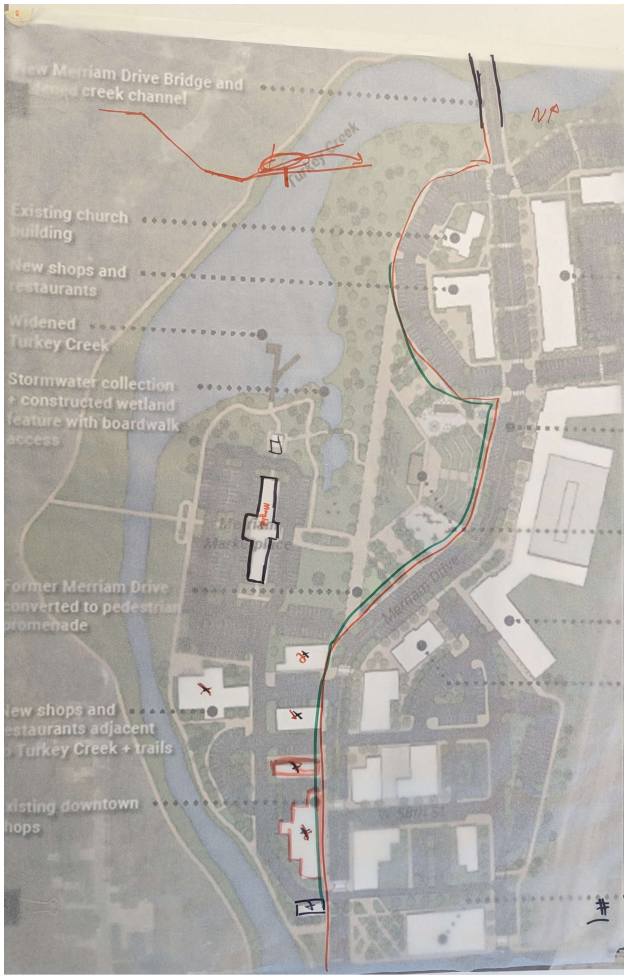
UPPER TURKEY CREEK - DESIGN CHARRETTE (AUG 2023) SIGN IN

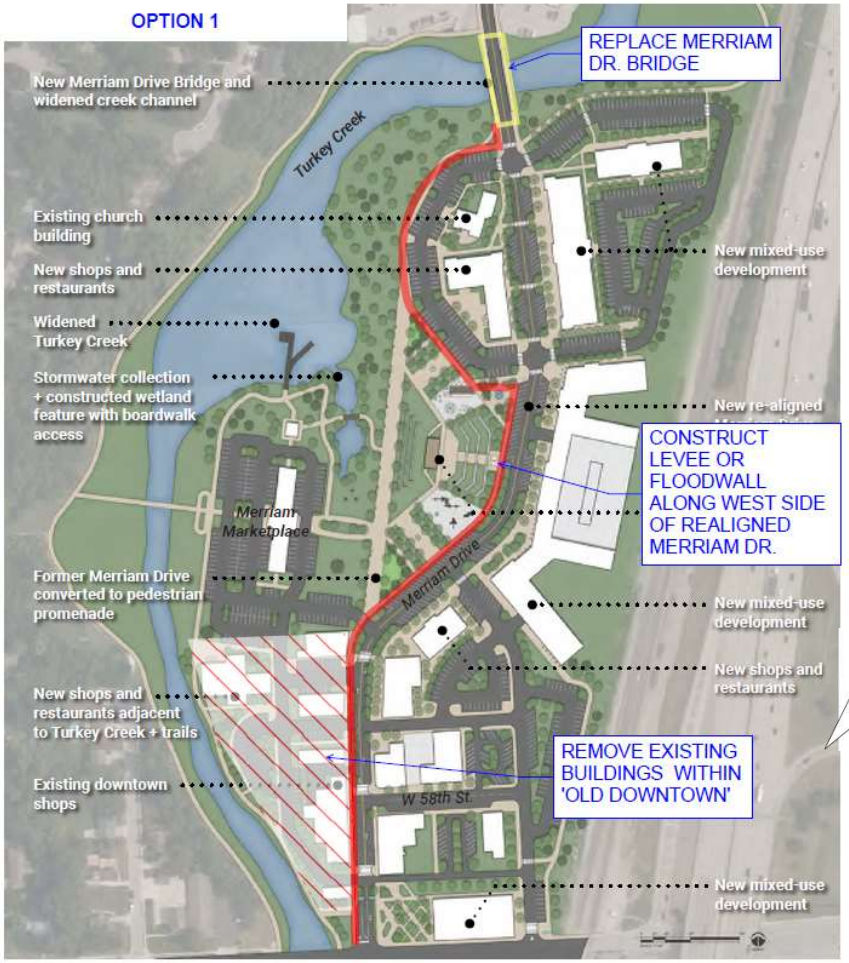
23-Aug	24-Aug	LAST	FIRST	AGENCY	POSITION	EMAIL	PHONE	PHONE 2
CE		Engel	Chris	City of Merriam	City Administrator	cengel@merriam.org	913-322-5511	
X	X	Dyer	Bryan	City of Merriam	Community Development Director	bdyer@merriam.org		
X	X	Gorton	Randall	BHC	Merriam City Engineer	Randall.Gorton@ibhc.com		
X	X	Bustos	Jazmin	City of Merriam	Planner	Jbustos@merriam.org		
X	X	Kumke	Celia	City of Merriam	Public Works Director	CKumke@merriam.org	913-322-	
X	X	Roth	Steve	Merriam Drainage District	Engineer	sroth@benesch.com	913-553-7237	
X	X	Cline	Chris	Confluence	PRINCIPAL	cline@thinkconfluence.com	816.256.6650	
X	X	Kellenberger	Lee	Johnson County	Stormwater Manager	Lee.Kellenberger@jocogov.org	913-548-9914	
X	X	Schmidt	Heather	Johnson County		heather.schmidt@jocogov.org		
X	X	Asgian	Jacob	Johnson County	Project Engineer	jacob.asgian@jocogov.org	816-719-1867	
X	X	Pillar	Mike	Johnson County	Wastewater	mike.piller@jcw.org		
		Eisele	Ryan	Johnson County		ryan.eisele@hdrinc.com		
X	X	Parks	Shannon	JCW		shannon.parks@jcw.org		
X	X	Marx	Katrina	USACE	Project Manager	katrina.s.marx@usace.army.mil	816-389-3247	816-642-6070
X	X	Forsythe	Courtney	USACE	Tech lead			
X	X	Jones	Adam	USACE	H&H	Adam.A.Jones@usace.army.mil	816-389-3188	
X	X	Harden	Gary	USACE	charrette facilitator			
X	X	Grothaus	John	USACE	Planning			
X		HENGGELER	GEOFF	"	PM			

U.S. ARMY

MERRIAM
just right.

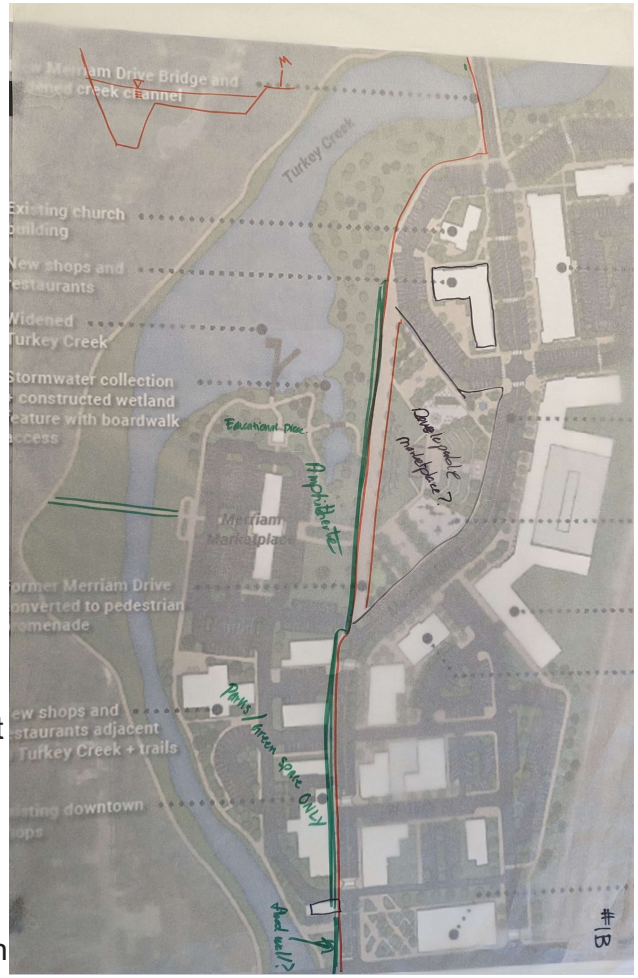
UPPER TURKEY CREEK - CONCEPT #1

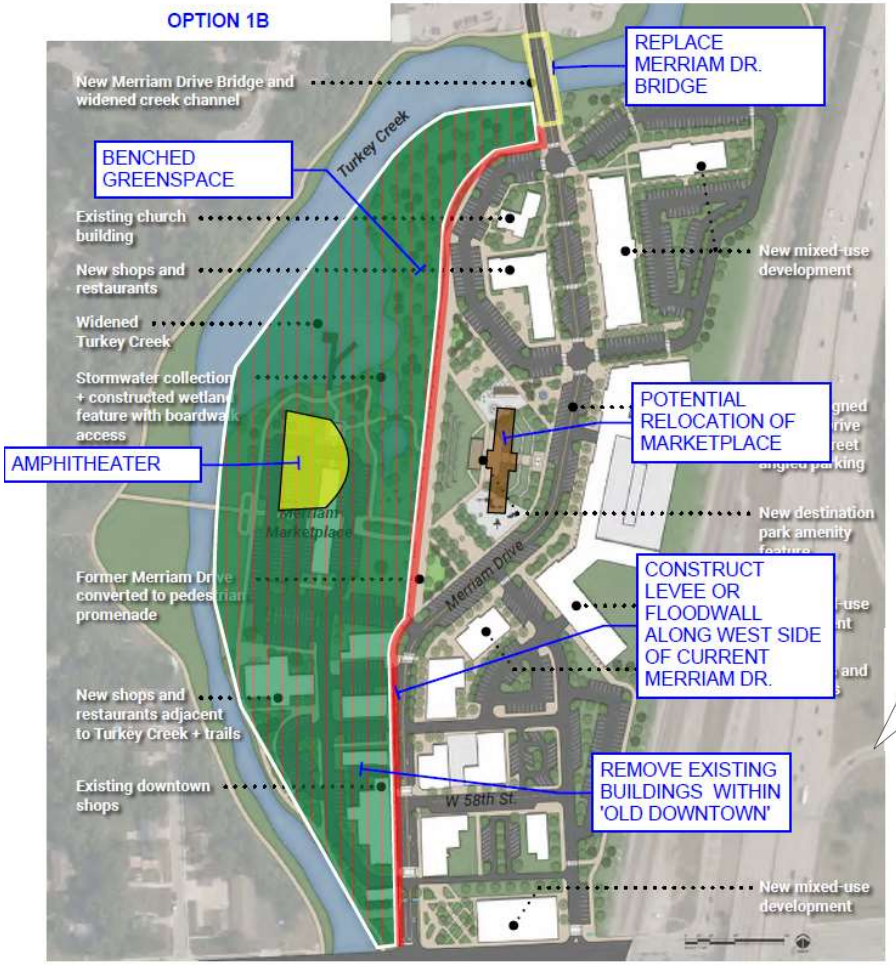




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

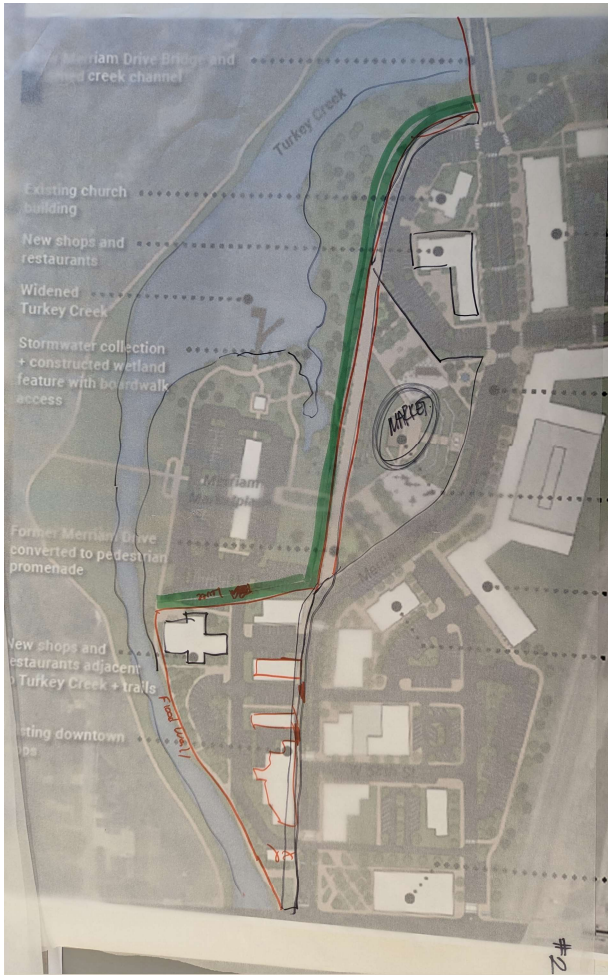
UPPER TURKEY CREEK - CONCEPT #1B

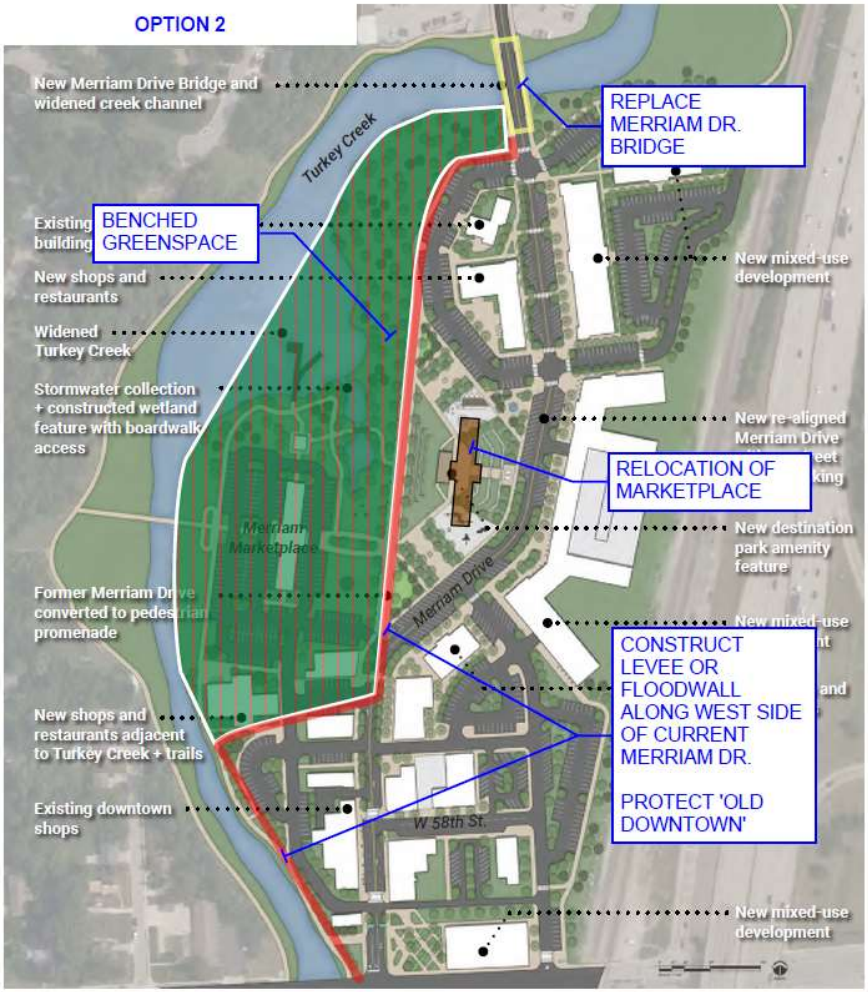




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

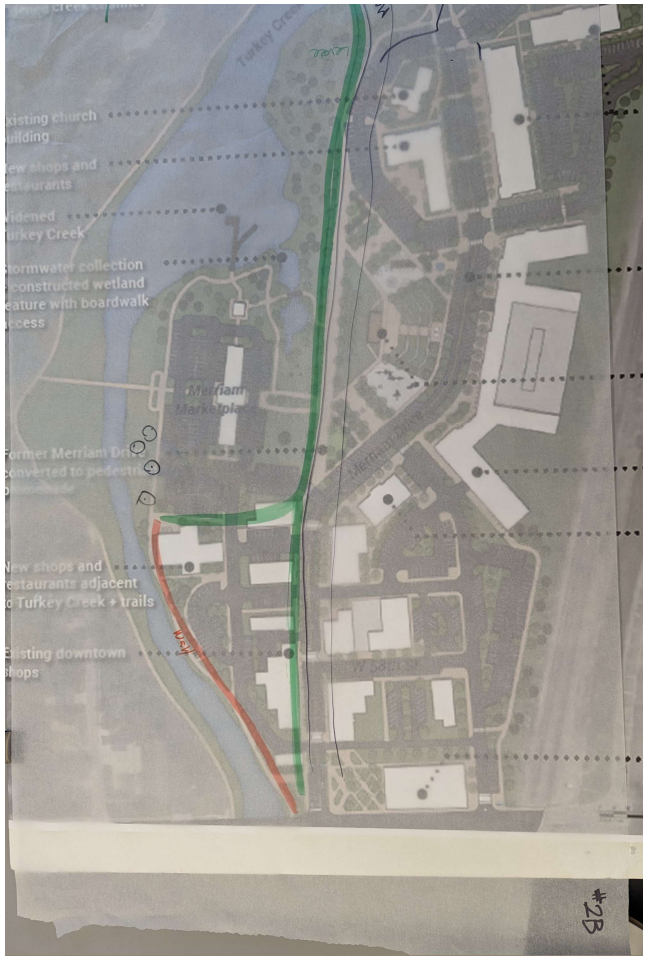
UPPER TURKEY CREEK - CONCEPT #2

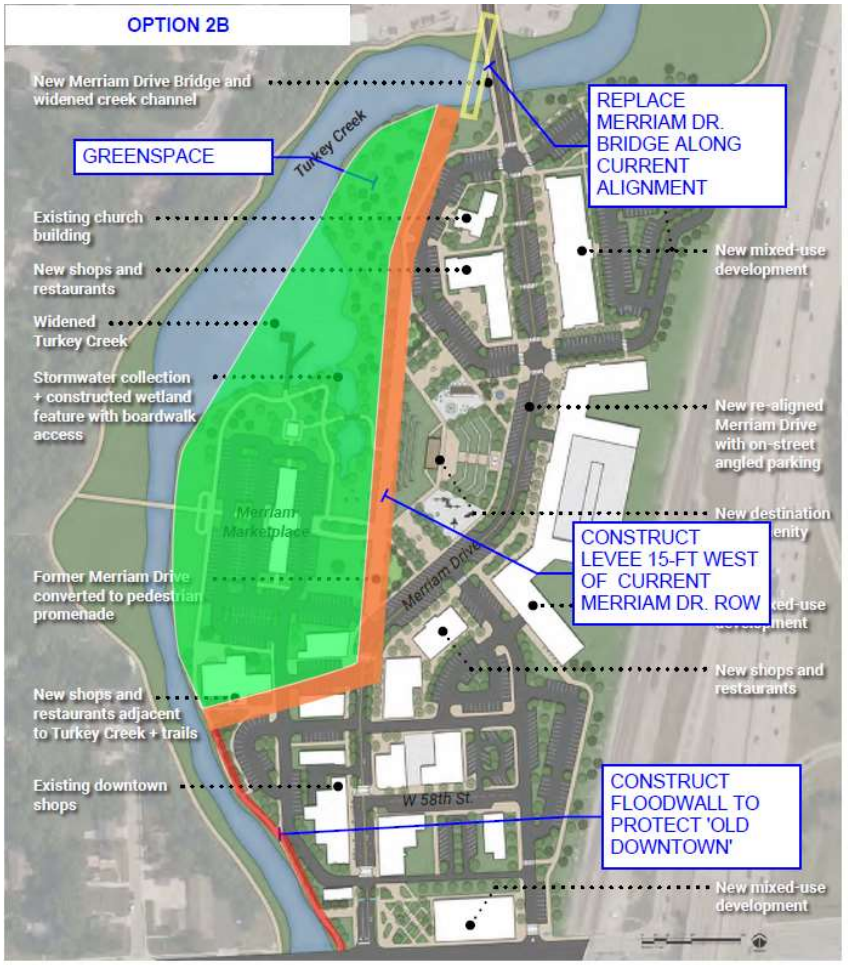




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

UPPER TURKEY CREEK - CONCEPT #2B

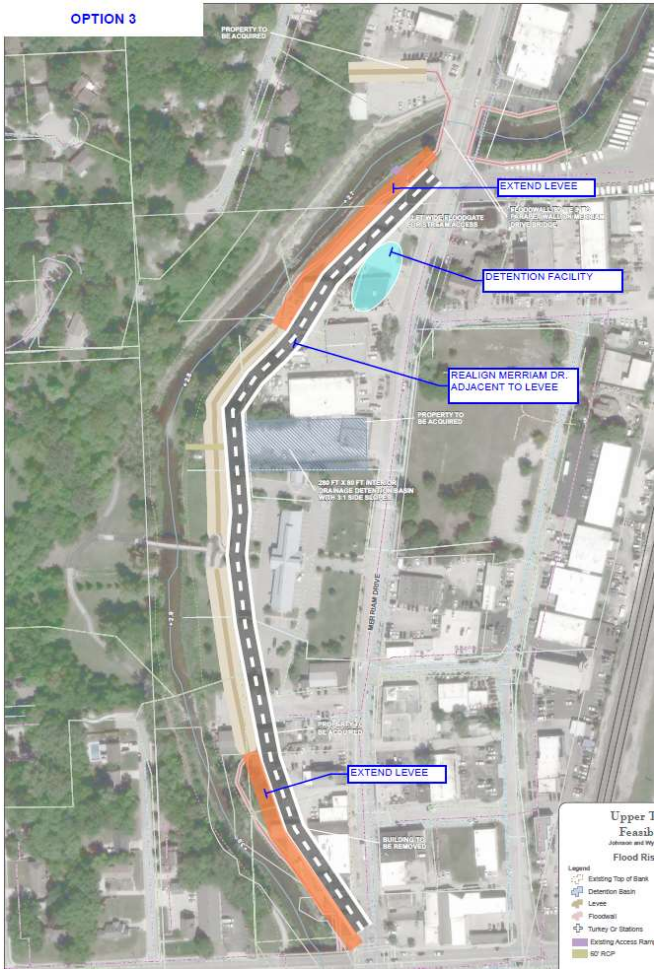




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

UPPER TURKEY CREEK - CONCEPT #3

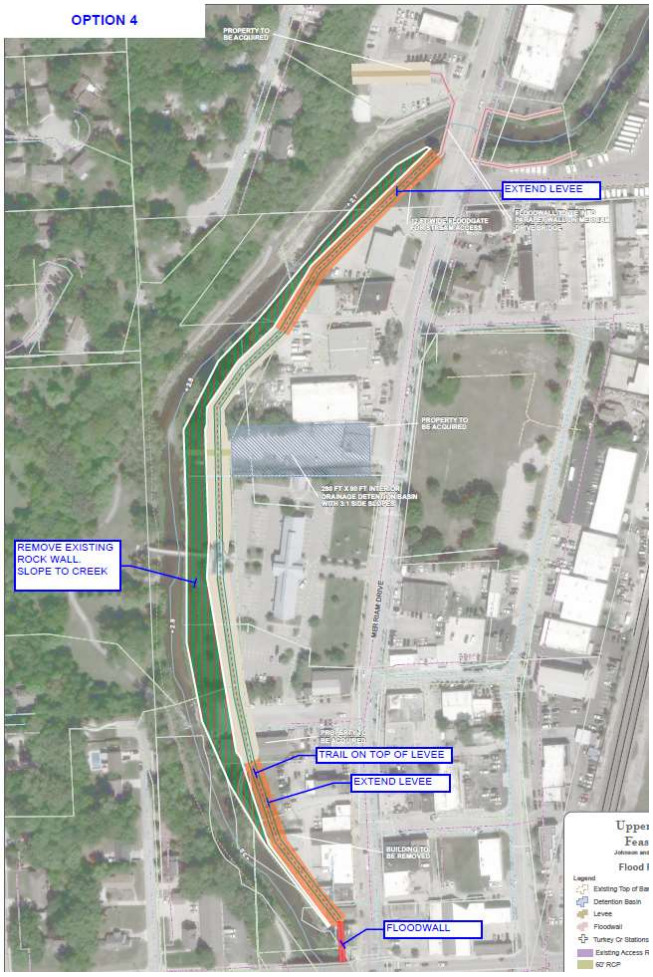




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

UPPER TURKEY CREEK - CONCEPT #4

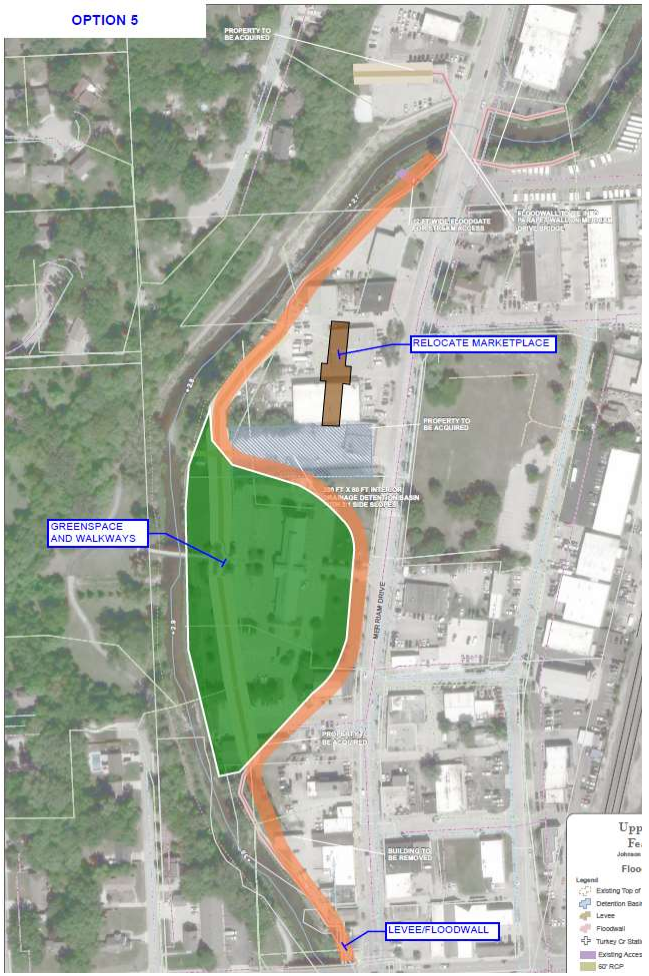




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

UPPER TURKEY CREEK - CONCEPT #5

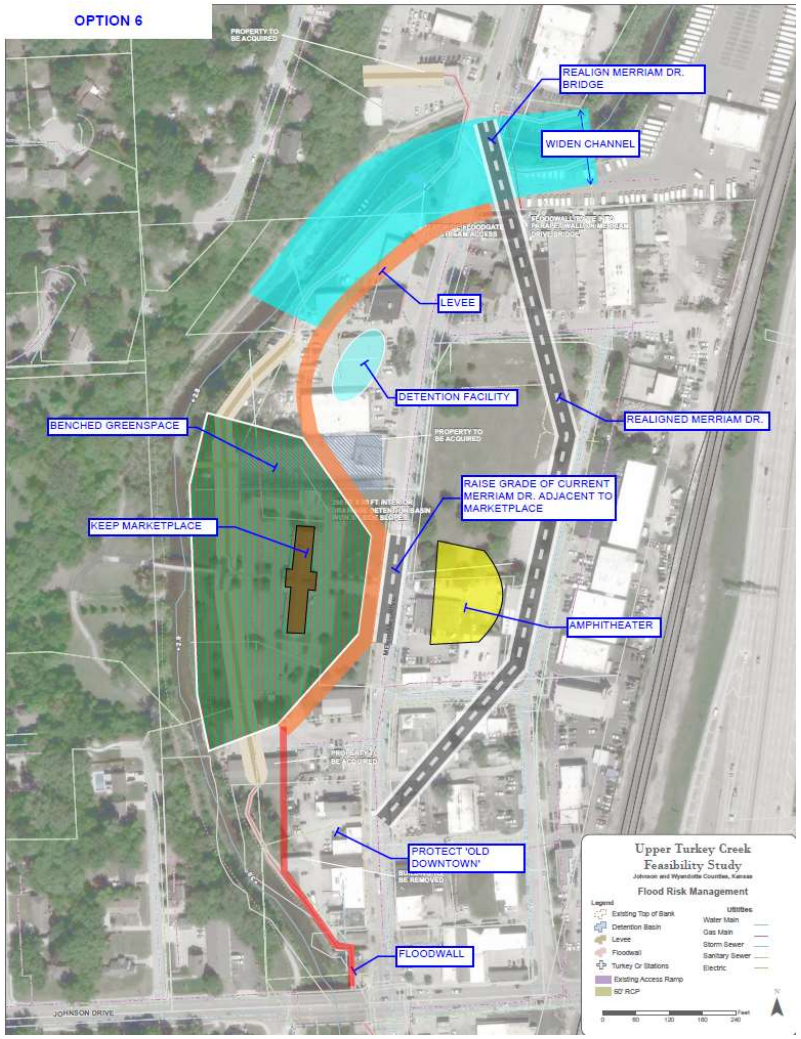




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

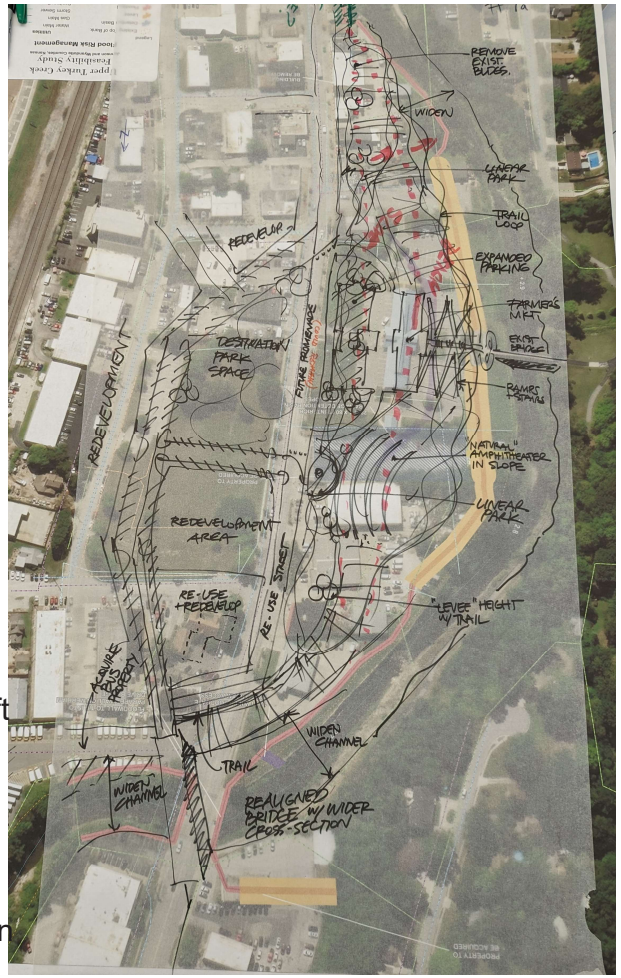
UPPER TURKEY CREEK - CONCEPT #6

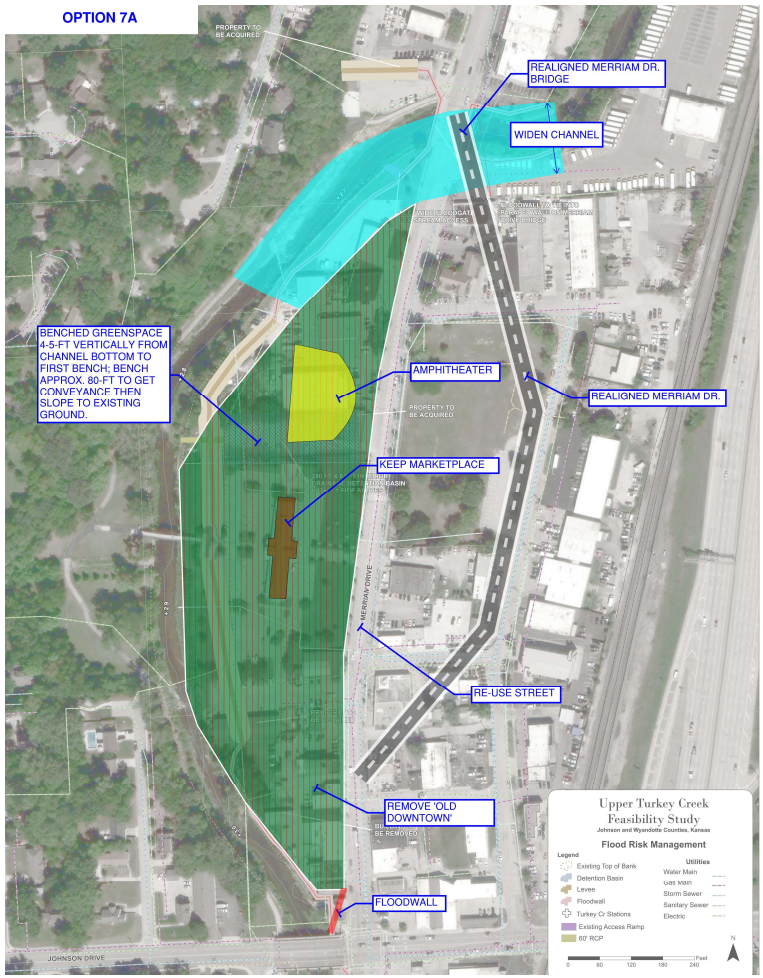




-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

UPPER TURKEY CREEK - CONCEPT #7A

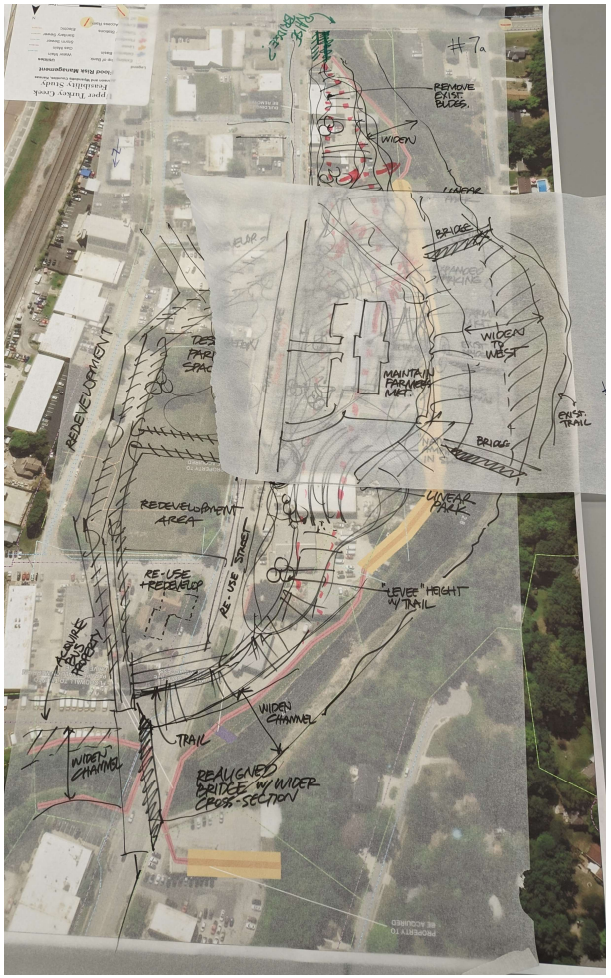


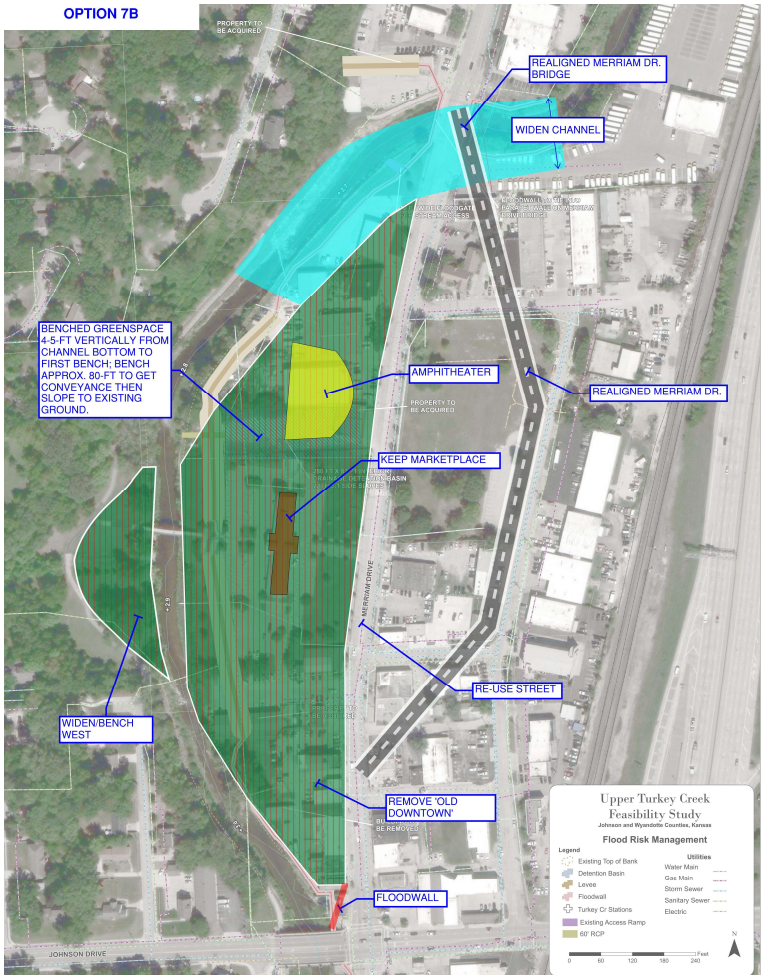


-Image on the left was sketched during the charrette.
-Image on the right is the annotated version.

MERRIAM
just right.

UPPER TURKEY CREEK - CONCEPT #7B





-Image on the left was sketched during the charrette.
-Image on the right is the annotated version

MERRIAM
just right.

UPPER TURKEY CREEK - EVALUATION

	carry forward?	LIKE	DON'T LIKE	RISKS	DESCRIPTION
1	no, 1b better	increasing green space and creek as an amenity; lower levee height because setback; farmers market next to amphitheater; opening up floodplain; only touching Merriam drive once; not as much SS relocation; city project - move road and relocate utilities ahead of flood reduction project	increases length of flood protection; don't gain much from bumpout to justify lengthening flood protection; less developable, more maintenace of green space; still need detention basin; loose historical buildings near johnson drive; adding buyouts and may not offset costs of flood protection; floodprotection hides the public spaces; utilities - would want more road and levee separation to allow for utility easements and eats into development space	more cost for merriam drive to be relocated now as part of this project; relocating merriam drive impacts developable area;	includes benching and new longer Merriam Drive bridge; eliminates buildings in old downtown; assumes relocated Merriam drive (but could keep Merriam Drive on current alignment too); assumes no buildings/development riverside of the new alignment
1b	yes	increasing green space and creek as an amenity; lower levee height because setback; farmers market next to amphitheater; opening up floodplain; only touching Merriam drive once; not as much SS relocation; city project - move road and relocate utilities ahead of flood reduction project; decreases length of floodprotection over 1a; more efficient hydraulically	less developable, more maintenace of green space; still need detention basin; loose historical buildings near johnson drive; adding buyouts and may not offset costs of flood protection; floodprotection hides less public spaces than 1a; utilities - would want more road and levee separation to allow for utility easements and eats into development space (a little less than 1b)	more cost for merriam drive to be relocated now as part of this project; relocating merriam drive impacts developable area;	same as 1
2	no, 2b better	increasing green space not as much as 1a/b; keeps old downtown - mix of old and new; lower levee height because setback; more levee than floodwall from FS plan; farmers market next to amphitheater; opening up floodplain not as much as 1a and b; only touching Merriam drive once	less developable, more maintenace of green space; still need detention basin; adding less buyouts than 1; floodprotection hides less public spaces than 1a	bridge relocation forces the city to relocate the road and negates the need to update merriam drive now	city doesn't need to relocate the bridge; USACE doesn't need to relocate the bridge; needs longer or overtopped for this project now
2b	yes will consider preserving old downtown area and replacing bridge on current alignment	no merriam drive relocation needed and city can decide what to do with Merriam drive			levee with separation of 15ft from merriam drive; no relocation of bridge but lengthing bridge still
3	no	maximizes developable area; levees less costly than levees and floodwalls; similar to FS - very little change to the authorized project	road next to levee - would need some separation from levee; little to no green space other than trail on levee top; creen is not really an amenity; would require all properties to be purchased by city alone; publically not a good option	public wont buy in	USACE alignment with all levee and Merriam Drive on alignment with levee; trail on top of levee
4	no	similar to FS but all levee; less cost levees instead of floodwalls; more takings than FS; more limited amount of riverside amenities could be added	no change to city plans - no green space or creek amenity; many sanitary conflicts like FS; loose old downtown		
5	no	similar to 6; less setback might still need higher levees/fw;	currently no impacts to the bus property; bumpin feature not very good benefit hydraulically		keeps existing merriam drive
6	yes consider bus property acquisition as a last resort for additional conveyance and detention	keeps old downtown; lower levee b/c setback; farmers market next to amphitheater; opens up floodplain; less costly levees vs floodwalls; meandering levee is more visually appealing; includes detention	more property acquisition of bus property - big impacts to that property;	bus company may have some sway with city council and likely if cut off access we will be into buying their whole property	raises current Merriam Drive, bridge lengthening; could include benching as needed
7a	yes	no floodprotection required - other than short floodwall DS of johnson drive (benching only); farmers market next to amphitheater; no detention needed; more natural visually; creek as amentiy; much of the development and road project is separate from flood risk reduction project	does not preserved old downtown; lower access to SS gets overtopped with water more frequently (water tight bolt down lids)	channel widening can result in sedimentation issues - could impact how the benching is done; could result in maintenance issues; big events drop out the sediment; would have to clear SS pipes	4-5 ft vertically from channel bottom to first bench; bench approximately 80ft to get conveyance then slope to existing ground; similar to 1b, but 1b has levees too
7b	yes	attempts to preserve market area where it is	ped bridge relocation or removal required		add west side of creek widening to 7a or really to any/all alternatives

CHARRETTE NOTES

Notable conclusions and comments from the Charrette:

- ❑ BRIDGES AND ROADS
  - Merriam Drive Realignment (see city corridor plan) may not be required if it is not needed to improve the alignment of the bridge at the Turkey Creek Crossing.
    - City currently has Johnson County CARS (County Assistance Road System) funding for Merriam Drive improvements along the current alignment.
  - Realigning the Merriam Drive bridge is not required for flood protection project.
    - Realigning would allow the bridge to be constructed without closing the existing bridge/road.
    - Realigning would shorten the bridge required to provide a larger creek opening, compared to keeping it on alignment, but additional real estate and roadway re-alignment may not be more cost effective in the end.
  - Replacing the Merriam Drive bridge with a longer bridge (more conveyance) will likely be required regardless because the 3-4ft parapet from the feasibility study is not a valid solution – currently sized at 6ft due to current hydraulic profile results in unstable bridge. Raising the bridge could also provide more conveyance, but could impact more properties.
  - Merriam Drive (regardless of alignment) will be considered the backbone of future development
  - Pedestrian bridge at Farmers Market is acknowledged as likely going to need to be removed as a result of benching in the area and could potentially be relocated with a city contract to another crossing location.
  - ADT on 55th St is 7-8000 vpd and on Johnson Drive 4-5000 vpd
  - Levee/Road – where the levee section is adjacent to the roadway would want separation – include a 15ft “buffer” between the levee toe and the roadway public right of way.

CHARRETTE NOTES - CONTINUED

Notable conclusions and comments from the Charrette:

- ❑ PROPERTIES
  - The marketplace can stay where it is or it can be relocated to another location – iconic area for Merriam. If it stays and is in flood plain, the city will need to include some warning system for the area in the event of a flood. Reconstructing in new location would be a city project and removal would be a levee project cost.
  - Bus property could be widened to increase flow capacity downstream – it would be a large and likely costly property to acquire since it is on both sides of the creek (functionally used together). If impacts to the bus property access are too great and have to acquire the property anyhow, it would be good to include for conveyance and/or detention.
  - There are no properties downtown that are on the historical register.
  - Need to consider trying to keep the old downtown area (between Johnson Drive and Hocker Drive on the West side of Merriam Drive) even though not a top priority for the city, in order to address question what is the impact project if we were to preserve that area. What are the consequences – Johnson Drive Bridge replacement, etc?
- ❑ UTILITY RELOCATIONS
  - If a utility is required to be relocated as part of the levee project and within their own easement, relocation costs are levee project costs and can be considered reimbursable LEERD (Land, Easements, Rights-Of-Way, Relocation, and Disposal Areas).

CHARRETTE NOTES - CONTINUED

Notable conclusions and comments from the Charrette:

- ❑ SANITARY SEWER
  - Most of the Sanitary Sewer in the downtown reach is 42” diameter and mostly in existing easement
  - Current estimates by JCW is a cost of $8-10M to relocate for the whole USACE project
  - Cover and clearance criteria/requirements is available on their website.
  - Current plans for upsizing elsewhere (large storage facility upstream of here), but if replacing/relocating required for this project they could include some upsizing of pipes instead or in addition to current plans.
- ❑ MISC
  - Merriam Drainage District expressed need for access to maintain limestone block wall and reminded the team of the constraint of not being able to traverse over sanitary sewer lines that cross in the creek bottom.
    - Merriam Drainage District acknowledges long term degradation of limestone block walls and material flowing through Upper Turkey Creek.
  - Feasibility study – 2g – which included Atlas 14 rainfall considerations – had a B/C of 1.8 vs B/C ratio of 2.0 for 2d (NED plan). See table 5-15 (page 5-36) in the feasibility study report.
  - City and County acknowledged that the decision to proceed with 2d was partially on the assumption that FEMA would not adopt the Atlas 14 updates (per FEMA/city meeting). Criteria to select recommended plan was based on net benefits, not on the ability for areas to be taken out of the floodplain.
  - Recreation improvements, such as trails – other than gravel trail and seeded grass – would be city funded (separate project or betterment to USACE project).
  - Impacts to local activities (ie at Marketplace) would be substantial during construction, but other options are available to accommodate – relocate and/or cancel.

CHARRETTE NOTES - CONTINUED

Notable conclusions and comments from the Charrette:

- ❑ MISC - Continued
  - Updated hydrology methodology will be incorporated in the hydraulic evaluation of the alternative (Johnson County currently working on the update and will provide near end of September)
  - Early City concepts show a water feature near the north end of the market place. During normal water levels, the water feature would be more of a dry detention area rather than a pond like effect. To hold water during normal water levels, a grade control structure would be required to back the water up in the creek enough to pond near the marketplace. Due to the relatively steep slope of the creek, the grade control structure would need to be several feet tall. Additionally, during high water the grade control structure would most likely need to be removed to maintain the capacity needed for the levee/floodwall system to handle the 1% AEP. Due to the flashiness of the creek, it would be difficult to temporarily remove this grade control structure in time without endangering personnel. During discussions, it was concluded this area is not conducive for a water feature like this to be implemented.

2024 CONCEPT MODELING RESULTS

CHARRETTE ALTERNATIVES 3-6





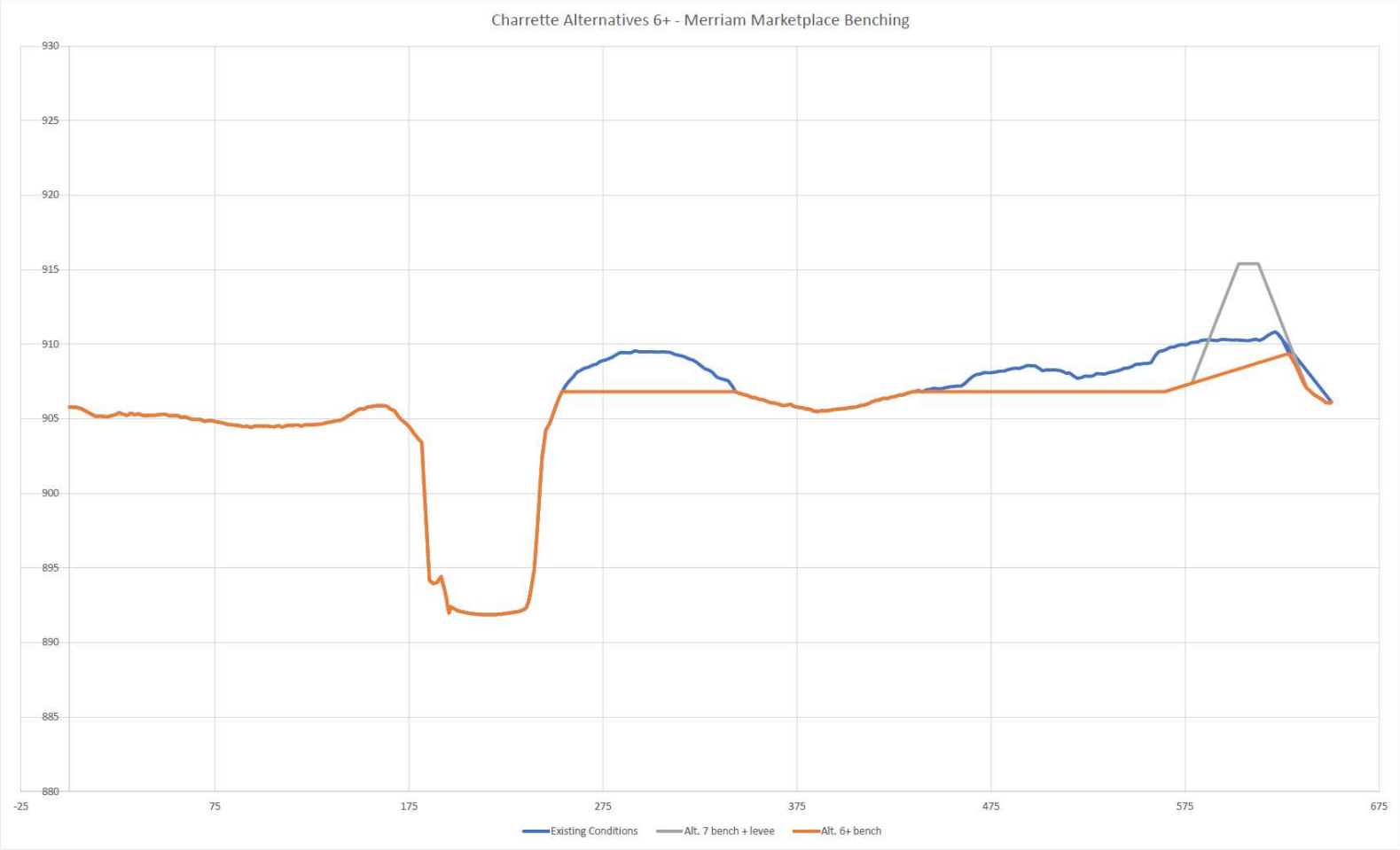
#3
Alternative 7a & 7b (including benching around market area) + preserve old downtown

#4
#3 + increase length of Merriam Drive bridge on current alignment

#5
#4 + remove market area and bench more

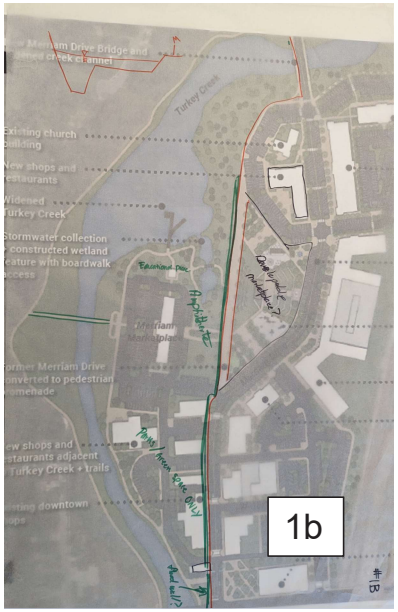
#6
#5 + take out old downtown for conveyance and floodwall transition from Johnson Dr.

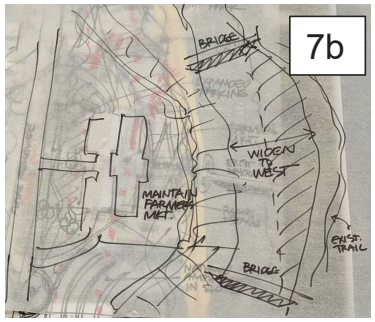
CHARRETTE ALTERNATIVE BENCHING



CHARRETTE ALTERNATIVES 7 & 8





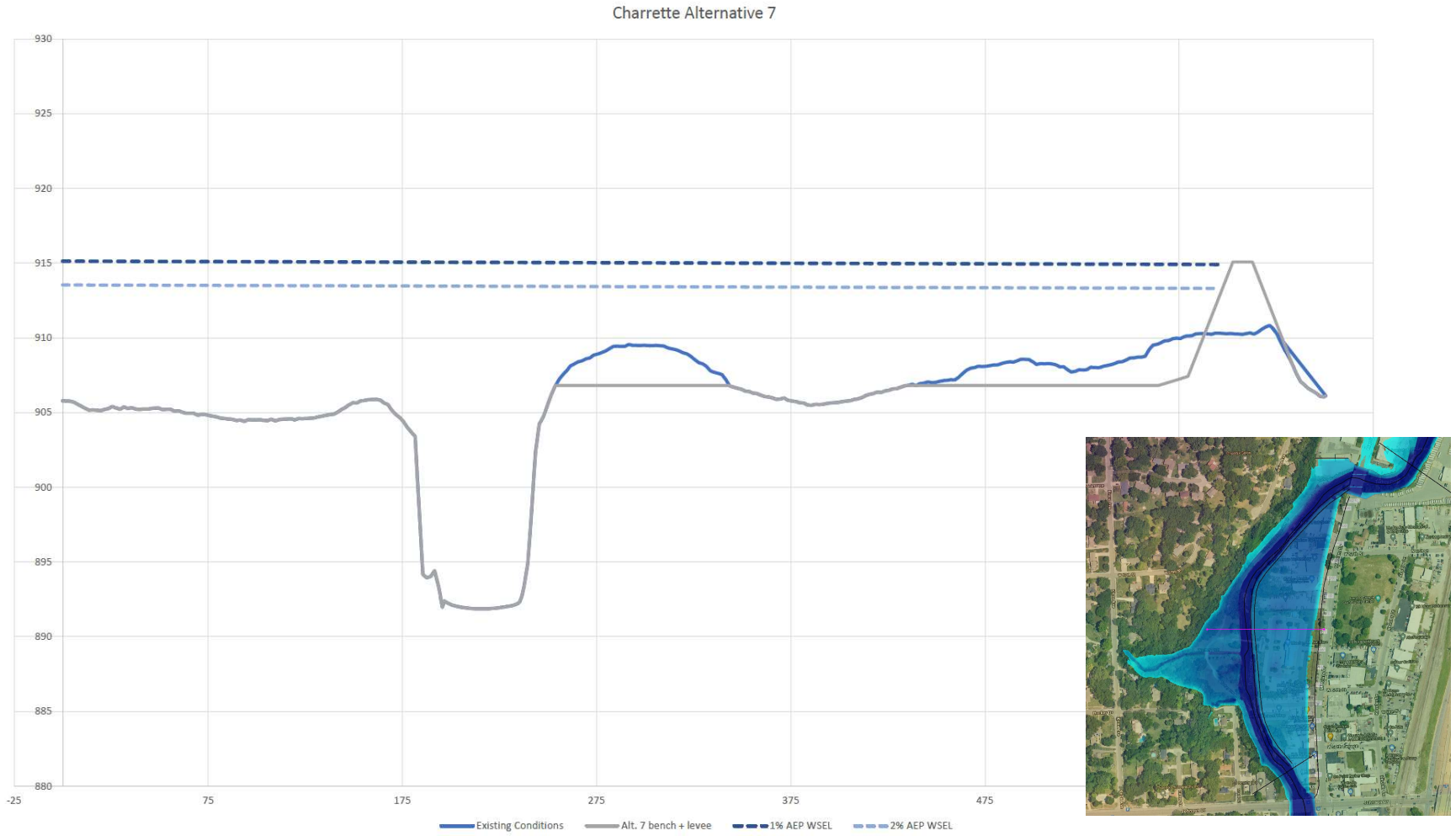


#7
#6 + add up to 2-3feet of levee as needed (see Alternative 1b)

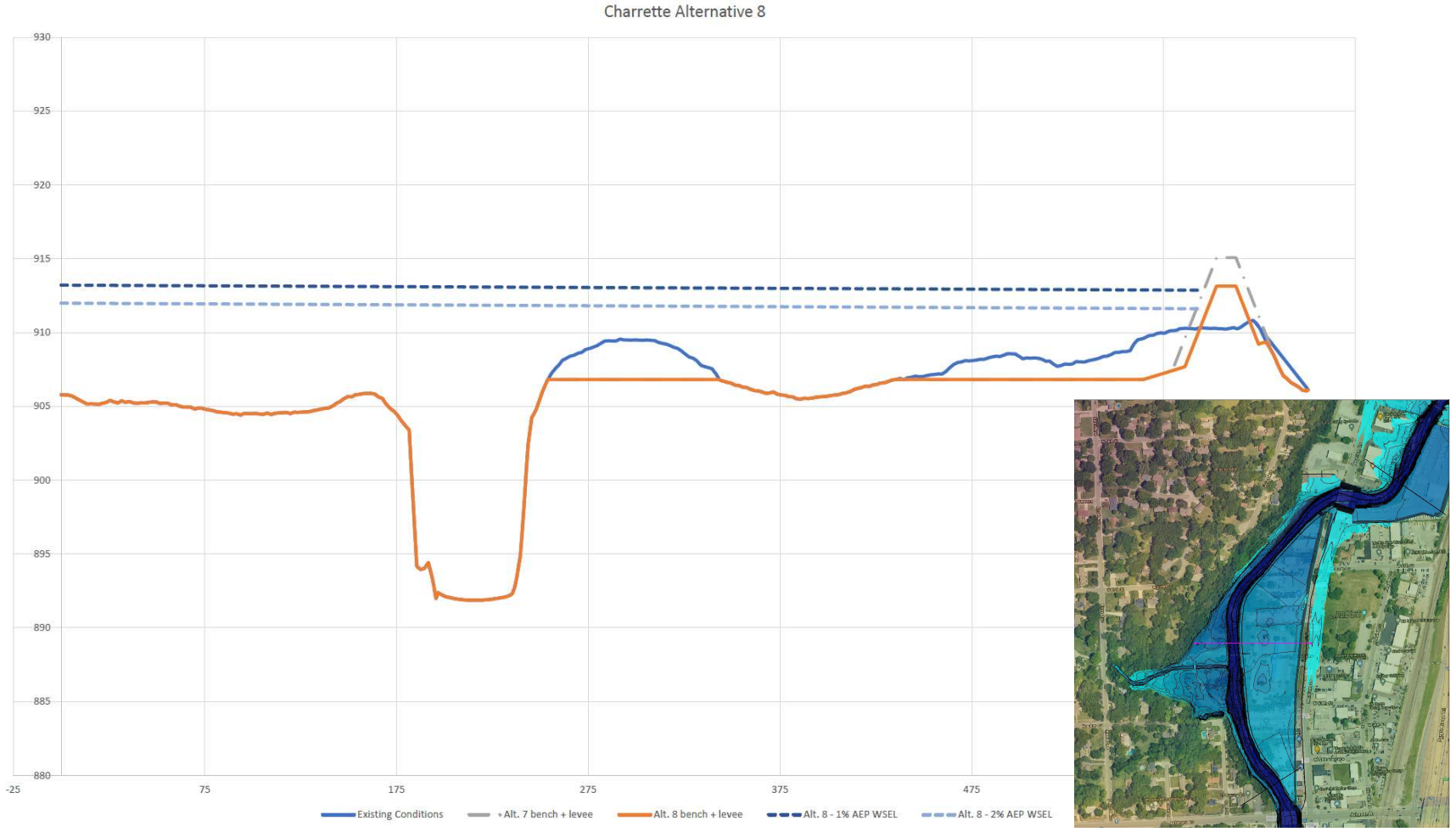
#8
#6 + bus property channel widening and remove bus private bridge

MERRIAM
just right.

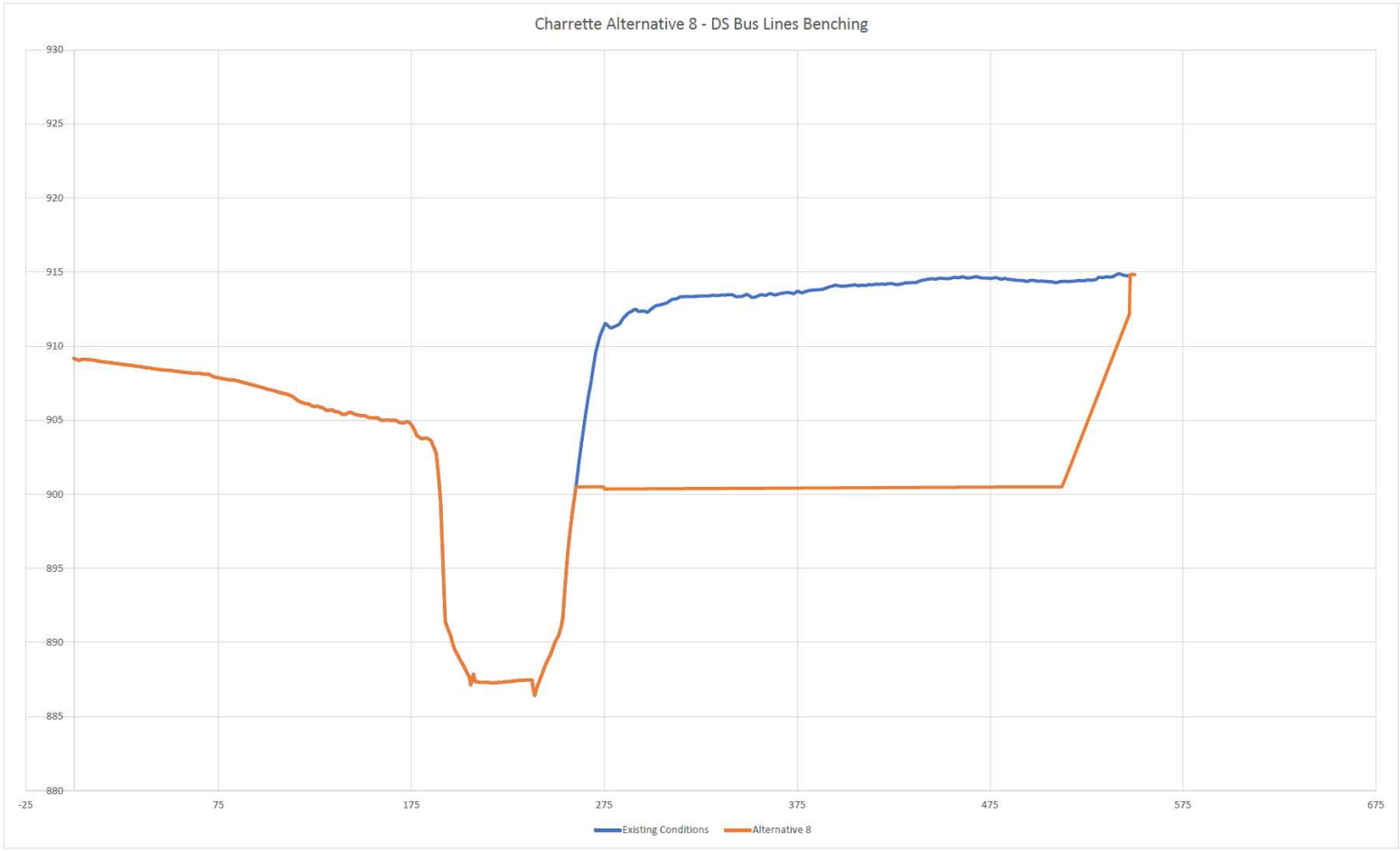
CHARRETTE ALTERNATIVE – ALTERNATIVE 7



CHARRETTE ALTERNATIVE – ALTERNATIVE 8



CHARRETTE ALTERNATIVE 8 ADD'L BENCHING



APPENDIX K – SPECIFICATION RESPONSIBILITY ASSIGNMENT MATRIX

K.1 SRAM

This page was intentionally left blank for duplex printing.

PROJECT TITLE: Upper Turkey Creek — **SPECIFICATION RESPONSIBILITY ASSIGNMENT MATRIX**

* Sections with a fifth level number 10=USACE, 20=NAVFAC, 23= USACE NWK, 30=AFCEC, 40=NASA

				SUBMITTAL		DESIGNER RESPONSIBILITY				
SPECIFICATION NUMBER *	SPECIFICATION TITLE	PRIMARY DISCIPLINE	LOCAL NOTES	65	95	CIVIL	ENVIRONMENTAL	GEOTECH	STRUCTURAL	TECHNICAL SERVICES
01 11 00	SUMMARY OF WORK	PROJECT MGR		X	X					
01 14 00	WORK RESTRICTIONS	PROJECT MGR		X	X					
01 22 00.00 10	PRICE AND PAYMENT PROCEDURES	PROJECT MGR		X	X					
01 30 00	ADMINISTRATIVE REQUIREMENTS	PROJECT MGR		X	X					
01 32 01.00 10	PROJECT SCHEDULE	PROJECT MGR		X	X					
01 33 00	SUBMITTAL PROCEDURES	PROJECT MGR		X	X					
01 35 26	GOVERNMENTAL SAFETY REQUIREMENTS	PROJECT MGR		X	X					
01 45 00.00 10	QUALITY CONTROL	PROJECT MGR		X	X					
01 45 00.15 10	RESIDENT MANAGEMENT SYSTEM CONTRACTOR MODE(RMS CM)	PROJECT MGR		X	X					
01 50 00	TEMPORARY CONSTRUCTION FACILITIES AND CONTROLS	PROJECT MGR		X	X					
01 57 19	TEMPORARY ENVIRONMENTAL CONTROLS	PROJECT MGR		X	X					
01 78 00	CLOSEOUT SUBMITTALS	PROJECT MGR		X	X					
01 78 23	OPERATION AND MAINTENANCE DATA	PROJECT MGR		X	X					
02 41 00	DEMOLITION	CIVIL		X	X	MAEVE VANLAN				
02 61 13	EXCAVATION AND HANDLING OF CONTAMINATED MATERIAL	ENVIRONMENTAL		X	X		PAUL SPECKIN			
03 30 00	CAST-IN-PLACE CONCRETE	STRUCTURAL		X	X				EDDIE FERNANDEZ	
03 42 13.00 10	PLANT-PRECAST CONCRETE PRODUCTS FOR BELOW GRADE CONSTRUCTION	STRUCTURAL		X	X				EDDIE FERNANDEZ	
05 50 14	STRUCTURAL METAL FABRICATIONS	STRUCTURAL		X	X				EDDIE FERNANDEZ	
05 50 15	CIVIL WORKS FABRICATIONS	STRUCTURAL		X	X				EDDIE FERNANDEZ	
09 97 02	PAINTING: HYDRAULIC STRUCTURES	STRUCTURAL		X	X				EDDIE FERNANDEZ	
31 05 19.13	GEOTEXTILE	GEOTECH		X	X			DEREK JENSON		
31 41 16	METAL SHEET PILING	GEOTECH		X	X			DEREK JENSON	EDDIE FERNANDEZ	
31 60 00	FOUNDATION PREPARATION	GEOTECH		X	X			DEREK JENSON		
32 15 00	AGGREGATE SURFACE COURSE	GEOTECH		X	X			DEREK JENSON		
32 16 13	CONCRETE SIDEWALKS AND CURBS AND GUTTERS	CIVIL		X	X	MAEVE VANLAN		DEREK JENSON		
32 31 13	CHAIN LINK FENCES AND GATES	CIVIL		X	X	MAEVE VANLAN				
32 92 19	SEEDING	CIVIL		X	X	MAEVE VANLAN				
33 40 00	STORM DRAINAGE UTILITIES	CIVIL		X	X	MAEVE VANLAN				
35 20 16.00 23	FLAP GATES MISCELLANEOUS DRAINAGE STRUCTURES	STRUCTURAL		X	X				EDDIE FERNANDEZ	
35 31 19	STONE, CHANNEL, SHORELINE/COASTAL PROTECTION FOR STRUCTURES	GEOTECH		X	X			DEREK JENSON		
35 41 00	LEVEE CONSTRUCTION	GEOTECH		X	X			DEREK JENSON		

This page was intentionally left blank for duplex printing.

APPENDIX L – ENGINEERING CONSIDERATIONS AND INSTRUCTIONSP FOR FIELD PERSONNEL (ECIFP)

This page was intentionally left blank for duplex printing.

L ENGINEERING CONSIDERATIONS AND INSTRUCTIONS FOR FIELD PERSONNEL (ECIFP)

Project Name: Upper Turkey Creek FRM

Project Location: Merriam, Kansas

P2 Number: 505229

L.0 VERSION HISTORY

Version No.	Version Date	Description of Change
0.0	11/26/2024	Original Version
1.0	03/242026	65% Design Update

L.1 PURPOSE

L.1.1 Purpose

This Engineering Considerations and Instructions to Field Personnel (ECIFP) report expands on the construction specifications and drawings in areas where added information, not considered essential to the Contractor, is considered helpful to construction personnel administering the contract. This information helps ensure that project features are constructed in accordance with design assumptions, constraints, and overall intent.

The ECIFP points out critical construction issues and concerns and highlights specific items that may need additional clarification. This document does not modify and should not conflict with contract plans and specifications. The Design Analysis (DA) is also available to the field team and documents the logic and calculations relevant to design decisions. Neither the DA nor the ECIFP are part of the contract.

This document establishes a baseline for communication and coordination between the design and construction personnel; not every issue is addressed in this document. Feedback and open communication between designers and field personnel, prior to and during construction, are encouraged.

L.1.2 Key Contacts

Name	Role	Phone	Email (@usace.army.mil)
Kailey Jones	Project Manager	816-389-2123	kailey.j.jones
Courtney Forsythe	Technical Lead	816-389-2256	courtney.a.forsythe
Conner Szarwinski	H&H	816-389-2006	conner.szarwinski
Maeve VanLandingham	Civil	816-389-2425	maeve.k.vanlandingham
John Westbrook	Structural Engineer	816-389-3249	john.w.westbrook
Derek Jenson	Geotechnical Engineer	816-389-2323	derek.h.jenson

L.2 GENERAL PROJECT INFORMATION

L.2.1 Project Description

a. Refer to DDR Part 01 for Project Narrative.

L.2.2 Work Restrictions

a. The creek is very flashy, so there isn't much advanced warning available. The Contractor will need to park their equipment overnight above the existing bank elevation. We will account for the creek coming up as weather days and not plan to issue stop work.

b. Some of the construction will require a lane to be closed along various roads including Mastin St near Shawnee Mission Pkwy, Merriam Dr just south of Johnson Dr, and Merriam Lane just south of Johnson Dr. Per a meeting with the City October 2024 they prefer for lane closures to not impact traffic as much as possible. This means that both directions should be able to still pass, and the duration of that closure should be kept to a minimum.

c. Utility relocations will be completed by third-party owners but will require coordination with the Contractor during Construction.

d. There will be two roadway bridges replaced as part of this project.

- Details of the traffic control will be worked through with the City of Merriam with potential input from the City of Shawnee. Through traffic from the west that would use Johnson Drive to reach I-35 entrance ramps would be detoured at the intersection of Johnson Drive and Nieman Road southbound on Nieman Road to Shawnee Mission Parkway, then eastbound to the entrance ramps of I-35 at Shawnee Mission Parkway. Destinations along Johnson drive east of the intersection of Nieman Road and the Bridge can still be accessed by setting up barricades for the road closure approximately 100 feet to the east of the intersection of Grant Street and Johnson Drive and using that intersection as a turn around point and construction entrance for the construction group.
- For the bridge construction at Merriam drive traffic could be diverted to the north to west 55th Street and to the south could be diverted to Johnson Drive. The two bridges would not be able to be done at the same time. The diverted traffic from south of the Merriam drive bridge would be able to then use Knox Ave to the west, but would need access via Johnson Drive Bridge.

e. There are several businesses impacted by floodwall/levee construction along the back of their property. Some of these impacts could negatively affect their business. It's important that the Contractor limit the impact per property owner. Specifically, floodwall constructed at the bus barn on the downstream end of the project should be limited to the school's summer break.

f. Care should be taken to not disturb the existing limestone block wall unless called out to remove in the plans. This wall is not stable enough to handle tracking of construction equipment.

g. RB floodwall south of Johnson Drive will start at the newly constructed bridge and head south. Due to limited space, the Contractor may not be able to pour the floodwall in alternating monoliths where space is limited, especially adjacent to the existing buildings. They may need to pour one base, wait the required 7 days, and then pour the next base immediately adjacent. This is possible but will drastically slow production. The Contractor is allowed to over excavate for a working pad, but should avoid excavating the stacked limestone block wall. If they over excavate, they may need to place more riprap than what is shown in the Drawings.

L.2.3 Project Schedule / Phasing

The current plan is to have two contracts. One for Merriam and Johnson Drive bridge, and one for the remaining levee and floodwall construction. The bridge contract will need to be complete prior to starting the levee and floodwall contract. It's estimated that each bridge will take one year to complete. They cannot be constructed at the same time since this would severely impact traffic in Merriam. One bridge at a time is allowed to be taken completely out of service to construct the new bridge. The bridge contract will total roughly two years. The levee and floodwall contract is estimated to take roughly three years to complete.

L.2.3.1 Conceptual Construction Phasing

a. Merriam Drive Bridge

1. Close the road and detour traffic. Maintain access to the few businesses adjacent to the bridge.
2. Coordinate utility relocations that could not be relocated before construction.
3. Demolish the bridge
4. Construct new bridge
5. Construct intersection/road tie ins

b. Johnson Drive Bridge

1. Close Johnson Drive from Campbell Lane to Merriam Drive and detour traffic. Keep east intersection open.
2. Coordinate utility relocations that could not be relocated before construction.
3. Demolish the bridge
4. Construct new bridge
5. Close Johnson Drive/Merriam Drive intersection. Construct intersection and Johnson Drive tie-in
6. Construct Merriam Drive tie in north of the intersection.
7. Construct Merriam Drive tie in south of the intersection.

c. Levee and Floodwall Construction Contract

1. It's assumed the Contractor will divide the levees and floodwalls into left bank, right bank from Shawnee Mission Pkwy to Johnson Drive, and right bank at downtown Merriam.
2. Demolish buildings within construction limits
3. Demolish other features within construction limits
4. Coordinate utility relocations
5. Construct storm features
6. Excavate detention basin and use any suitable material to construct the levee through this reach. Conceptual quantities suggest there should be enough suitable material to construct the levee. During drilling, petroleum was discovered. Care should be taken to dispose of any contaminated soil.
7. Construct levee and access ramps
8. Construct floodwalls. It's assumed the Contractor will start at the newly constructed bridges and work their way out from there.
9. Final cleanup and seeding

L.2.4 Haul Routes/Staging Areas

a. Haul routes for Construction will be limited to public roads. There are some private roads within the project limits that should not be used for haul routes. See Drawings haul routes.

b. Staging areas will be limited to land owned by the City of Merriam, Merriam Drainage District, or properties to be acquired as part of the project. The Contractor can elect to acquire additional temporary easement agreements with the property owners if they wish.

L.2.5 Advanced Modeling Requirements

a. Refer to Part 17 – References, paragraph 17.4 Computer Programs.

L.3 DISCIPLINE SPECIFIC INFORMATION

L.3.1 Environmental

a. Hazardous, Toxic, and Radiological Waste

A boring (ADU-201A) encountered petroleum contamination 10-ft bgs north of the parking lot on at 5920 Merriam Dr. A floodwall is proposed 40-ft was of this location. If petroleum contaminated soil is encountered during installation of the floodwall it will need to be disposed off-site at a permitted facility.

Another boring (ADU-212) encountered buried debris 5-ft bgs at 5726 Merriam Dr. A detention pond and levee is proposed at this location so there may be a large volume of debris encountered that requires disposal at off-site at a permitted facility.

A GPR survey identified a number of subsurface anomalies. The nature of the anomalies is unknown. Properties where the anomalies were encounter are listed below:

- 5568 Merriam Dr
- 5643 Merriam Dr
- 5726 Merriam Dr
- 9420 W 61st St
- 9420 W 61st St
- 80 feet west of the intersection of W 61st St and Merriam Ln

The location of these anomalies can be found in Appendix F – Environmental.

It is possible during construction of the flood control features that currently unknown contamination or debris will be encountered. The specifications should require the Contractor to prepare a Contingency Plan outlining actions they will take if contamination soil or debris are encountered.

L.3.2 Geotechnical

a. Levee Inspection Trenches

The levee segments of the system will have inspection trenches to verify satisfactory foundation soils. An inspection trench will be excavated along levee centerline for all levee reaches. For levees 6 feet or higher, the trench will be 6 feet deep. For shorter levees, the depth of the trench will be equal to the levee height. The trench width should be approximately 6 feet to allow for compaction equipment.

The inspection trench will allow for detection of any continuous pervious zones, voids, or unacceptable fill material. The trench will be backfilled with the excavated material compacted in lifts. If unacceptable material is encountered, it will be replaced with impervious fill to ensure there are no continuous seepage paths.

The construction QA representative should observe all inspection trenches and communicate any issues with the project geotechnical engineer. The geotechnical engineer should be present with the QA representative for the first inspection trench and make regular weekly visits to check progress thereafter.

b. Floodwall Foundations

The construction QA representative should observe footing excavations for T-walls and should observe installation of sheet piles for I-walls, with the geotechnical engineer making regular weekly visits during foundation preparation.

L.3.3 Civil

a. There is an existing trail system running along and within the project limits. There will need to be some additional signage for the public during Construction. The Sponsor will determine trail detour routes, if required. Staging areas adjacent to existing trails may require additional security measures as equipment and potentially hazardous work areas could be visible to the public.

L.3.4 Structural

a. Gatewells & Manholes

There is an existing limestone block wall within the creek. Portions of it will need to be removed to outlet storm drainage during Construction. See the plans for details.

b. Enlarging the key of the floodwalls for ease of construction for contractor

During conversations with team members of the Construction Division it was learned that in past projects the contracts have requested to make an extra thick key to ease construction efforts and to gain efficiencies. The additional materials

required to make the keys much thicker than the design thickness were offset by the gained efficiencies. During the design preliminary CT Wall runs were made for the 10 foot tall wall profile. These preliminary runs did not meet the required minimums for factors of safety for sliding failure. If the request is made by the contractor to use an alternate profile for the wall's foundation to ease in the construction, the structural engineer should be engaged early in that process to determine if the key that is being suggested by the contractor will sufficiently resist sliding and overturning.

L.3.5 Hydrology & Hydraulics

a. Woody vegetation, including trees and shrubs, will need to be cleared within 15-feet of the levee and floodwall toe for both the riverside and landside.

b. As mentioned above, Turkey Creek watershed is very flashy with no automatic flood stage forecasting. 2-3 inches of rainfall within an hour could result in isolated areas of out of bank flooding. Monitoring rainfall forecasts is recommended.

L.4 UTILITY / PERMIT INFORMATION

All utility relocation coordination is the responsibility of the non-Federal Sponsor. Utility coordination will begin after USACE issues the RE NTP. USACE personnel will provide technical support during utility relocation coordination.

L.5 RECOMMENDED SITE VISITS BY DESIGN PERSONNEL

Will be completed after 50%.

APPENDIX M – INTERIOR DRAINAGE MODEL DEVELOPMENT

M.1 STANTEC'S TURKEY CREEK 2D WATERSHED HYDROLOGIC AND HYDRAULICS REPORT

M.2 PRECIPITATION TIME SERIES

M.3 HEC RAS PIPE NETWORK INFORMATION

M.4 DRAINAGE STRUCTURE INFORMATION

APPENDIX M.1 – RAINFALL FOR FLOOD STUDIES: INDIAN AND TOMAHAWK CREEKS

Research Report

Rainfall for Flood Studies: Indian and Tomahawk Creeks

Prepared for
Stormwater Management Program
Johnson County Public Works

By
Bruce McEnroe, PhD, PE
Bruce McEnroe Consulting LLC

November 30, 2023

1. Background and Objectives

Johnson County is currently working to update its floodplain maps using HEC-RAS 2D modeling technology. The updated 2D models for the Indian Creek and Tomahawk Creek watersheds have been developed but not yet finalized. One remaining issue is selection of rainfall inputs for estimation of flood levels with specified recurrence intervals. This report provides findings and recommendations from research on that issue. The main concerns are the storm duration, the temporal distribution of rainfall within the storm, and the possible application of areal reduction factors to point rainfall for watersheds over a certain size.

The Federal Emergency Management Agency (FEMA) provides some basic guidance for rainfall inputs to flood models in the *Hydrology: Rainfall-Runoff Analysis* (2019) volume of its *Guidance for Flood Risk Analysis and Mapping* document series. FEMA's guidance includes the following statements:

- "For most applications reflected on FIRMs, the spatial distribution of rainfall is taken to be constant."
- "Temporal storm distributions must be chosen to reflect the local climatic conditions."
- "The choice of temporal storm distribution must be fully documented. If the source of the distribution is not a federal, state, or regional agency, the documentation must include a detailed description of the derivation of the distribution, including sources of data and the means of fitting those data to a particular distribution."
- "The storm duration chosen must exceed the time necessary for runoff everywhere in the basin to reach the outlet, also known as the time of concentration."
- "The Mapping Partner must document the use of areal reduction factors (or lack thereof)."

This report addresses all of these points and provides fully documented recommendations for the Indian and Tomahawk Creek flood models with potential application to other Johnson County flood models.

2. Gage Records and Flood Frequency

Indian Creek is a major stream in the suburban area of east-central Johnson County. At the Kansas-Missouri state line (the eastern boundary of Johnson County), its drainage area is 64.2 mi^2. Tomahawk Creek is a right-bank tributary of Indian Creek. Tomahawk Creek joins Indian Creek 0.8 miles west of the state line, at which point its drainage area is 23.3 mi^2.

The U.S. Geological Survey (USGS) currently maintains two streamflow gages on Indian Creek and one gage on Tomahawk Creek. Station 6893300, Indian Creek at Overland Park, is located on the downstream side of the bridge at Marty Street, where the drainage area is 26.6 mi^2. From March 1963 until May 1977, the gage was located 700 feet downstream at 103rd Street. Peak-flow records are available from water year 1964 to present, and continuous stage and discharge records date from 1989. Station 6893390, Indian Creek at State Line Road, is located on the downstream side of the bridge at that location. Continuous stage and discharge records at State Line Road date from March 2003. Station 6893350, Tomahawk Creek near Overland Park, is located on the downstream side of the bridge at Roe Avenue. The drainage area at that location is 20.5 mi^2. Continuous stage and discharge records at the Tomahawk Creek gage date from July 2011. Because USGS stage and discharge data are recorded and reported at 15-minute intervals, those data can slightly understate the actual flood peaks.

Johnson County's extensive network of ALERT flash-flood warning system includes 28 rainfall gages and 22 water-level gages in the Indian and Tomahawk Creek watersheds. Many other gages surround the two watersheds. The ALERT gage records provide high-resolution data that are useful for flood model calibration.

The USGS gage on Indian Creek at Marty Street is the only discharge gage in the two watersheds with a record of sufficient length for a meaningful statistical analysis. The record includes 59 years of peak-flow data (1964-2022). Figure 1 shows the results of a flood-frequency analysis for the entire peak-flow record by the Bulletin 17C methodology, using the station skew rather than a weighted skew. This analysis yields a 1%-annual-chance discharge of 13,560 cfs and a recurrence interval of 134 years for the 8/22/2017 flood of record. However, six of the seven highest annual peaks lie above the fitted curve, which suggests that the frequency analysis might overstate the recurrence intervals of the largest floods.

In the USGS dataset of annual peak flows for station 6893300, the values from 1980 to present are flagged as significantly impacted by urbanization. The Indian Creek watershed upstream of the Marty Street gage did undergo substantial urbanization during the first two-thirds of the period of record, with hydrologic impacts increasing gradually over time. For present purposes, the current hydrologic regime can be considered to start in 1977, when the USGS gage was relocated to the new Marty Street bridge. Figure 2 shows the results of a Bulletin 17C flood-frequency analysis for the 46-year period 1977-2022. This analysis yields a 1%-annual-chance discharge of 15,020 cfs and a recurrence interval of 80 years for the flood of record on 8/22/2017. Table 1 compares the flood-frequency estimates from the analyses of the 1964-2022 and 1977-2022 annual peaks. The estimates derived from the 1977-2022 data probably better reflect current conditions because the Indian Creek watershed upstream of Marty Street was much less developed prior to 1977 than at present.

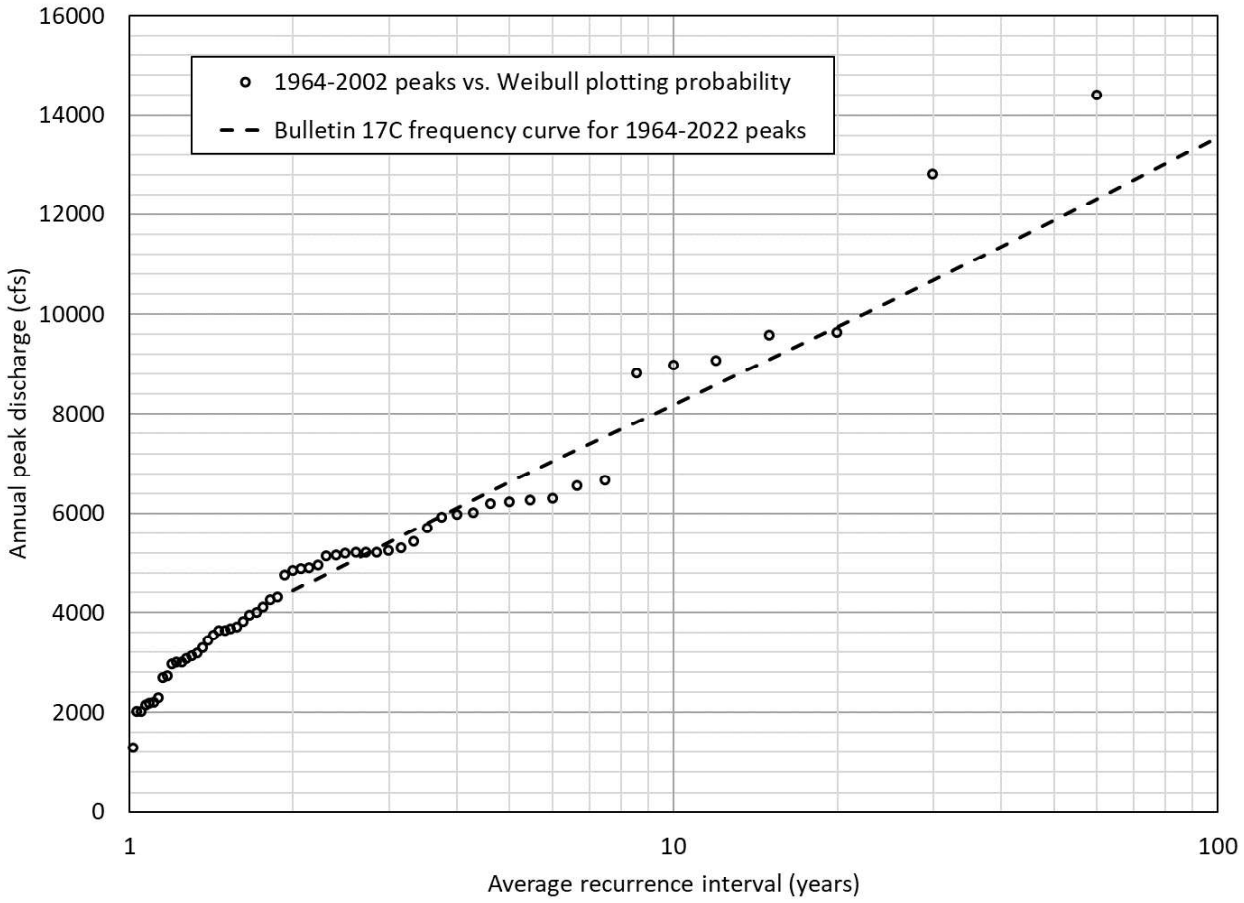


Figure 1. Discharge-frequency relationship for USGS station 6893300, Indian Creek at Overland Park (Marty Street), for period 1964-2022

.

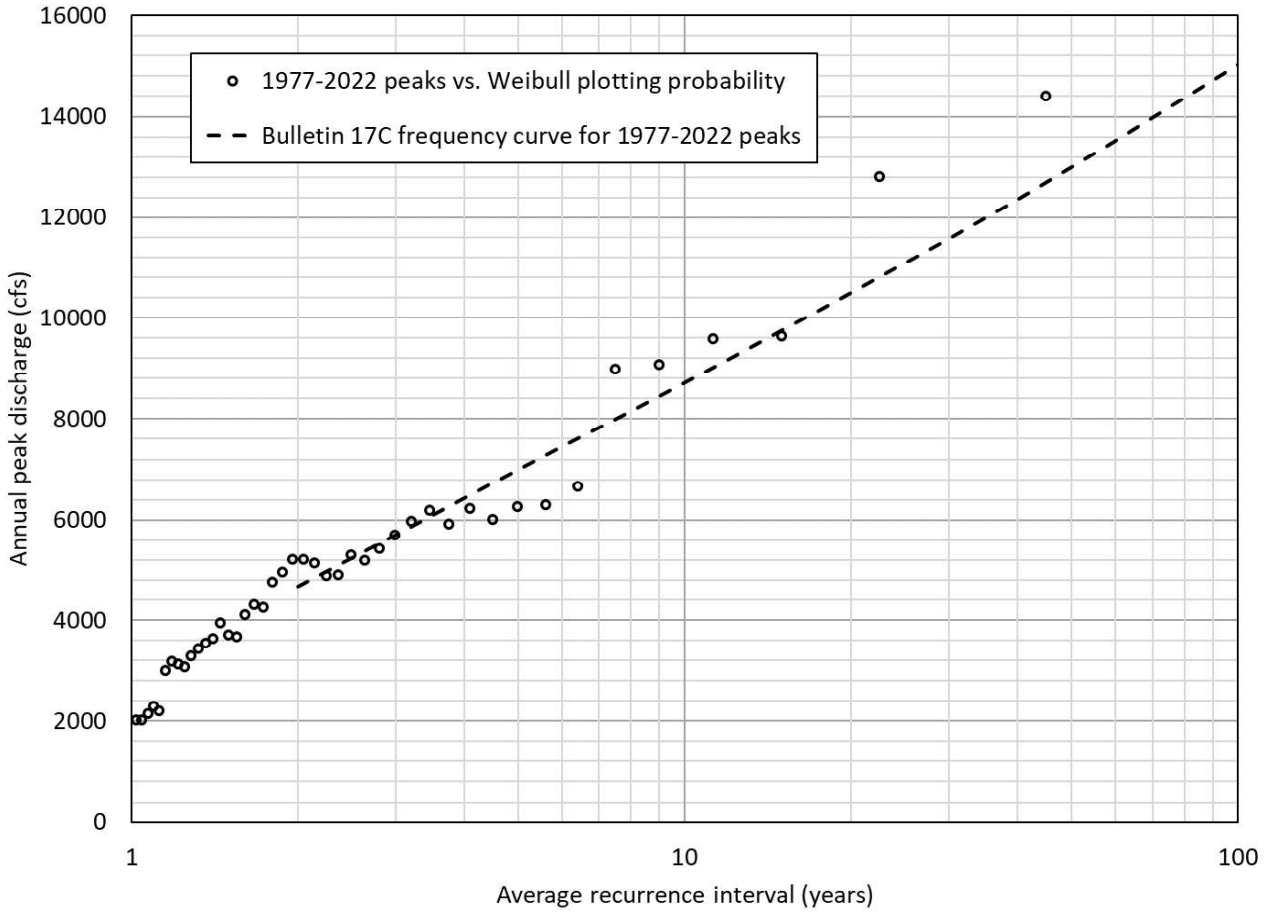


Figure 2. Discharge-frequency relationship for USGS station 6893300, Indian Creek at Overland Park (Marty Street), for period 1977-2022

Table 1. Discharge Frequency Estimates for Indian Creek at Marty Street

Annual exceedance probability	Discharge (cfs)	
	Period 1964-2022	Period 1977-2022
50%	4440	4469
20%	6631	6994
10%	8188	8711
5%	10260	11080
2%	11880	12990
1%	13560	15020

The quantity measured directly at USGS stream-gaging stations is not discharge but rather stage, defined as water-surface elevation above a fixed datum. A previously established rating curve/table (discharge-stage relationship) is used to estimate the discharge from the measured stage. The USGS makes field measurements of discharge by current-meter survey periodically, typically several times a year, at each station and uses those measurements to verify or adjust the rating curve.

Figure 3 shows the USGS's current rating curve for the gage on Indian Creek at Marty Street, with field measurements and major flood peaks superimposed. Most of the discharges measured in the field were under 100 cfs; only a few were much higher. The highest discharge measured directly was 4190 cfs on 3/04/2004. The stage-discharge data from the field measurements, particularly those from 2001 to present, plot in a fairly tight curve. The relationship does not appear to have shifted significantly over time. At flows above 4190 cfs, the rating curve has no hydraulic basis. It is simply an extrapolation of the equation shown on the graph, which was fitted to the field measurements for flows from 600 cfs to 4190 cfs. Discharge estimates for flood peaks with water-surface elevations above 869 feet (gage readings above 12 feet) may be somewhat in error due to the uncertainty of the rating curve. However, the reported water-surface elevations for those events are reliable.

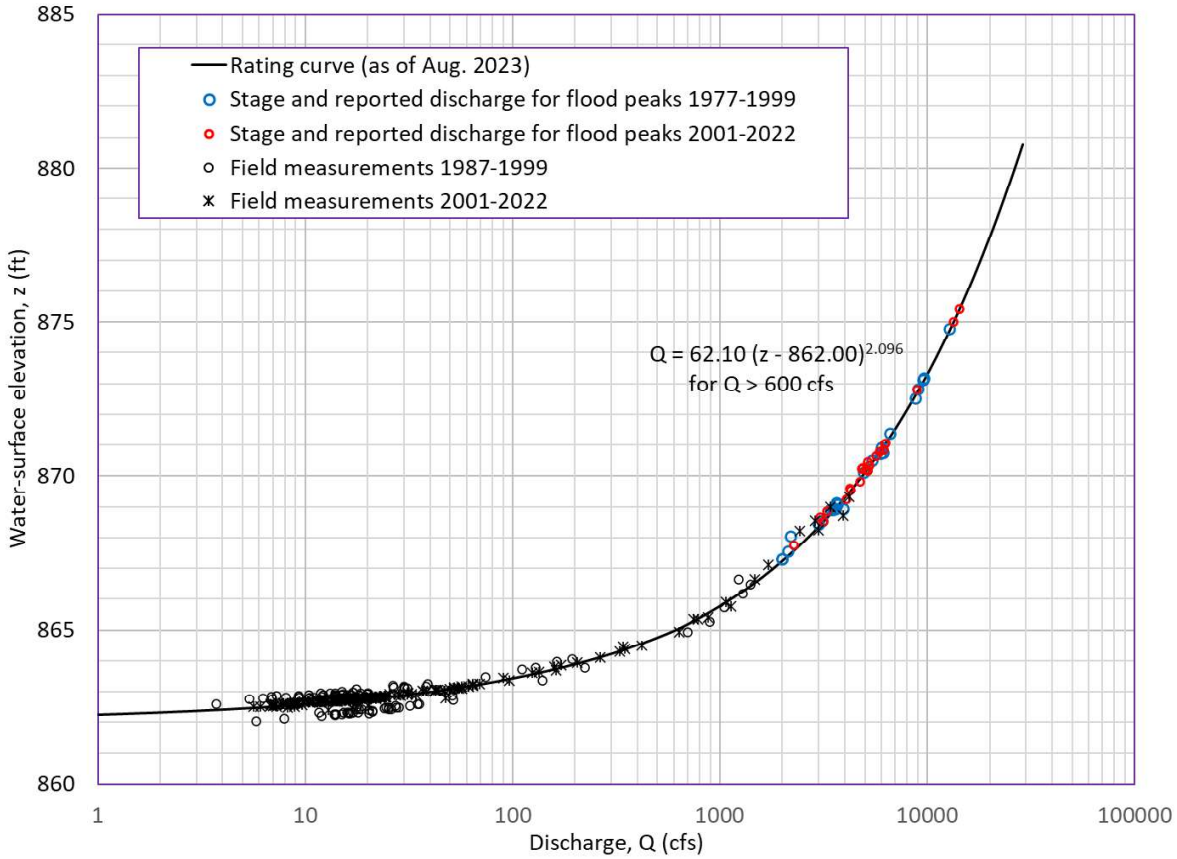


Figure 3. Rating curve for USGS station 6893300, Indian Creek at Overland Park (Marty Street)

Figure 4 shows the flood frequency relationship in Figure 2 re-plotted with discharges replaced by the corresponding water-surface elevations from the USGS rating curve. The 1%-chance discharge of 15,020 cfs corresponds to a 1%-chance stage of 875.70 feet.

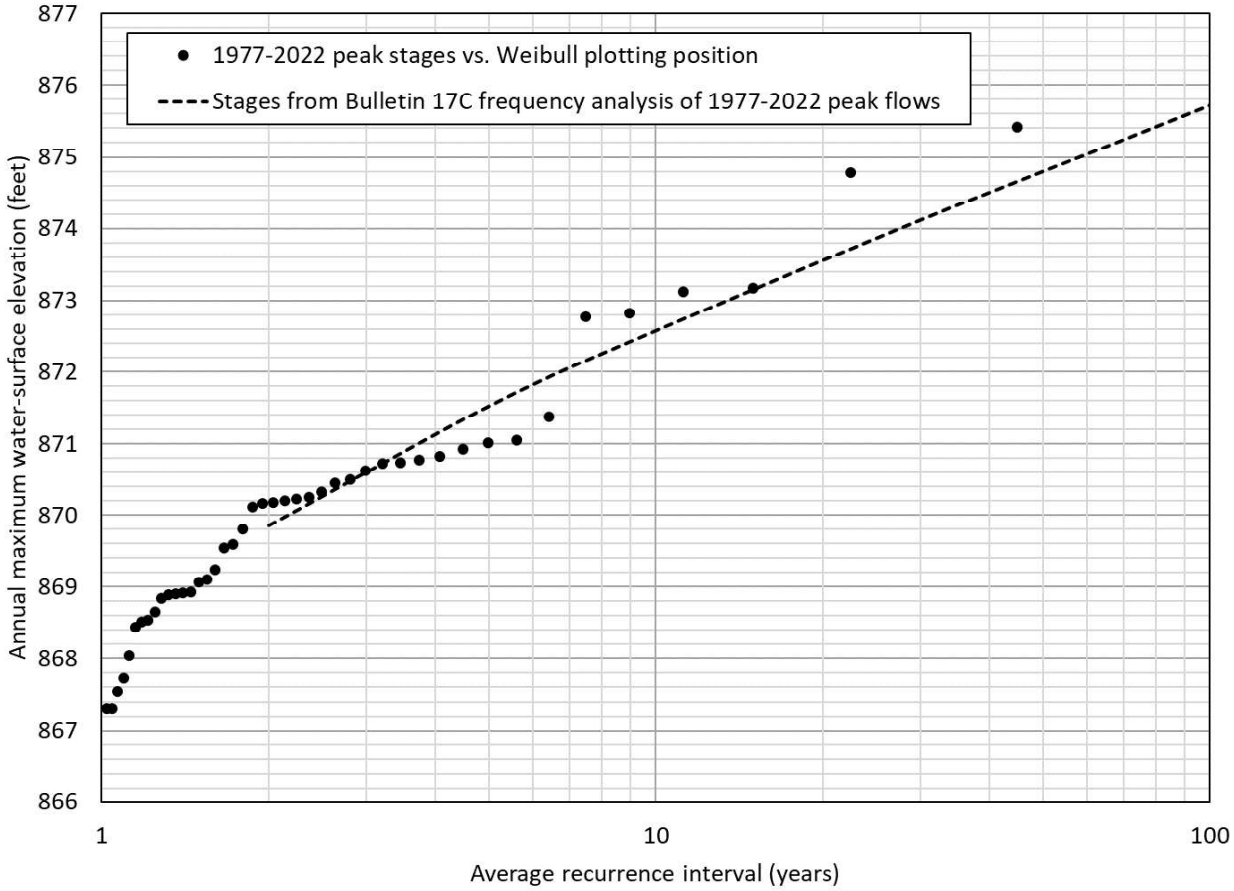


Figure 4. Stage-frequency relationship for USGS station 6893300, Indian Creek at Overland Park (Marty Street), for period 1977-2022

The flood of record on Indian Creek at Marty Street occurred on August 22, 2017. USGS records show a peak stage of 875.41 feet and a corresponding peak discharge of 14,400 cfs. ALERT gage IN09 at the same location recorded a peak stage of 875.66 feet. The discrepancy is probably due to the difference in the frequency of the measurements. USGS gages report stages at 15-minute intervals, while ALERT gages report much more frequently when conditions are changing rapidly.

3. Historic Storms and High Flows on Indian and Tomahawk Creeks

This examination of historic storms in the Indian and Tomahawk Creek watersheds focuses on gaged events that produced flows with estimated recurrence intervals of 2 years or greater. The figures in Appendix A show plots of rainfall intensities at 5-minute intervals at the Marty Street ALERT gage superimposed on USGS discharge hydrographs at the Marty Street and State Line gages for eighteen such events. Appendix B provides similar plots for three substantial floods at the Tomahawk Creek gage. The events in each appendix are ordered from highest to lowest peak flow. The hyetographs and hydrographs are plotted with identical discharge and intensity scales for ease of comparison. The table beside each graph shows the maximum rainfall depths and corresponding Atlas 14 recurrence intervals for durations from 1 to 24 hours.

Inspection of the hyetographs, hydrographs, and rainfall frequency data In Appendices A and B leads to the following general observations:

- High flows in Indian and Tomahawk Creeks have resulted from storms with a wide variety of durations and temporal patterns. No single duration or temporal pattern can be considered typical.

- Many of the storm events included multiple periods of high-intensity rainfall separated by periods of little or no rain.

- In none of the 22 events did rain fall continuously for a duration of 12 or more hours. The duration of continuous rainfall is shorter than 6 hours in most cases and shorter than 3 hours in many cases.

- The recurrence intervals of the maximum rainfall depths vary considerably with duration.

- The recurrence interval of the peak flow can exceed the recurrence intervals of the maximum rainfall amounts of all relevant durations if antecedent moisture conditions are unusually wet. The 10/04/1998 event provides one such example.

4. NOAA's Statistical Data on Rainfall Temporal Patterns

NOAA has developed statistical data on the temporal distributions of heavy rainfalls throughout the U.S. The methodology and the findings for the Midwestern States study area are summarized in Appendix A.5 of NOAA Atlas 14, Volume 8 (2013). NOAA's database consisted of hourly rainfall data for events with 6-hour, 12-hour, or 24-hour rainfall totals with recurrence intervals of 2 years and above. Separate statistics were developed for the four different regions of the Midwestern States study area shown in Figure 5. Johnson County lies within the Region

3, South Plains. NOAA's database for Region 3 included 8903 events of 6-hour duration, 9097 events of 12-hour duration, and 8635 events of 24-hour duration. It is important to note that the listed duration is the duration of the analysis period rather than the duration of continuous rainfall. NOAA's analysis periods always start with active rainfall, but in many cases, rainfall ceases before the analysis period ends.

NOAA further classified the events according to the quartile of the duration with the most rainfall. Table 2 shows, for each duration, the percentages of events with the heaviest rainfalls in the first, second, third, and fourth quartiles. First-quartile events are by far the most common for all three durations. One should note that NOAA defined the quartiles for the 6-hour duration unconventionally. The first quartile was defined as the first hour, the second quartile as the second and third hours, the third quartile as the fourth hour, and the fourth quartile as the fifth and six hours. If the quartiles had been defined conventionally with equal durations of 90 minutes, the percentage of first-quartile events would have been well over 50%.

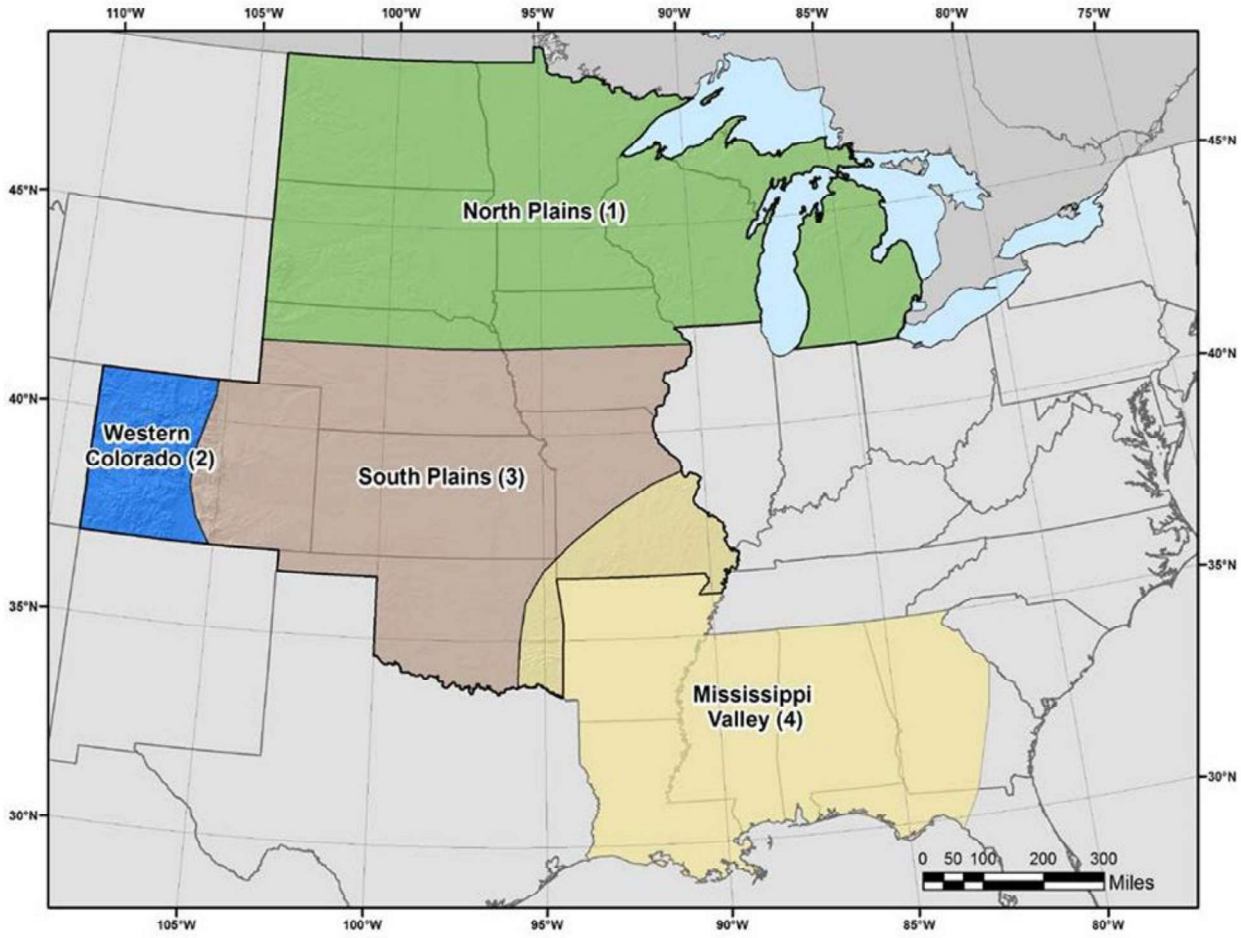


Figure 5. Four climate regions delineated in NOAA Atlas 14 Volume 8

Table 2. Distribution of rainfall events by quartile of duration with heaviest rainfall in NOAA's South Plains region

Duration (hours)	Percentage of events of stated duration with heaviest rainfall in stated quartile of duration			
	1st quartile	2nd quartile	3rd quartile	4thquartile
6	48%	29%	16%	7%
12	56%	22%	14%	8%
24	52%	18%	17%	13%

For each of the three observational durations (6, 12, and 24 hours), NOAA computed frequency distributions for the cumulative percentage of the total rainfall at half-hour intervals. Figures 6, 7, and 8 show these results graphically. The curve labels represent the exceedance probabilities of the cumulative percentages of total rainfall. On each graph, the solid 50% curve represents the median relationship. The plotted curves are composite statistical results for thousands of events rather than representative temporal patterns for individual events. Hyetographs of individual storms are generally quite irregular, but those irregularities disappear when the temporal patterns of many different storms are averaged together.

NOAA's statistical results for 6-hour, 12-hour, and 24-hour durations show that the rainfall temporal distributions for these durations are, on average, markedly front-loaded. In the median case for the 6-hour duration, 50% of rain falls in the first third of the duration and 73% falls in the first half. In the median case for the 12-hour duration, 58% of rain falls in the first third of the duration and 80% falls in the first half. In the median case for the 24-hour duration, 52% of the rain falls in the first third of the duration and 73% falls in the first half.

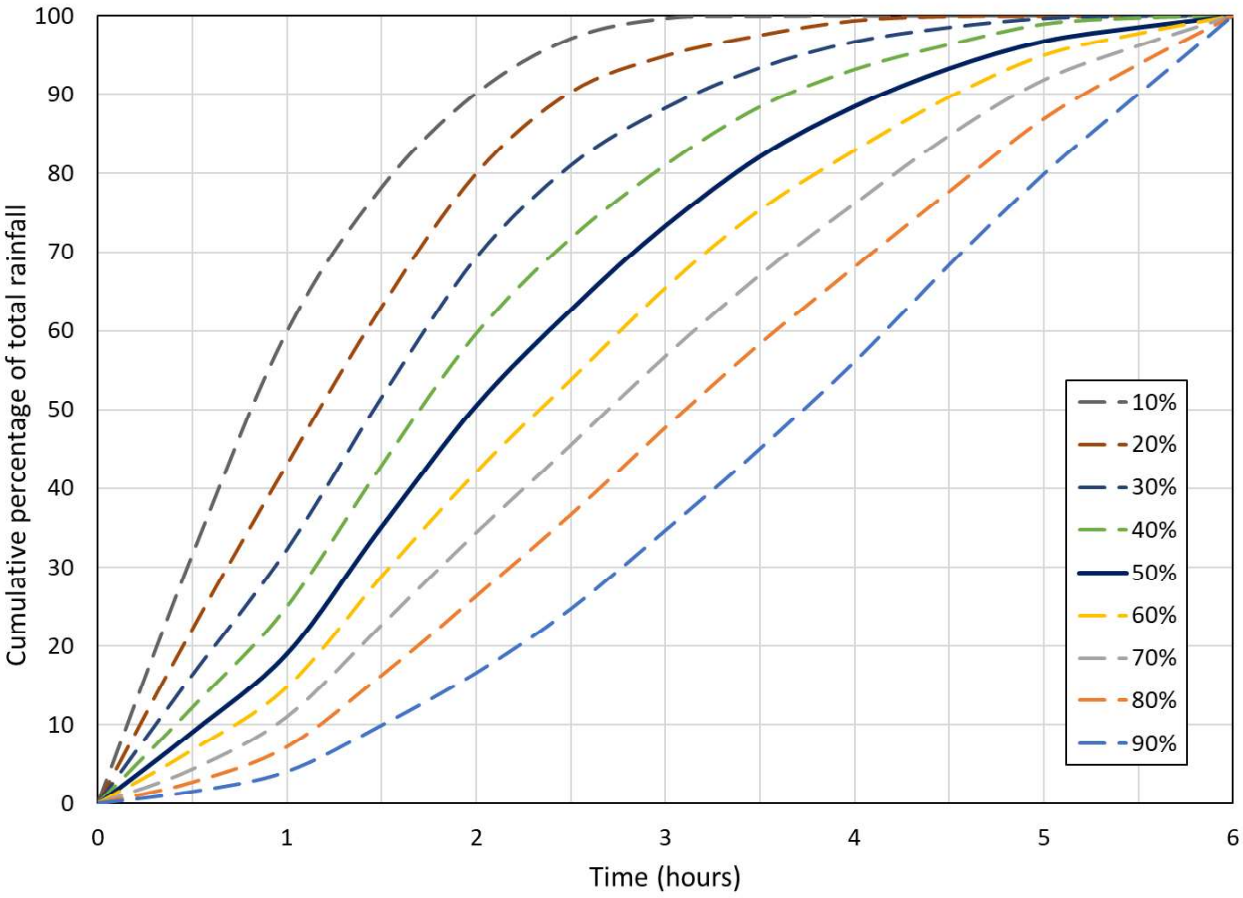


Figure 6. NOAA's frequency distribution for cumulative percentage of total rainfall versus time for 6-hour events in the South Plains region

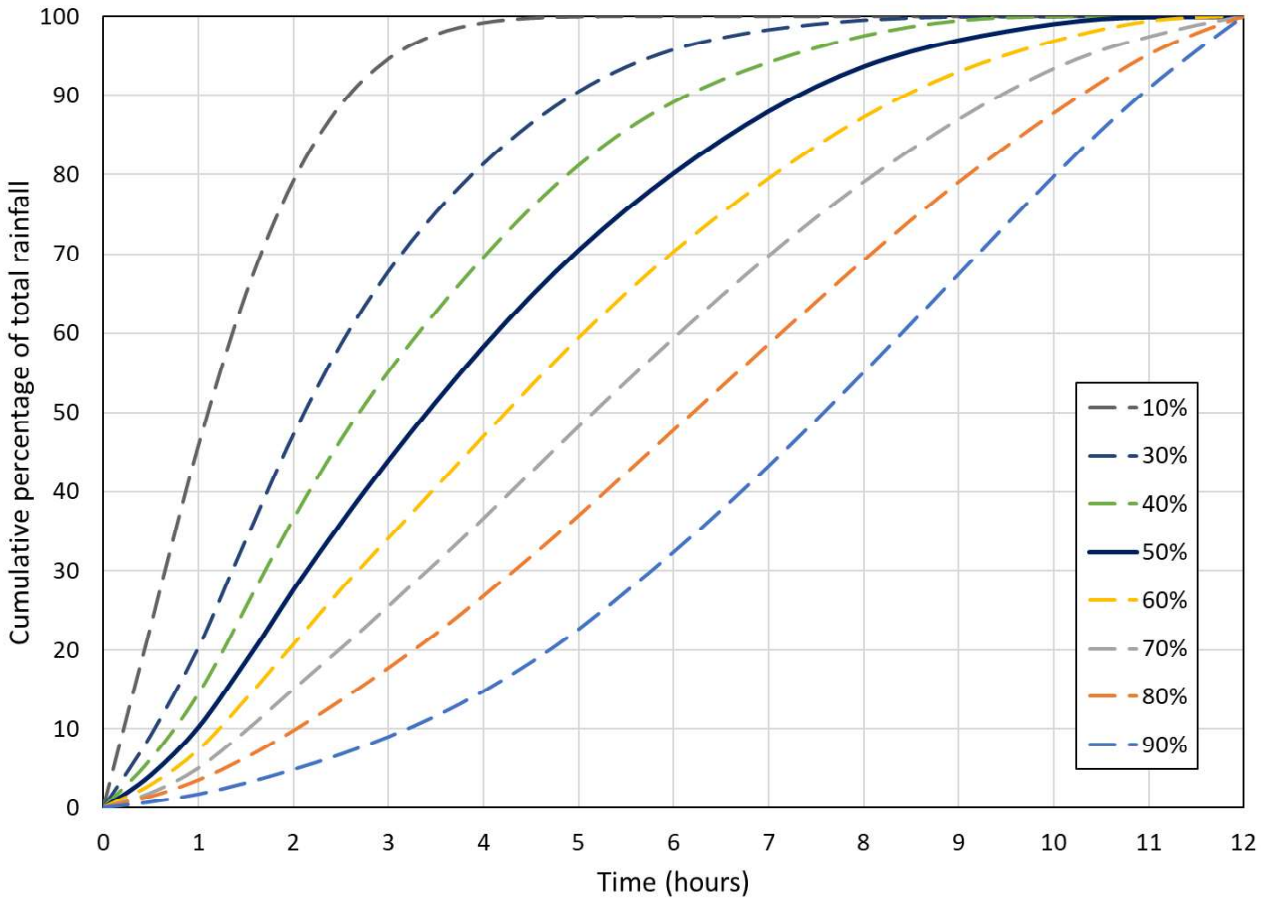


Figure 7. NOAA's frequency distribution for cumulative percentage of total rainfall versus time for 12-hour events in the South Plains region

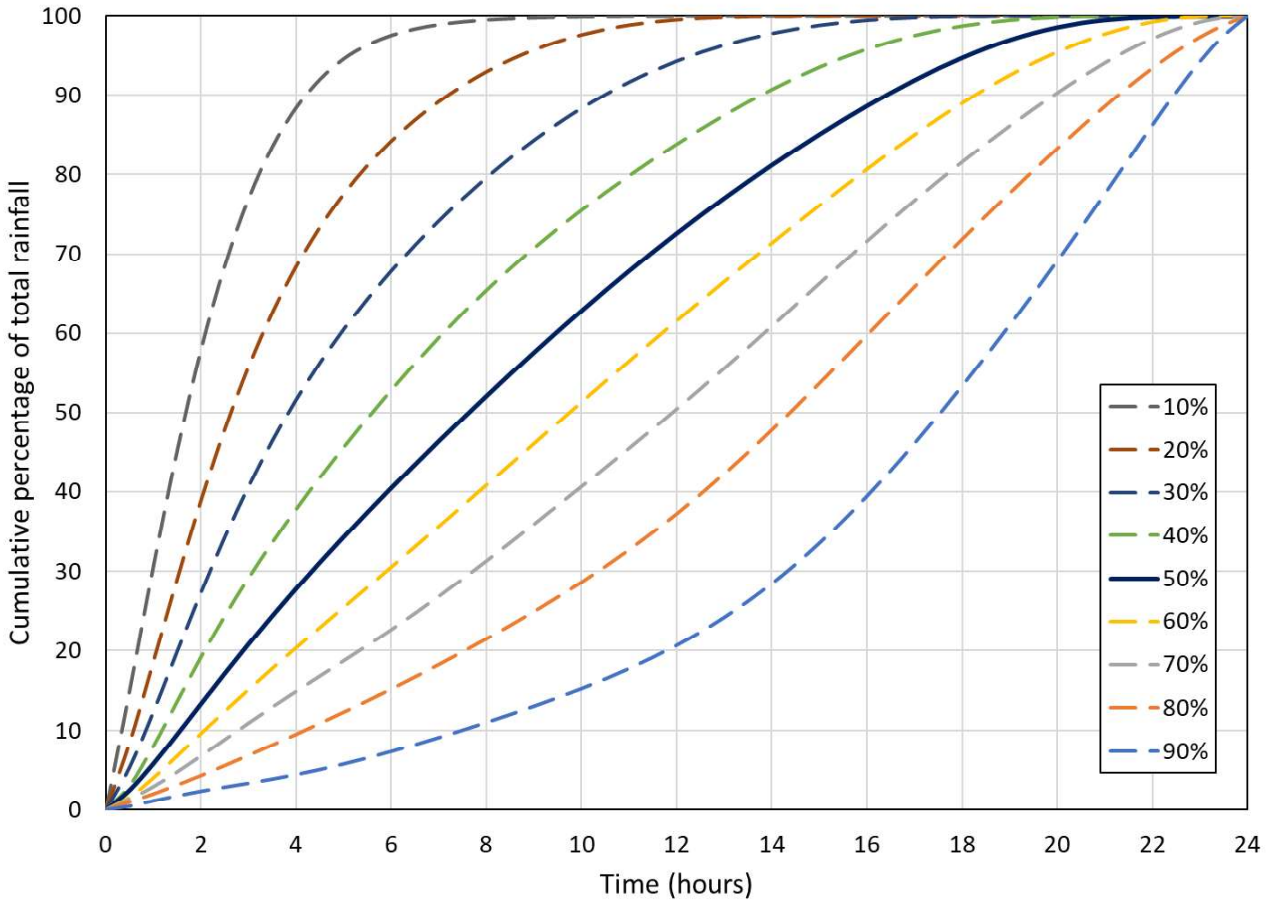


Figure 8. NOAA's frequency distribution for cumulative percentage of total rainfall versus time for 24-hour events in the South Plains region

5. Nested Synthetic Storms for Flood Studies

In flood studies, it is generally assumed that the discharge with a specified annual exceedance probability (AEP) at a given point along a stream is produced by rainfall with the same AEP over a duration equal to the hydrologic lag time or time of concentration at that point. That period of heavy rainfall might be embedded within a storm of longer duration, with rainfall distributed over the duration in a particular temporal pattern. An average antecedent moisture condition is generally assumed. The key variables are the overall duration of the synthetic storm and the temporal distribution of the rainfall within the storm.

Where a single flood model must account for flood conditions at multiple locations with widely differing times of concentration, a nested-type synthetic storm is most appropriate. In a nested storm, the rainfall is distributed over the storm duration so that the maximum rainfall depth of any shorter duration has the same AEP as the total storm rainfall. For example, within a nested storm of 24-hour duration, the maximum rainfall depths over durations of 1, 2, 3, 6, and 12

hours would all have the same AEP as the 24-hour rainfall depth. The key variables for a nested synthetic storm are the total duration of the storm and the time to the peak intensity.

Appendix C presents a step-by-step procedure for developing a nested hyetograph with desired values of rainfall frequency, duration, time to peak intensity, and rainfall time interval.

The HEC-HMS (Hydrologic Modeling System) program of the U.S. Army Corps of Engineers includes a nested storm option, termed the "frequency storm." The duration of the frequency storm can be set to 1, 2, 3, 6, 12, or 24 hours (or longer), and the peak intensity can be set to occur at 25%, 33%, 50%, 67%, or 75% of the storm duration. The rainfall inputs for an n%-chance frequency storm of 24-hour duration are the n%-chance rainfall depths for durations of 5, 10, 15, and 30 minutes and 1, 2, 3, 6, 12, and 24 hours.

The most common synthetic storm model for flood studies is a 24-hour nested storm with a 12-hour time-to-peak. However, in many cases this storm model may not be the most appropriate one. In our region, the periods of intense rainfalls that produce flooding on small urban watersheds typically occur within convective storms with shorter durations than the large-scale frontal events that produce extreme 24-hour rainfalls. A synthetic storm model that combines extreme rainfalls of all durations out to 24 hours could be overly conservative. The placement of the peak intensity at the midpoint of the storm duration might also be inappropriate. NOAA's statistical data show that temporal distributions of heavy rainfalls in our region tend to be strongly front-loaded. This finding is important because front-loaded storms produce lower peak flows than back-loaded storms.

The effects of a nested storm's duration and time-to-peak on computed peak flow was investigated with a simple HEC-HMS model of the Indian Creek watershed. The model was calibrated to match the peak flow, time-to-peak, and runoff volume for the 8/22/17 flood event at the Marty Street gage, with the rainfall record from that location as the precipitation input. The calibrated model was run with 1%-chance nested storms of 24-hour and 12-hour duration with peak intensities at 50% and 25% of those durations. Table 3 shows the computed peak flows and their relative differences from the result for the 24-hour storm with the 12-hour time-to-peak.

Table 3. Preliminary test of sensitivity of peak flow to nested storm characteristics

1%-AEP nested storm		Indian Creek at Marty Street	
Duration (hours)	Time-to-peak (hours)	Peak flow (cfs)	Change from baseline*
24	12	15,701	-----
24	6	14,663	-6.6%
12	6	14,663	-6.6%
12	3	13,732	-12.5%

*Baseline storm: 24-hour duration, 12-hour time-to-peak

These results show that the nested storm's time-to-peak has a substantial impact on the peak flow. Shortening the 24-hour storm's time-to-peak from 12 to 6 hours reduces the peak discharge by more than 6%. Shortening the 12-hour storm's time-to-peak from 6 to 3 hours also reduces the peak discharge by more than 6%. However, the 24-hour and 12-hour storms with 6-hour times-to peak yield identical peak flows, which shows that the amount of rain that precedes the peak matters much more than the amount of rain that falls after the peak.

6. Temporal Patterns Derived from NOAA's Statistical Distributions

Some flood modelers distribute synthetic storm rainfall in temporal patterns derived from NOAA's statistical distributions. That approach is evaluated here by examining five 1%-chance, hyetographs for Indian Creek derived from NOAA's statistical tables for the 24-hour duration. Those tables provide values of cumulative percentage of total rainfall versus time at 30-minute intervals for exceedance probabilities from 10% to 90%. The five hyetographs, plotted in Figure 9, were developed from the cumulative rainfall percentages exceeded by 10% of all events, 50% of all events, 90% of all events, 10% of first-quartile events, and 90% of fourth-quartile events in the South Plains region. Those five hyetographs span the range of NOAA's statistical findings for the 24-hour duration. At the extremes, the 10% first-quartile temporal pattern is more front-loaded than 95% of all events in NOAA's dataset, and the 90% fourth-quartile pattern is more back-loaded than 99% of all events in the dataset. The other three patterns cover the middle ground. Three of the five hyetographs have effective durations much shorter than 24 hours. The other two hyetographs have very low intensities at all times.

Table 4 shows the maximum depths of rainfall for durations of 30 minutes and 1, 2, 3, 6, and 12 hours for the five hyetographs. Included and highlighted for comparison are the maximum depths for the 24-hour nested storm, which are the 1%-chance depths from Atlas 14. Table 5 shows the recurrence intervals of the maximum depths in Table 4. The five hyetographs derived from NOAA's statistical data all contain maximum rainfall depths for durations shorter than 24 hours that differ greatly from the 1%-chance depths from Atlas 14. The maximum 30-minute depths are much too low in all five cases. For the 50% and 90% all-event hyetographs, the maximum depths are much too low for all durations up to 12 hours. On the other hand, for the 10% all-event and 10% first-quartile hyetographs, the maximum depths for the intermediate durations have recurrence intervals far in excess of 100 years.

These findings indicate that none of the NOAA-based temporal patterns would work well in a flood model that must provide reasonable estimates of flood levels with a consistent recurrence interval at multiple locations with widely differing times of concentration. When the 24-hour rainfall is distributed in any of the five patterns, the maximum rainfalls depths for durations of 30 minutes and 1, 2, 3, 6, and 12 hours have recurrence intervals that differ greatly from the recurrence interval of the 24-hour rainfall.

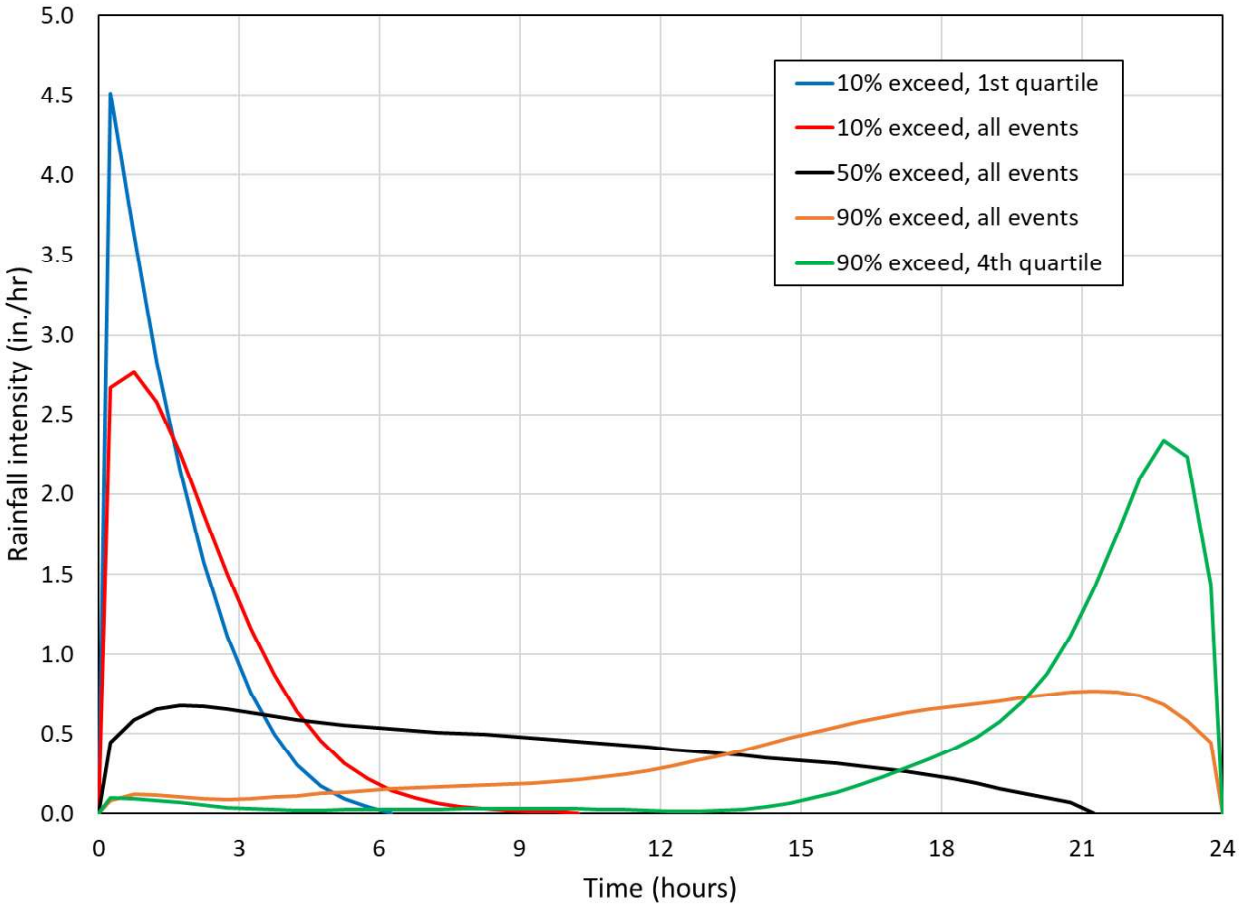


Figure 9. 1%, 24-hour hyetographs for Indian Creek with five NOAA temporal distributions

Table 4. 1%-chance 24-hour synthetic storms for Indian Creek: maximum depth vs. duration

Temporal distribution for 1%-AEP 24-hr storm	Maximum rainfall depth of stated duration (inches)						
	30 min	1	2 hr	3 hr	6 hr	12 hr	24 hr
Nested	**2.70**	**3.65**	**4.60**	**5.30**	**6.56**	**7.82**	**8.86**
NOAA 10%, 1st quartile	2.26	4.07	6.56	7.90	8.83	8.85	8.86
NOAA 10%, all events	1.38	2.72	5.14	6.82	8.64	8.86	8.86
NOAA 50%, all events	0.34	0.67	1.33	1.95	3.63	6.43	8.86
NOAA 90%, all events	0.38	0.77	1.52	2.25	4.27	7.03	8.86
NOAA 90%, 4th quartile	1.17	2.28	4.21	5.63	7.70	8.42	8.86

Table 5. 1%-chance, 24-hour synthetic storms for Indian Creek: recurrence intervals of maximum depths

Temporal distribution for 1%-AEP 24-hr storm	Recurrence interval of maximum depth of stated duration (years)						
	30 min	1	2 hr	3 hr	6 hr	12 hr	24 hr
Nested	**100**	**100**	**100**	**100**	**100**	**100**	**100**
NOAA 10%, 1st quartile	24	192	927	1158	568	204	100
NOAA 10%, all events	3	21	189	439	497	205	100
NOAA 50%, all events	<2	<2	<2	<2	7	37	100
NOAA 90%, all events	<2	<2	<2	3	13	57	100
NOAA 90%, 4th quartile	2	10	61	139	246	152	100

7. Areal Reduction of Rainfall

The rainfall depths in NOAA Atlas 14 are point values that apply to specific locations. The n%-chance value of the spatially averaged rainfall over a watershed is always less than the n%-chance point rainfall. The ratio of the spatially averaged depth to the point depth for the same duration and frequency is termed the areal reduction factor (ARF). ARF values vary with the watershed size and the specified duration. The relevant rainfall duration is generally the time of concentration or lag time to the point of interest. The larger the watershed and the shorter the duration, the smaller the ARF value. Table 6 shows ARF values obtained from equations fitted to the widely accepted curves published by the U.S. Weather Bureau in 1958 (McEnroe and Young, 2007).

Table 6. Areal reduction factors for rainfall

Area (mi^2)	Areal reduction factors for rainfall of specified duration							
	5 min	15 min	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr
1	0.985	0.990	0.995	0.996	0.997	0.998	0.998	0.999
2	0.970	0.981	0.990	0.992	0.993	0.995	0.996	0.997
5	0.926	0.954	0.974	0.981	0.984	0.988	0.991	0.993
10	0.857	0.910	0.951	0.963	0.969	0.977	0.983	0.987
20	0.733	0.833	0.908	0.932	0.943	0.957	0.968	0.976
30	0.627	0.767	0.871	0.904	0.920	0.940	0.956	0.967
40	0.536	0.710	0.840	0.881	0.900	0.926	0.945	0.959
50	0.457	0.661	0.813	0.861	0.883	0.913	0.935	0.952
60	0.390	0.619	0.789	0.843	0.868	0.902	0.927	0.946
70	0.332	0.582	0.769	0.828	0.856	0.893	0.920	0.941

When the objective of a hydrologic calculation or model is the n%-chance discharge or stage at a single location, the appropriate ARF values can be applied to the n%-chance rainfall depths of the different durations. However, in a watershed study where n%-chance conditions are needed at multiple points with different times of concentration, no single set of ARF values will suffice. Multiple model runs with different ARF values would be needed to determine the n%-chance flood levels at multiple locations with different drainage areas. The HEC-HMS hydrologic modeling system incorporates a "depth-area analysis" feature that makes multiple model runs with ARF values for different drainage areas and then estimates the n%-chance conditions at the various locations by interpolation. However, HEC-RAS 2D does not have that capability. Making multiple model runs with ARF values applied to the point rainfall then interpolating the results is infeasible in HEC-RAS 2D due to the excessive runtimes that would be needed.

Some flood modelers have attempted to account for the areal reduction of rainfall by trimming off the leading and trailing tails of the nested hyetograph. In this scheme, the total storm depth is reduced to the desired value by shortening the storm duration without altering the rainfall intensities within the remainder of the storm. Tests performed as part of this investigation indicate that this scheme does not achieve the desired objective. Trimming the hyetograph's tails reduces peak flows by similar percentages at points with widely differing drainage areas fact. This result is unrealistic. The impact of the areal reduction of rainfall on peak flows should be negligible for small drainage areas and increase in the downstream direction.

In fact, there is no way to properly account for the areal reduction of rainfall at different locations in a HEC-RAS 2D flood model without making multiple model runs. However, neglecting the areal reduction of rainfall in the Indian Creek watershed study will not affect the results much. At State Line Road, Indian Creek has a drainage area of 64.2 square miles and a time of concentration on the order of 6 hours. The ARF value for that drainage area and time of concentration is 0.90. At its confluence with Tomahawk Creek, 0.8 miles upstream of State Line Road, Indian Creek's drainage area is 38 square miles and the applicable ARF value is 0.93. At most other locations of interest in the two watersheds, drainage areas are much smaller and ARF values for relevant rainfall durations are closer to 1.0.

8. Selection of Nested Storm for Indian and Tomahawk Creek Flood Studies

The peak discharge produced by an n%-chance nested storm depends on the storm's duration and time-to-peak. The objective is to find a combination of duration and time-to-peak that produces a peak discharge with the same recurrence interval as the storm rainfall. This investigation focuses on the Indian and Tomahawk Creek watersheds, but the findings and recommendations might also be applied to flood models of other Johnson County watersheds, so the characteristics of those watersheds should be considered.

FEMA's guidance states that the duration of the synthetic storm must exceed the longest time of concentration in the watershed. The times of concentration of the Indian and Tomahawk

Creek watersheds are no longer than 6 hours, but the Cedar Creek and Kill Creek watersheds in western Johnson County are larger and less developed and therefore have longer times of concentration. Cedar Creek and Kill Creek each have a USGS discharge gage located near the lower end of the watershed and several ALERT rainfall gages located in and around the watershed. The lag times of Cedar Creek and Kill Creek were estimated by calibrating a simple HEC-HMS model of each watershed with ALERT rainfall data and USGS streamflow data for the event of 7/27/2017. Best-fit values of runoff curve number and lag time were computed with the HEC-HMS parameter optimization feature. Time of concentration was assumed to equal five-thirds of lag time. Cedar Creek at 83rd Street (USGS station 682495) was found to have a lag time of 5.9 hours and a corresponding time of concentration of 9.9 hours. Kill Creek at 95th Street (USGS station 682360) was found to have a lag time of 6.0 hours and a time of concentration of 10.0 hours. These findings indicate that a synthetic storm template to be applied throughout Johnson County should have a duration of 10 hours or longer.

The HEC-RAS 2D flood models for Indian and Tomahawk Creeks were calibrated for two major historical flood events at the three USGS gages. As a separate task, further calibration was recently completed for other major events, including the 8/22/2017 flood of record, using data from the ALERT gage network. For most events, at most stage-gage sites, the computed peak stages are reasonably close to the recorded values. However, for a reason yet undetermined, the computed peak stages for Indian Creek at Marty Street are somewhat higher than the gaged values for those events. For the 8/22/2017 event, the computed peak stage at that location is 876.99 feet, 1.33 feet higher than the peak stage recorded at the ALERT gage.

The three 1%-chance nested storms shown in Figure 10 were tested in the 2D model for Indian Creek. Table 7 shows the computed peak stages. The three values are all significantly higher than the 1%-chance stage of 875.70 feet from the frequency analysis of the 1977-2022 gage data. However, the current HEC-RAS 2D model overestimates the peak stage for the 8/22/2017 flood of record by 1.33 feet at that location. The 2D model might be expected to overestimate the peak stage for the 1%-chance event by a similar amount for reasons unrelated to the synthetic storm. Of the three nested storm configurations, the 12-hour storm with the 3-hour time-to-peak yields the 1%-chance flood level closest to the value from the frequency analysis of the USGS gage data.

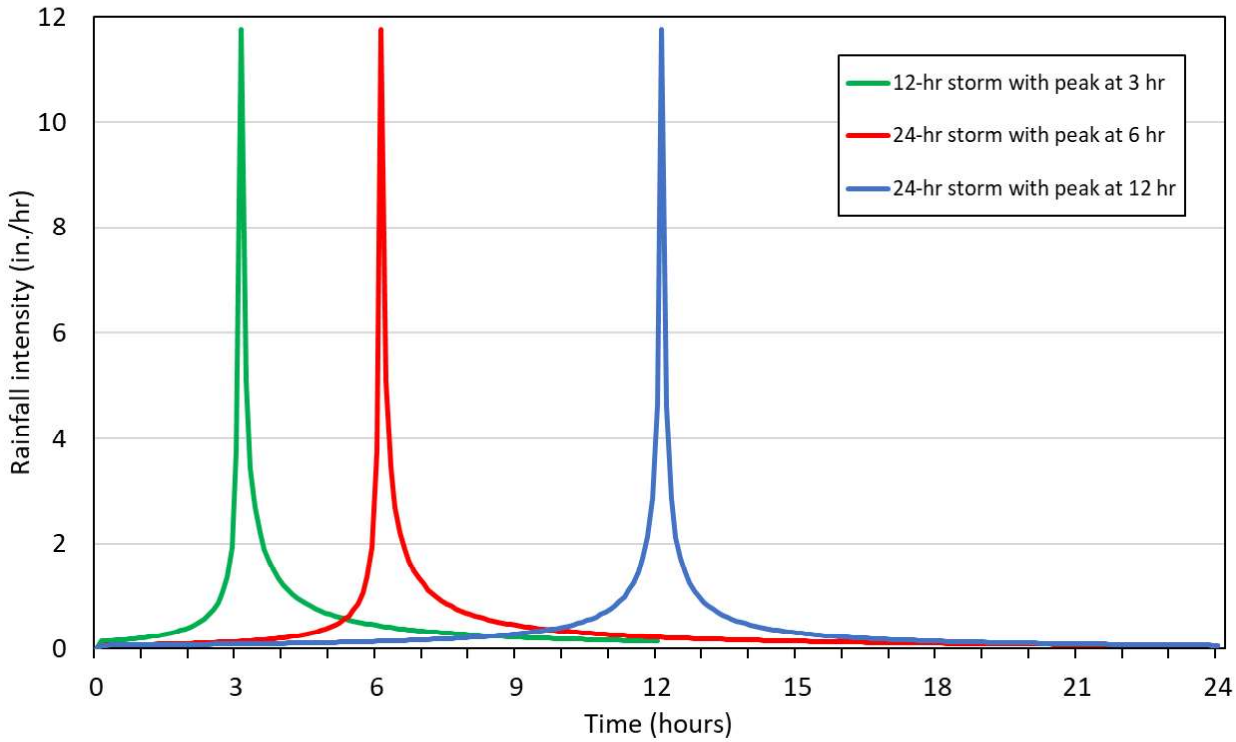


Figure 10. 1%-chance nested storms with different durations and times-to-peak

Table 7. HEC-RAS 2D peak stages in Indian Creek at the Marty Street gage for three 1%-chance nested storms

Storm duration (hours)	Storm time-to-peak (hours)	Peak stage (feet)
24	12	878.11
24	6	877.51
12	3	877.34

9. Recommended Synthetic Storm

The recommended storm configuration for the Indian and Tomahawk Creek flood models is a nested storm of 12-hour duration with a 3-hour time-to-peak. A nested storm is the best option for computation of flood levels with a consistent recurrence interval at locations with widely varied times of concentration in a single model run. Of the three configurations tested in the 2D model, the 12-hour with the 3-hour time-to-peak yielded the most realistic value for

the 1%-chance stage at the gage site with the longest record. The hyetographs from the high-flow events from 1993 to present in Appendices A and B provide support for a nested storm duration of 12 hours rather than 24 hours. In none of those historical events did rain fall continuously for more than 12 hours. NOAA's statistical findings for heavy 12-hour rainfalls provide support for the recommended 3-hour time-to-peak. In NOAA's median temporal distribution for the 12-hour duration, 50% of the total rainfall occurs in the first 3.4 hours. The 12-hour nested storm with a 3-hour time-to-peak matches that characteristic exactly.

Figure 10 compares the hyetographs of the recommended 1%-AEP synthetic storm (nested, 12-hour duration, 3-hour time-to-peak) and the 8/22/2017 flood-of-record event at the Marty Street gage. The start of the shorter 8/22/2017 event is lagged to align the timing of the peak intensities. This comparison highlights the difference between a synthetic rainfall event with a single peak and a real rainfall event with multiple peaks and a more random-appearing temporal pattern.

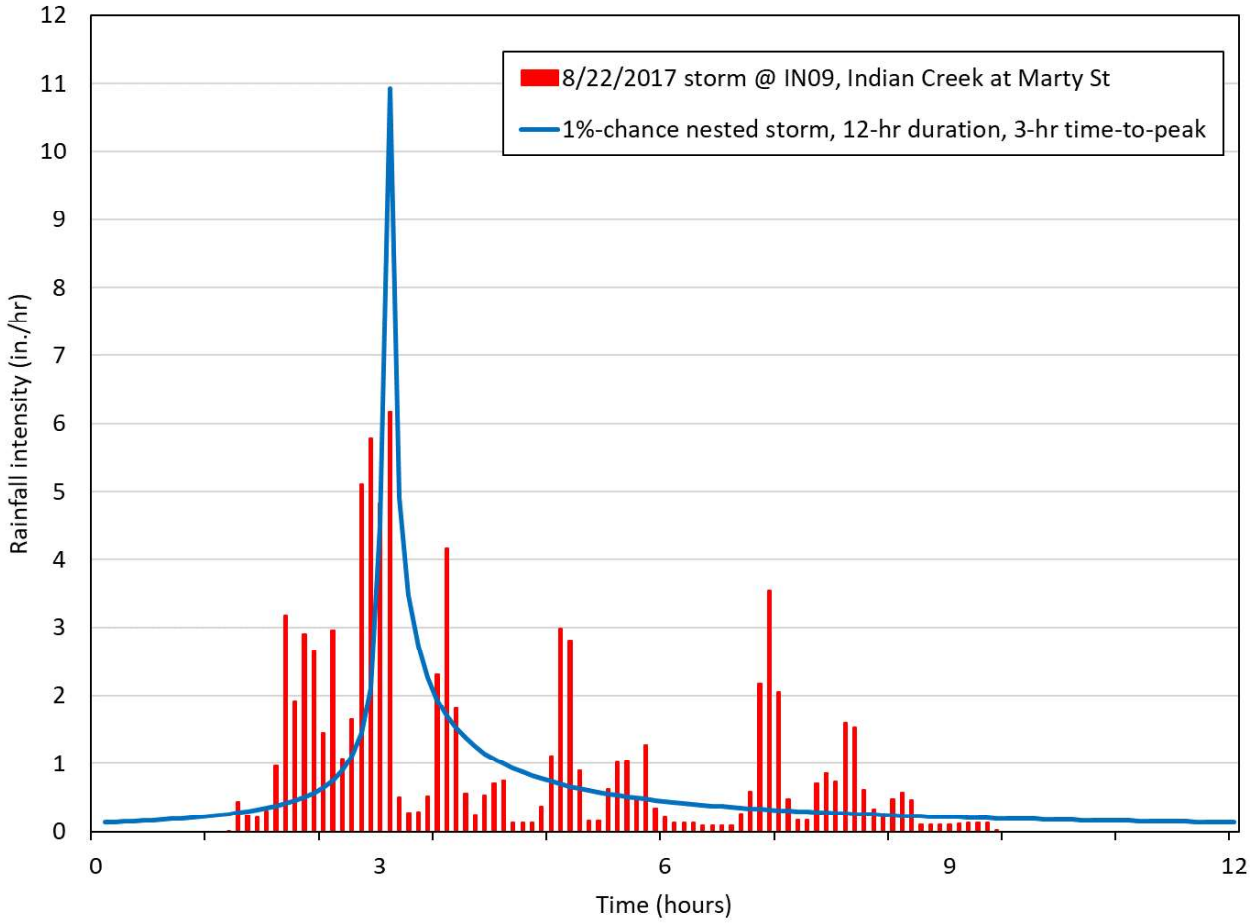


Figure 10. Comparison of recommended 1%-chance nested storm and rainfall on 8/22/2017

No areal reduction factor should be applied to the storm rainfall. There is no practical way to account for the areal reduction of rainfall at different locations in a single HEC-RAS 2D model run. Neglecting the areal reduction of rainfall in the Indian and Tomahawk Creek flood models will have an insignificant impact on the computed peak flows.

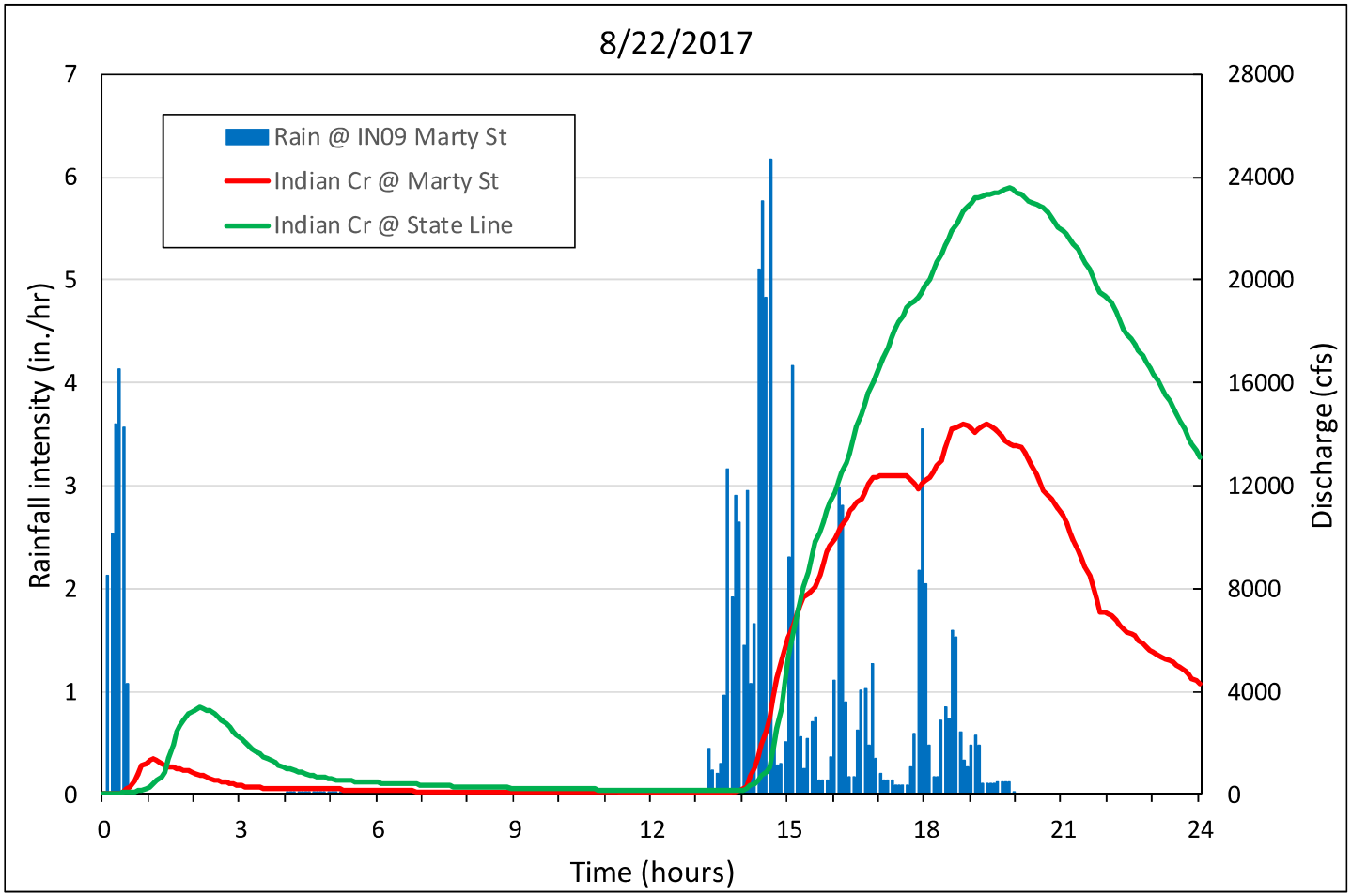
References

Federal Emergency Management Agency (2019). *Hydrology: Rainfall-Runoff Analysis, Guidance for Flood Risk Analysis and Mapping*

McEnroe, B.M. and C. B. Young (2007). *Flood Frequency Relationships for Small Watersheds in Kansas*, Report No. K-TRAN: KU-06-4, Kansas Department of Transportation

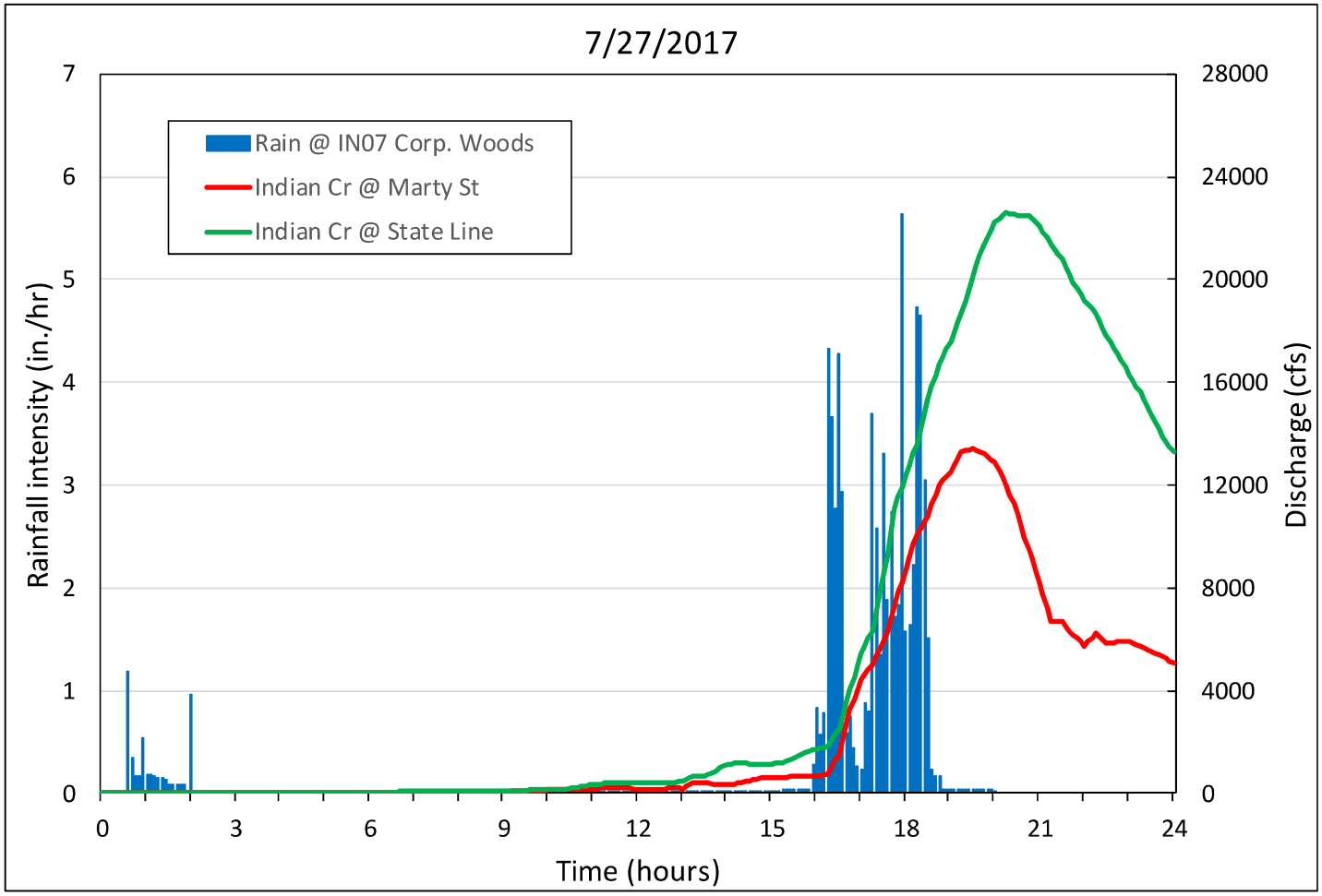
National Atmospheric and Oceanic Administration (2013) Precipitation Frequency Atlas of the United States, Volume 8, Version 2.0: Midwestern States

Appendix A – Significant Events on Indian Creek



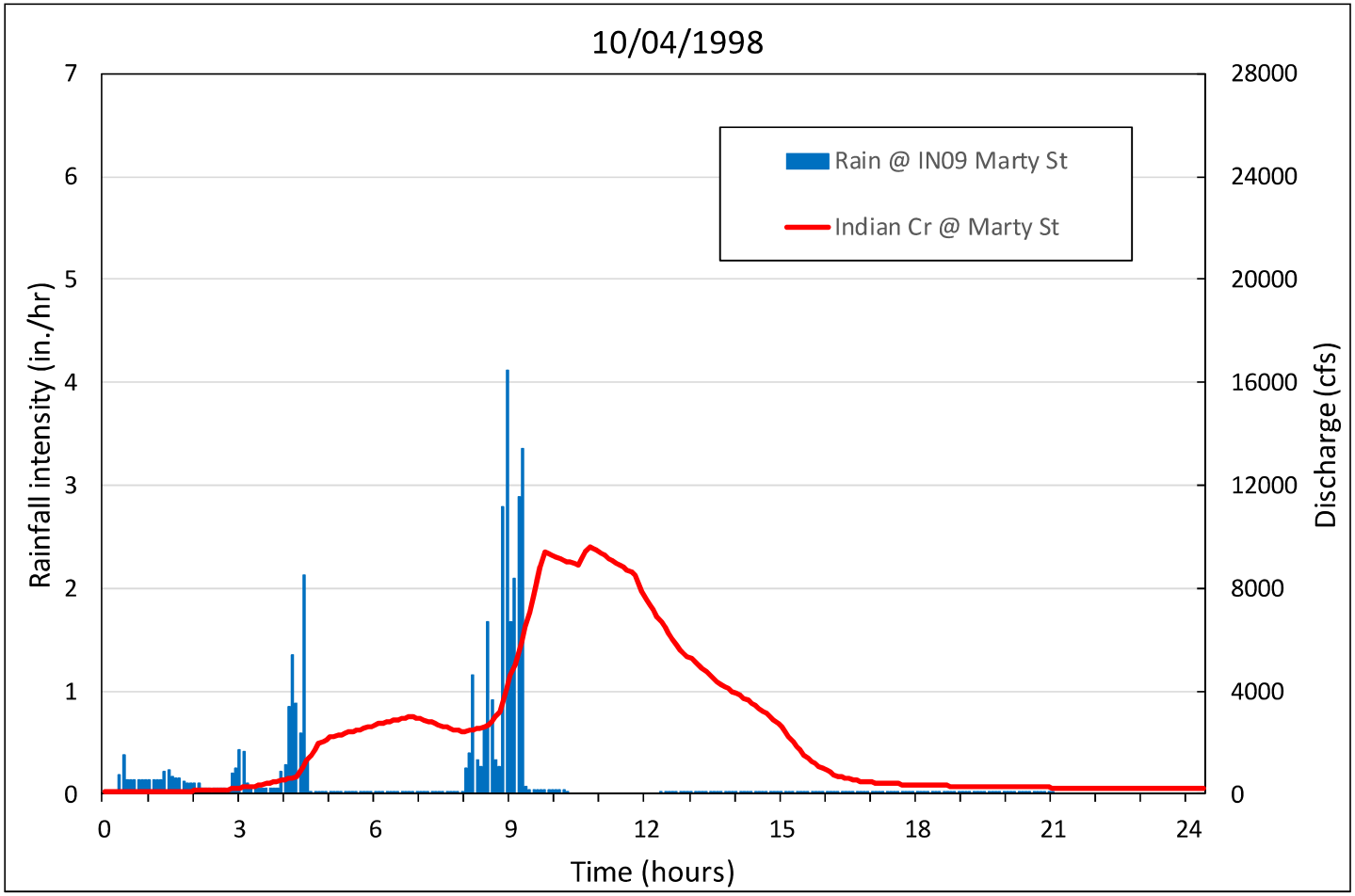
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	8.75	93
12	7.31	70
6	7.21	168
3	5.23	93
2	4.37	75
1	3.30	56

Figure A-1. Event of 8/22/2017



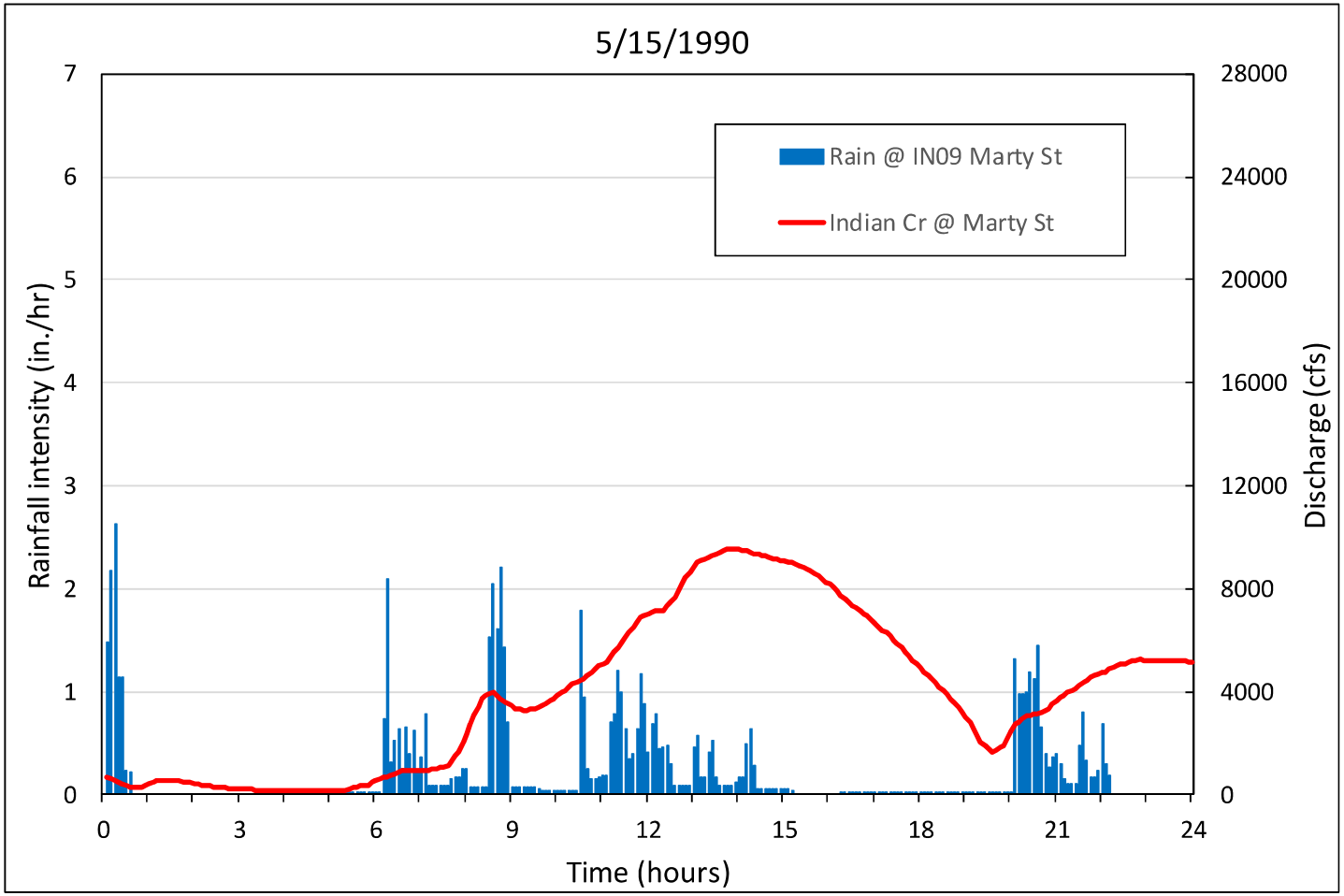
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	6.44	20
12	5.94	25
6	5.85	55
3	5.77	159
2	4.49	87
1	2.92	30

Figure A-2. Event of 7/27/2017



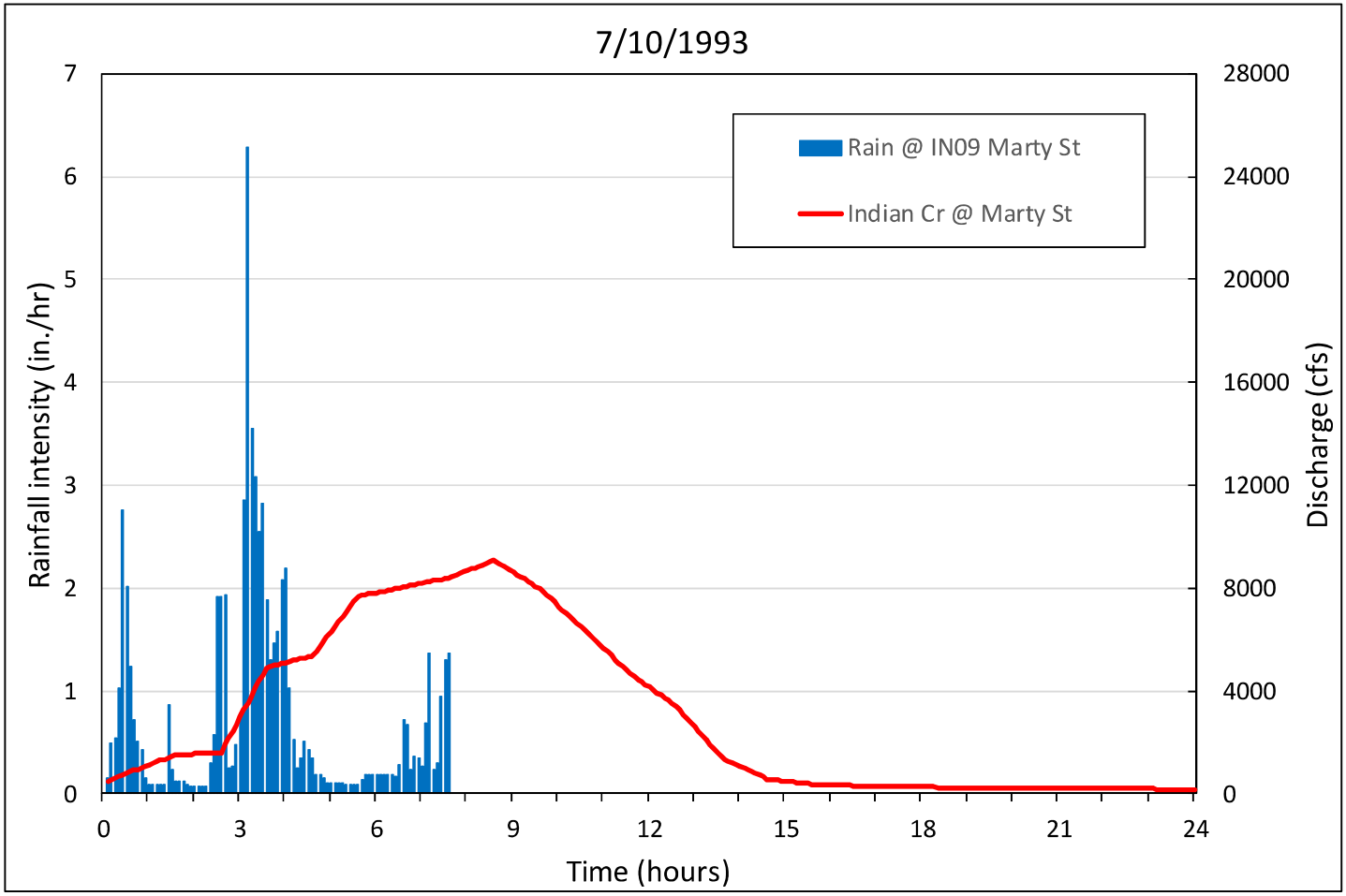
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.07	< 2
12	3.03	2
6	2.55	2
3	1.98	< 2
2	1.96	3
1	1.75	4

Figure A-3. Event of 10/04/1998



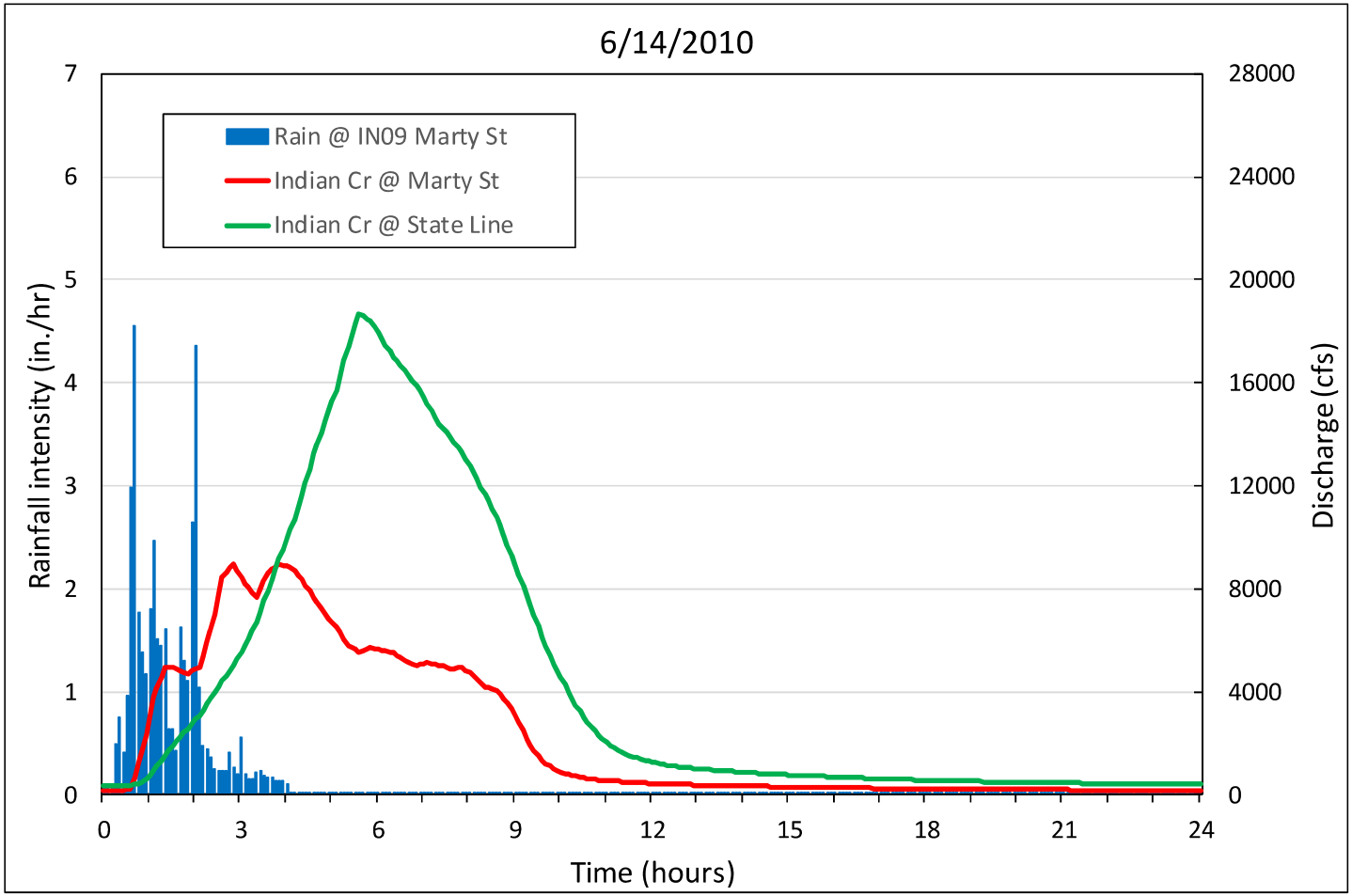
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	5.39	9
12	3.44	3
6	2.71	3
3	1.58	< 2
2	1.24	< 2
1	0.84	< 2

Figure A-4. Event of 5/15/1990



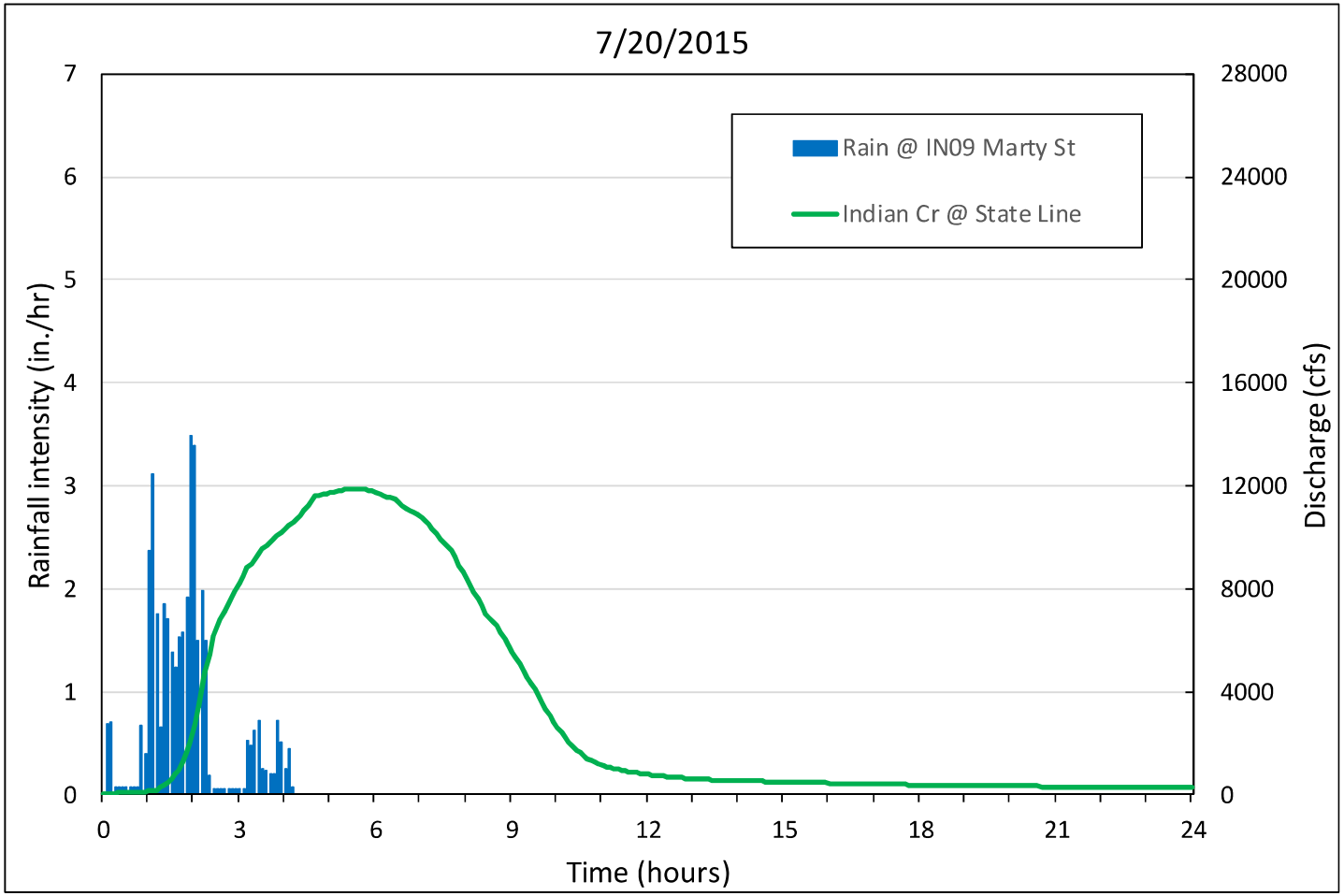
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	5.67	12
12	5.67	21
6	4.84	23
3	3.71	17
2	3.49	24
1	2.64	18

Figure A-5. Event of 7/10/1993



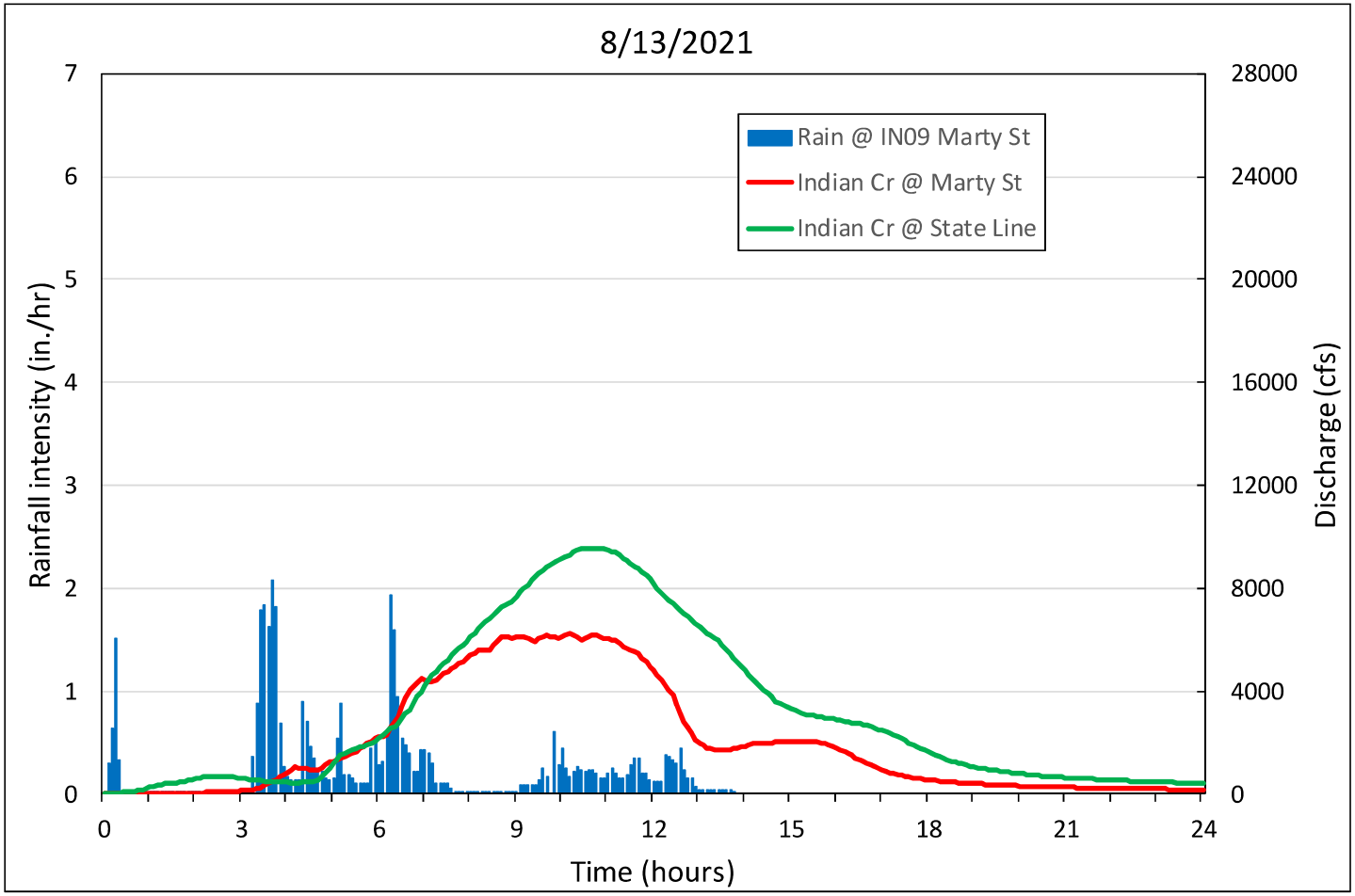
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.73	3
12	3.69	4
6	3.68	7
3	3.42	12
2	3.13	15
1	1.86	4

Figure A-6. Event of 6/14/2010



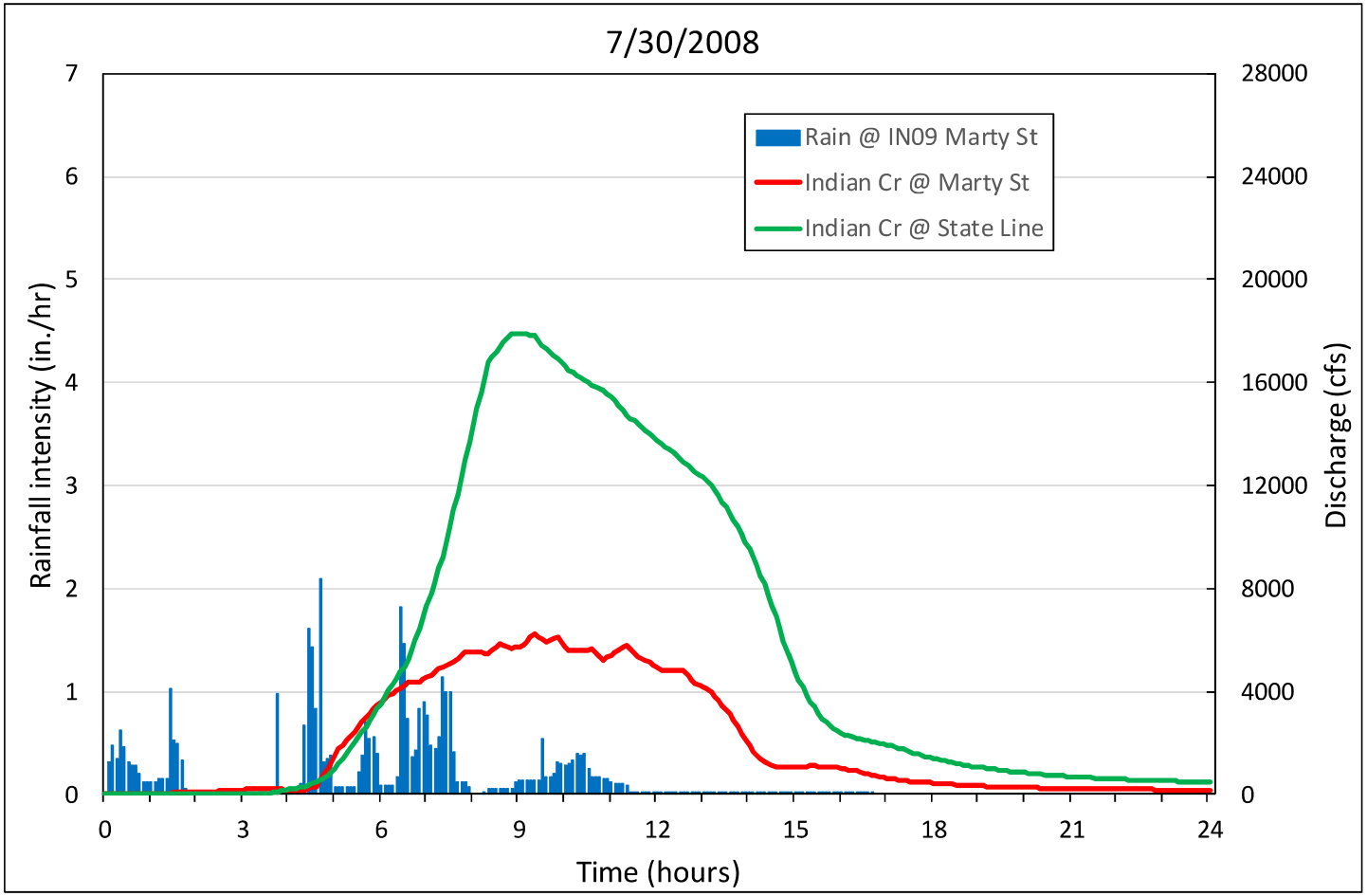
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.30	< 2
12	3.30	3
6	3.30	5
3	3.00	7
2	2.71	8
1	1.97	5

Figure A-7. Event of 7/20/2015



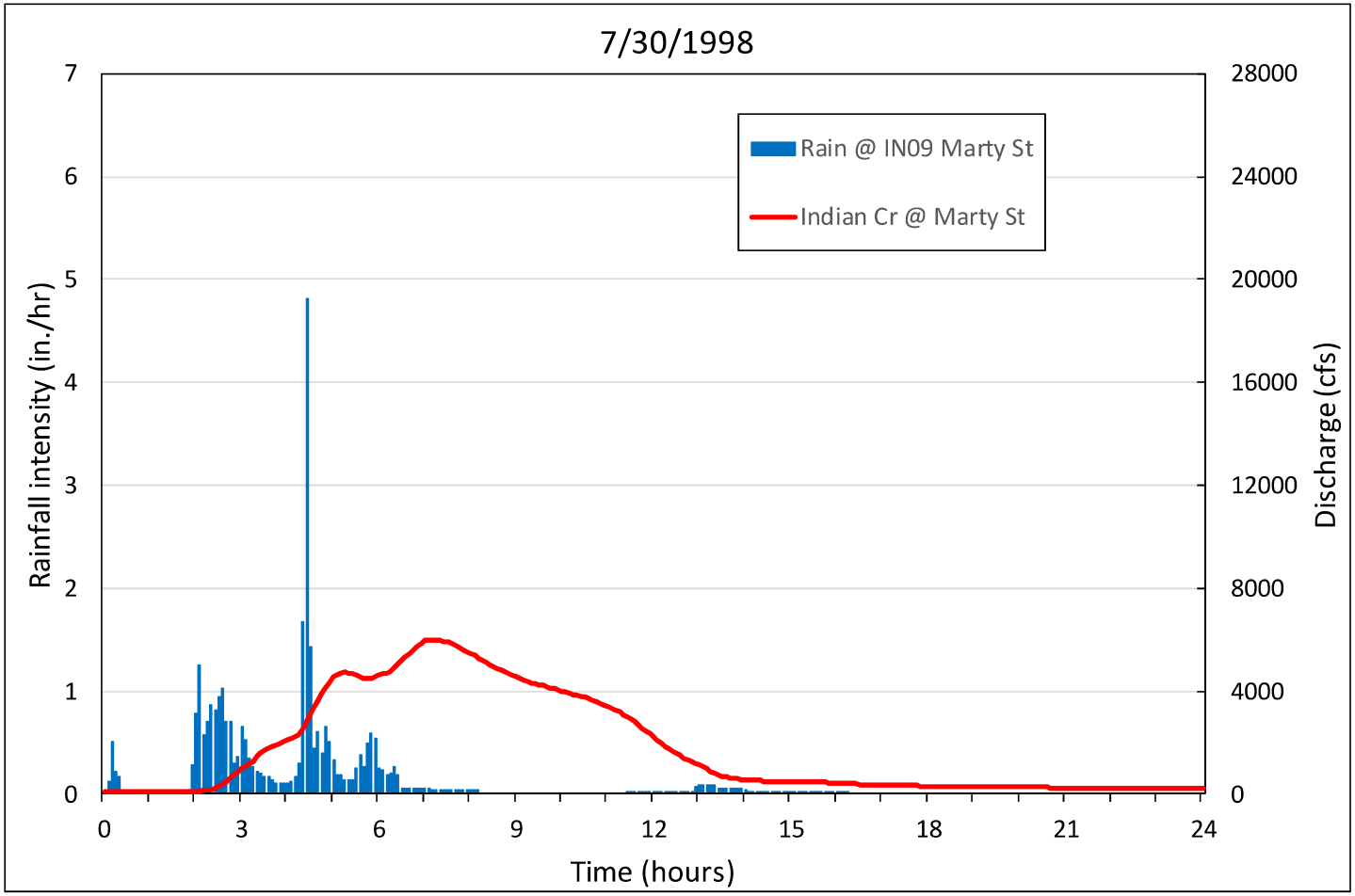
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.50	2
12	3.25	3
6	2.38	< 2
3	1.83	< 2
2	1.38	< 2
1	0.98	< 2

Figure A-8. Event of 8/13/2021



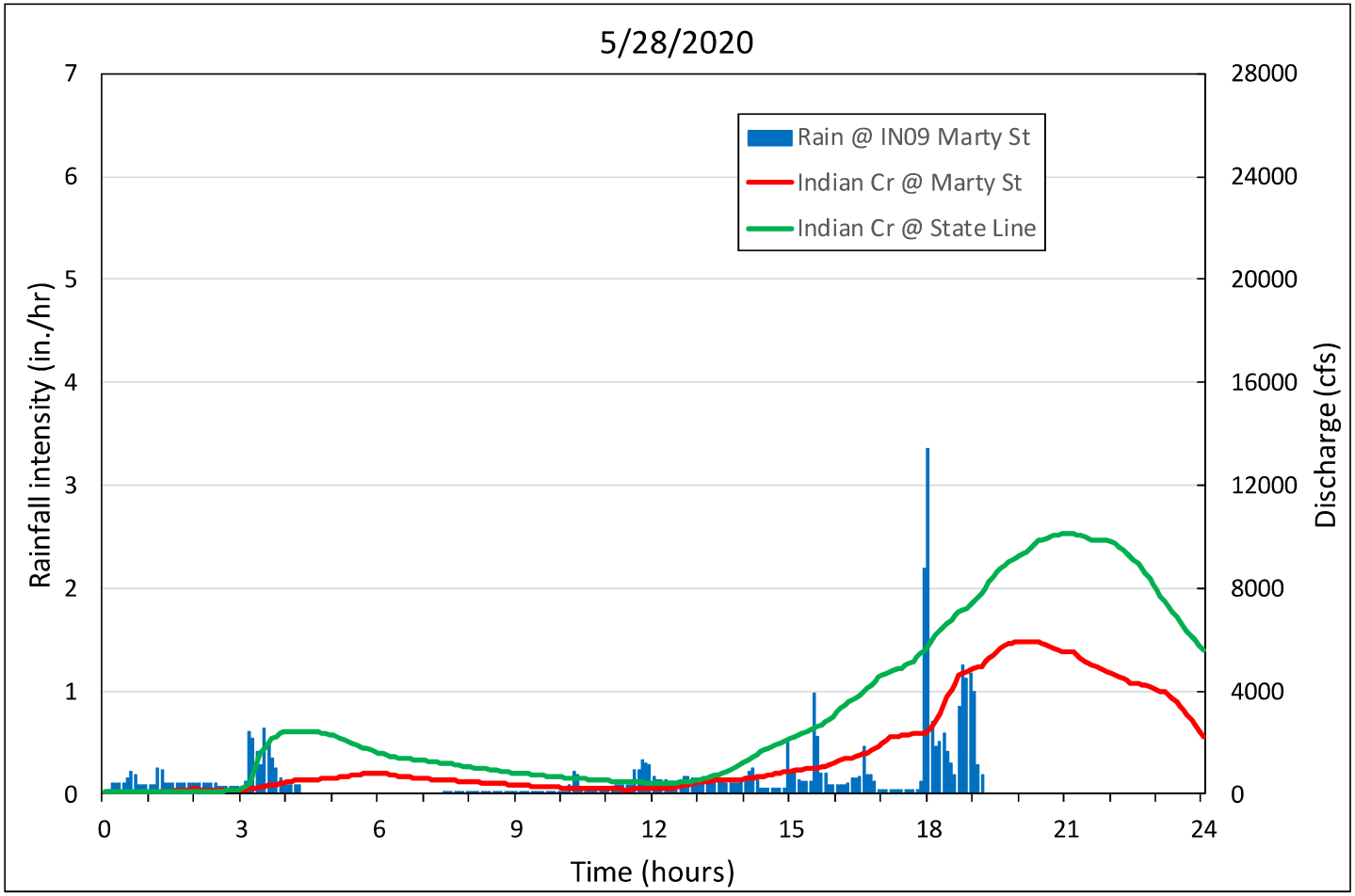
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.34	< 2
12	3.28	3
6	2.37	< 2
3	1.72	< 2
2	1.25	< 2
1	0.83	< 2

Figure A-9. Event of 7/30/2008



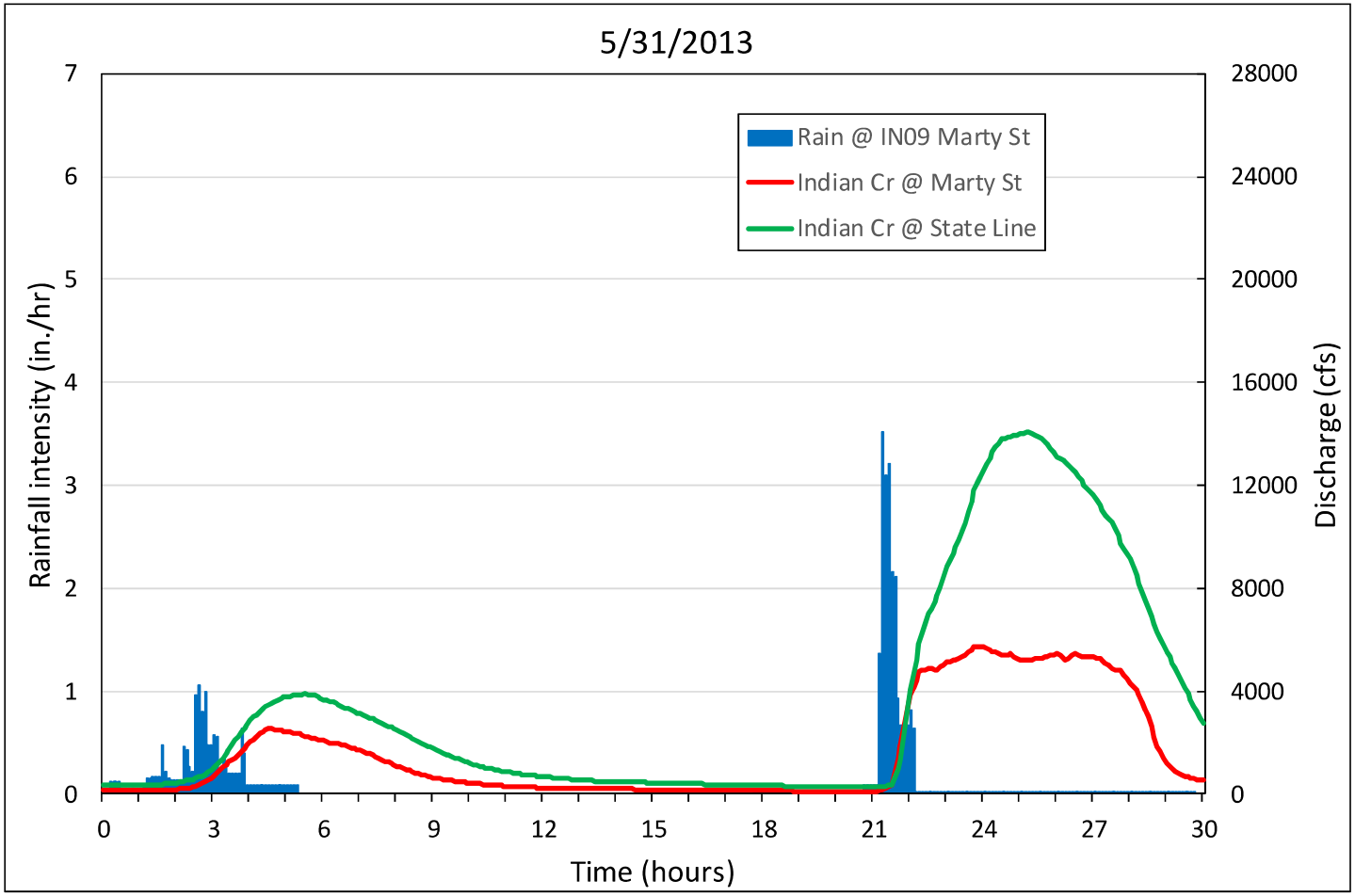
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	2.72	< 2
12	2.58	< 2
6	2.43	< 2
3	1.93	< 2
2	1.26	< 2
1	0.96	< 2

Figure A-10. Event of 7/30/1998



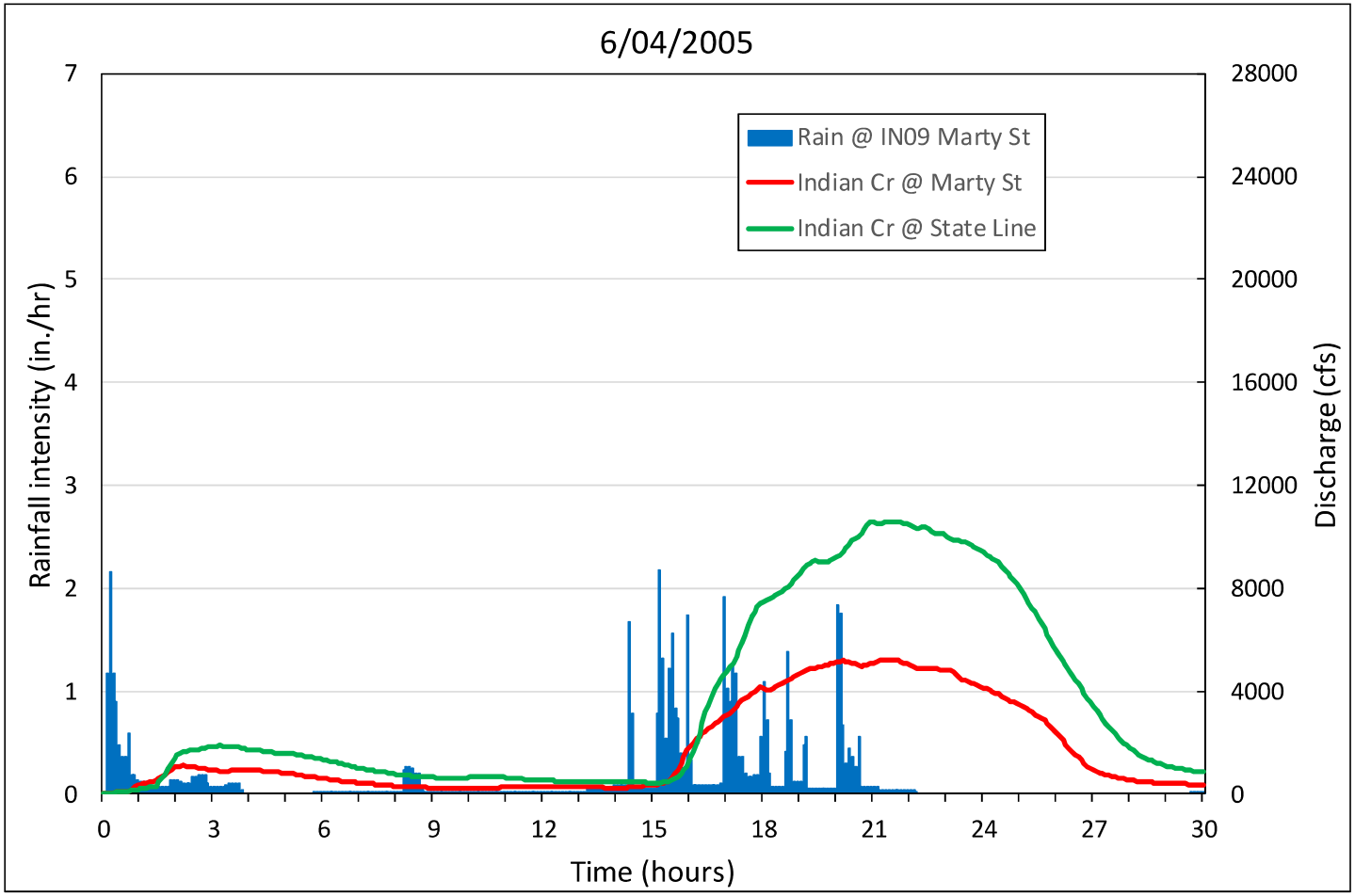
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.00	< 2
12	2.33	< 2
6	1.89	< 2
3	1.38	< 2
2	1.25	< 2
1	0.99	< 2

Figure A-11. Event of 5/28/2020



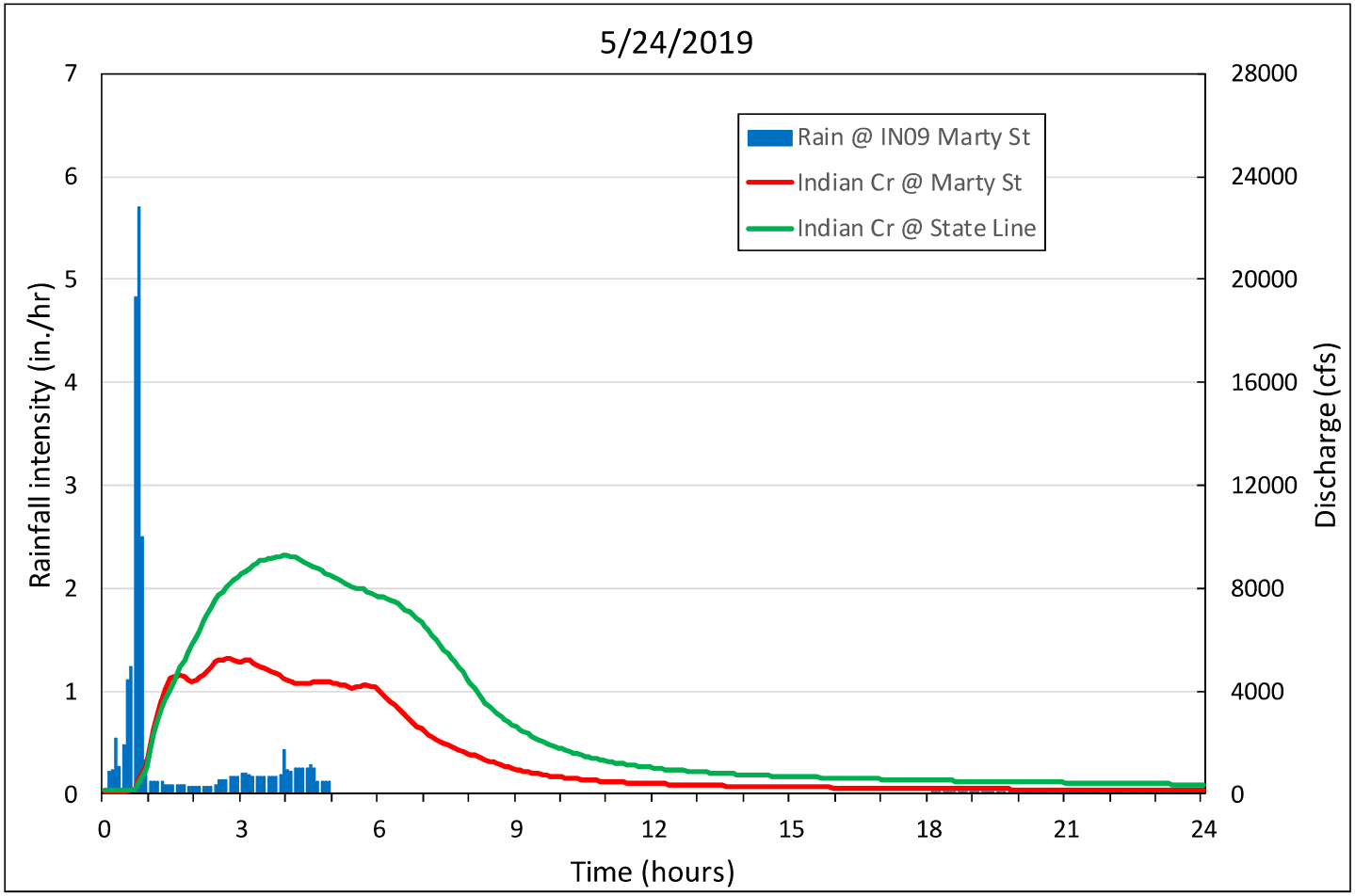
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	2.97	< 2
12	1.81	< 2
6	1.75	< 2
3	1.71	< 2
2	1.70	< 2
1	1.65	3

Figure A-12. Event of 5/31/2013



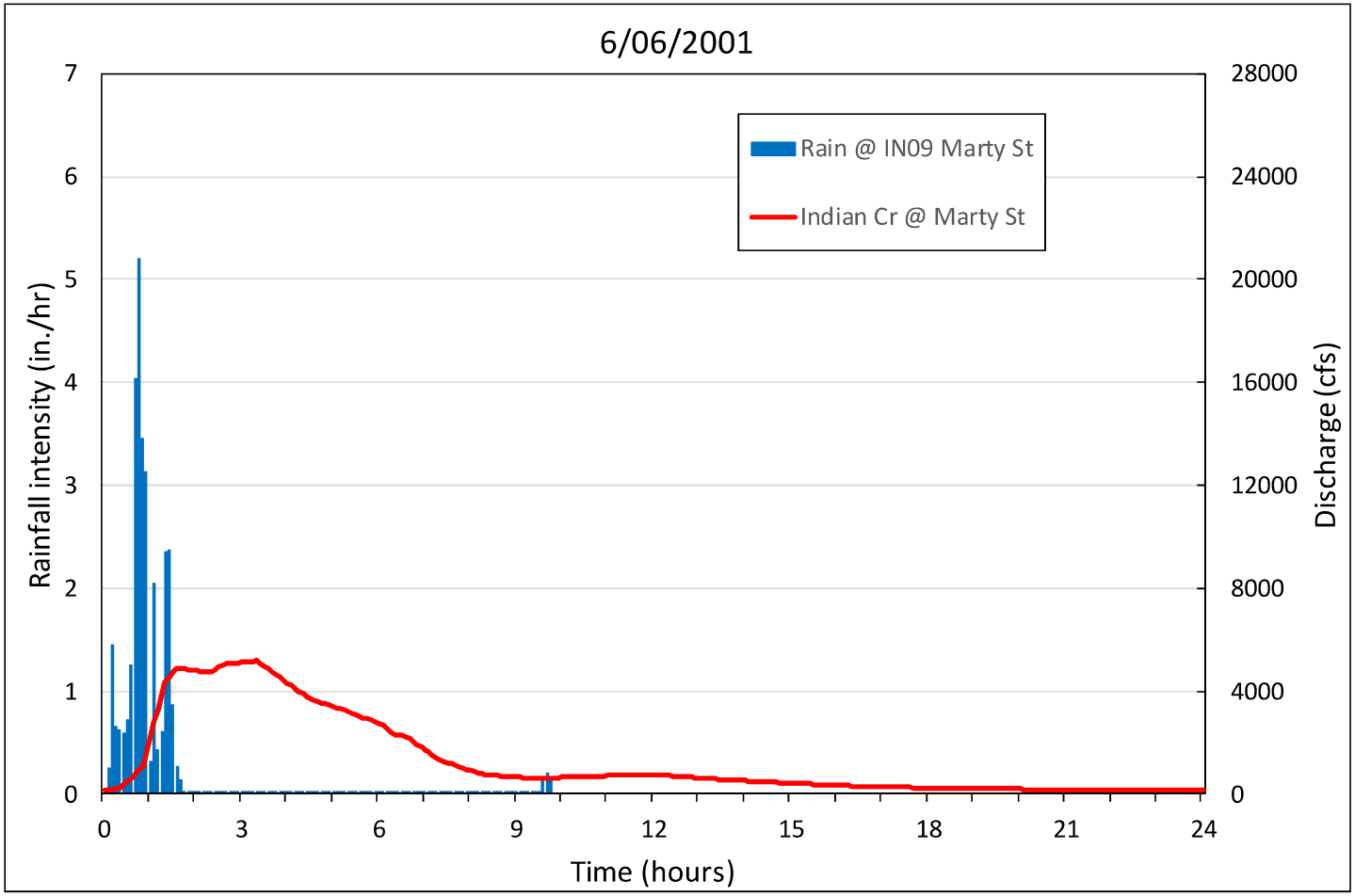
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	4.37	4
12	3.26	3
6	2.94	3
3	1.86	< 2
2	1.32	< 2
1	0.99	< 2

Figure A-13. Event of 6/04/2005



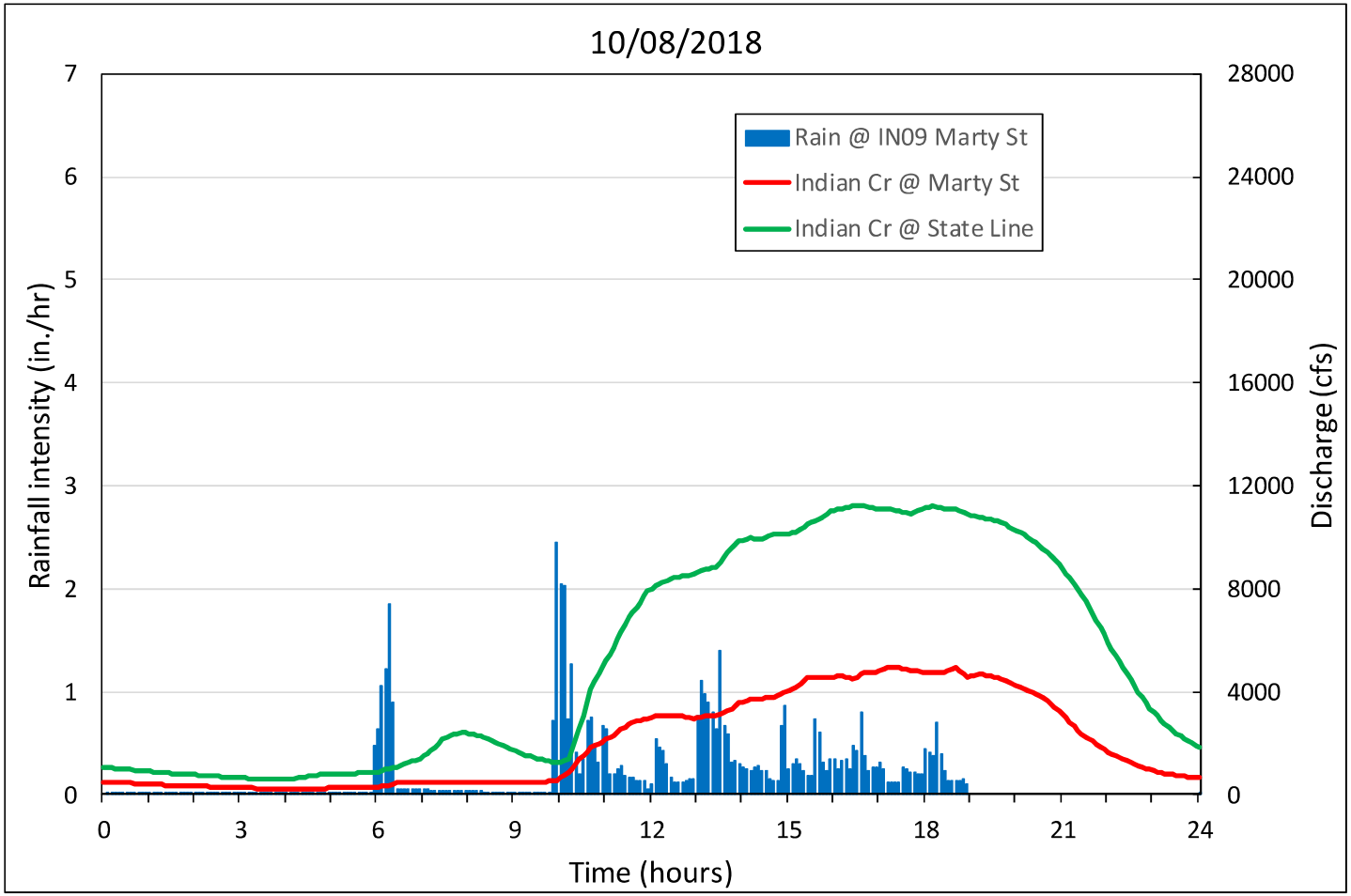
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	2.07	< 2
12	2.05	< 2
6	2.05	< 2
3	1.67	< 2
2	1.55	< 2
1	1.46	2

Figure A-14. Event of 5/24/2019



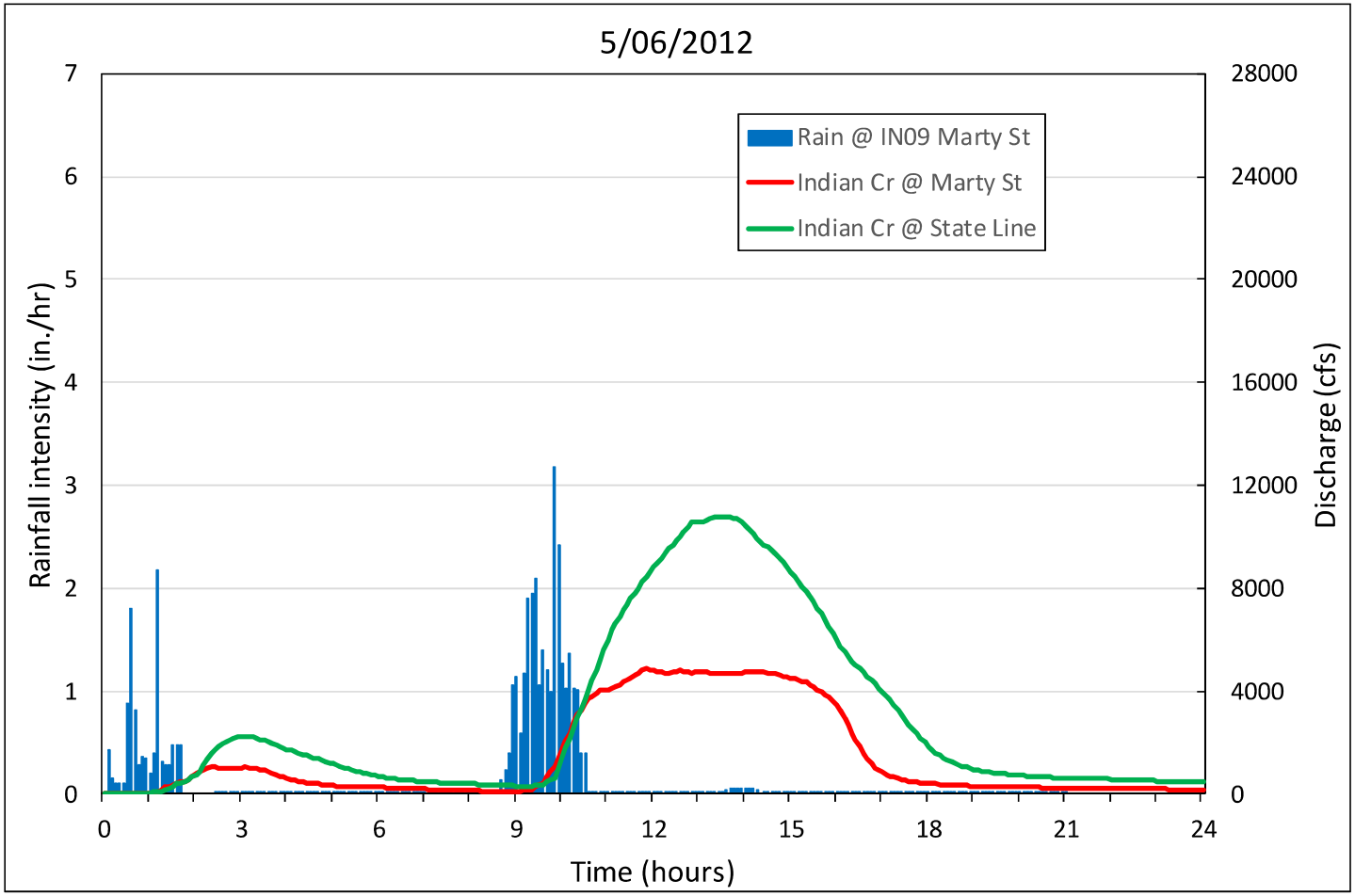
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	2.71	< 2
12	2.61	< 2
6	2.58	2
3	2.57	4
2	2.17	7
1	1.52	8

Figure A-15. Event of 6/06/2001



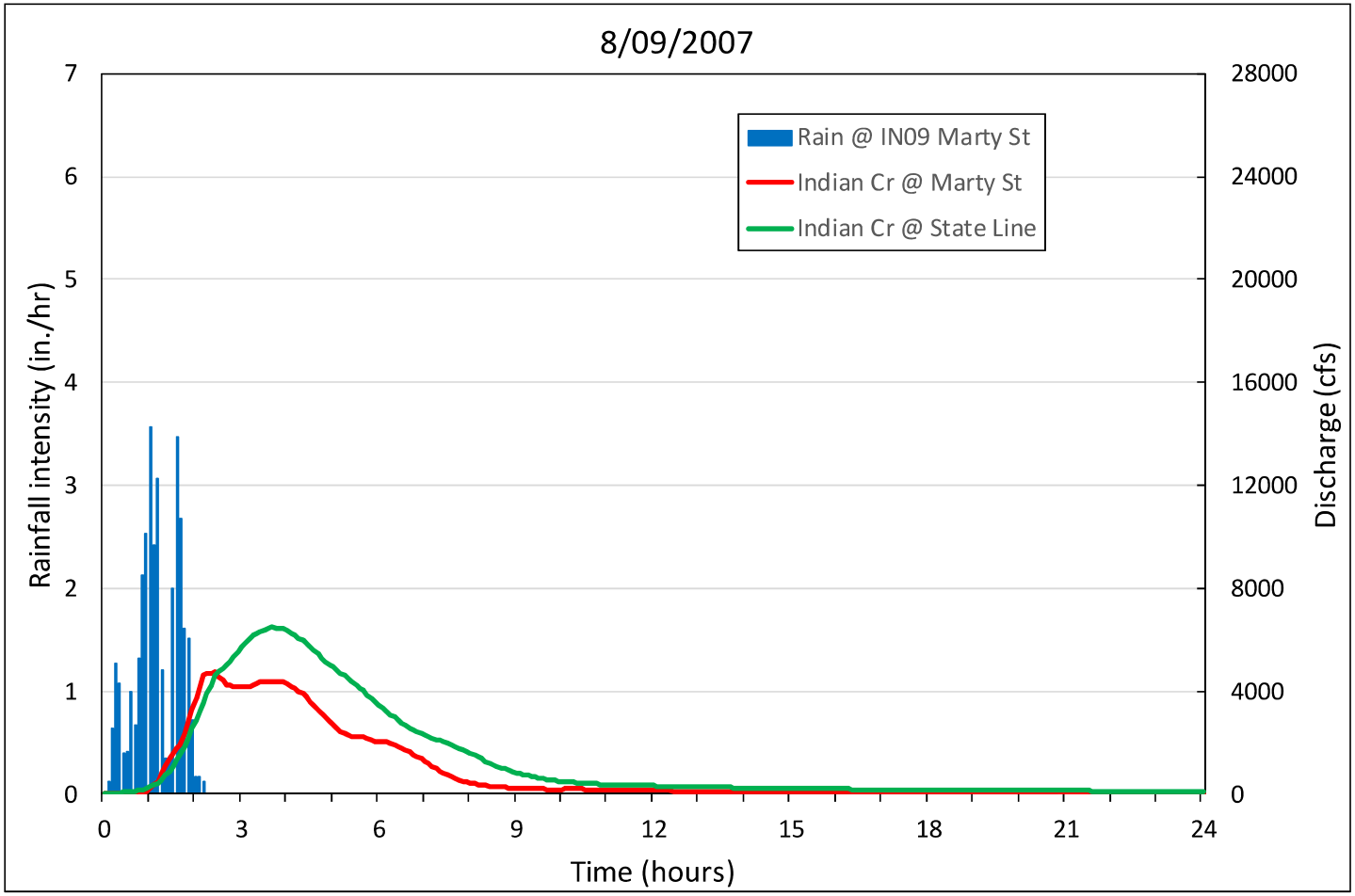
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	4.49	5
12	4.08	6
6	2.86	3
3	1.51	< 2
2	1.29	< 2
1	1.01	< 2

Figure A-16. Event of 10/08/2018



Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.35	< 2
12	3.21	3
6	2.36	< 2
3	2.29	3
2	2.28	4
1	1.65	3

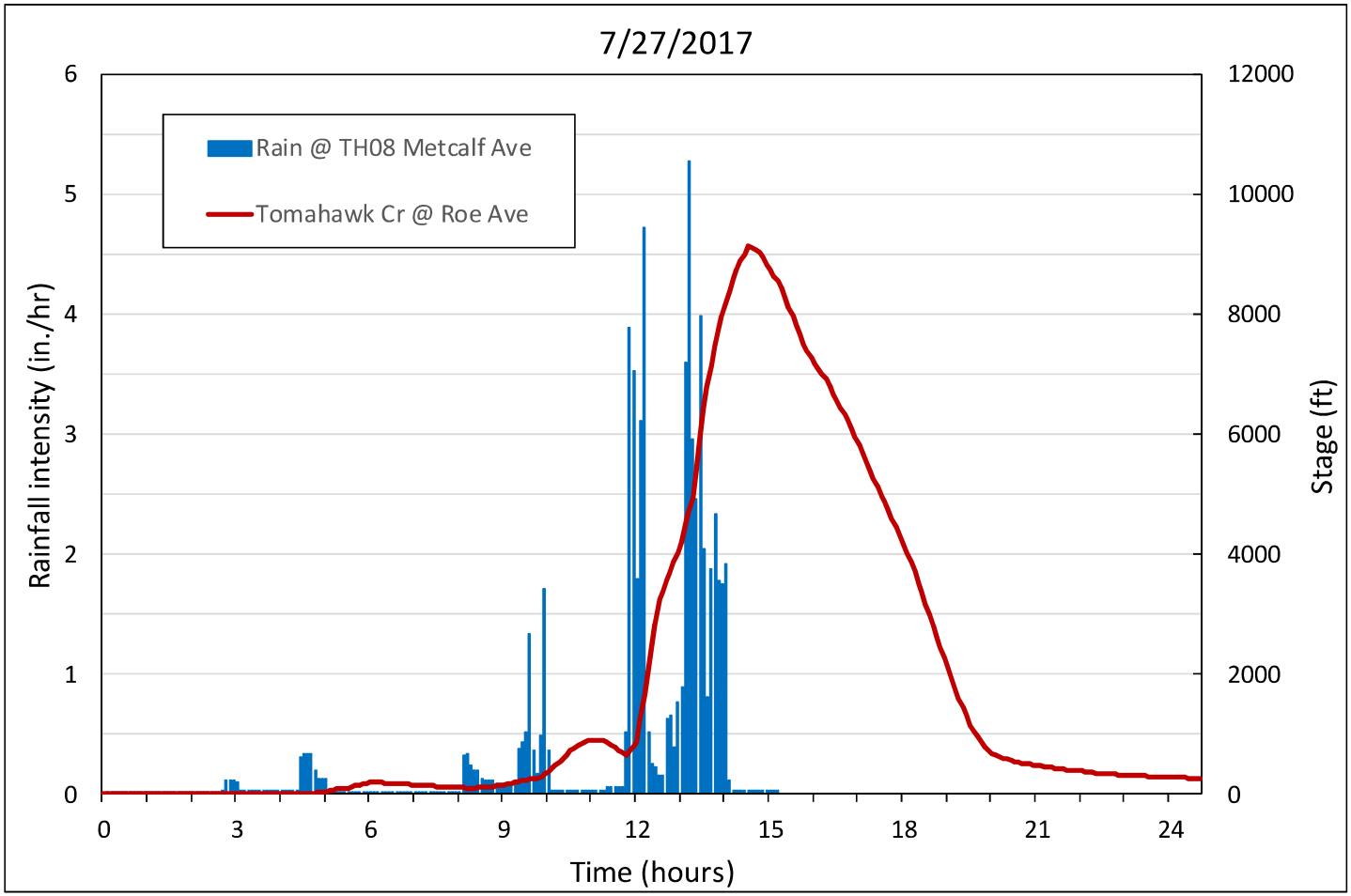
Figure A-17. Event of 5/06/2012



Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	2.91	< 2
12	2.91	2
6	2.91	3
3	2.91	7
2	2.89	10
1	2.11	7

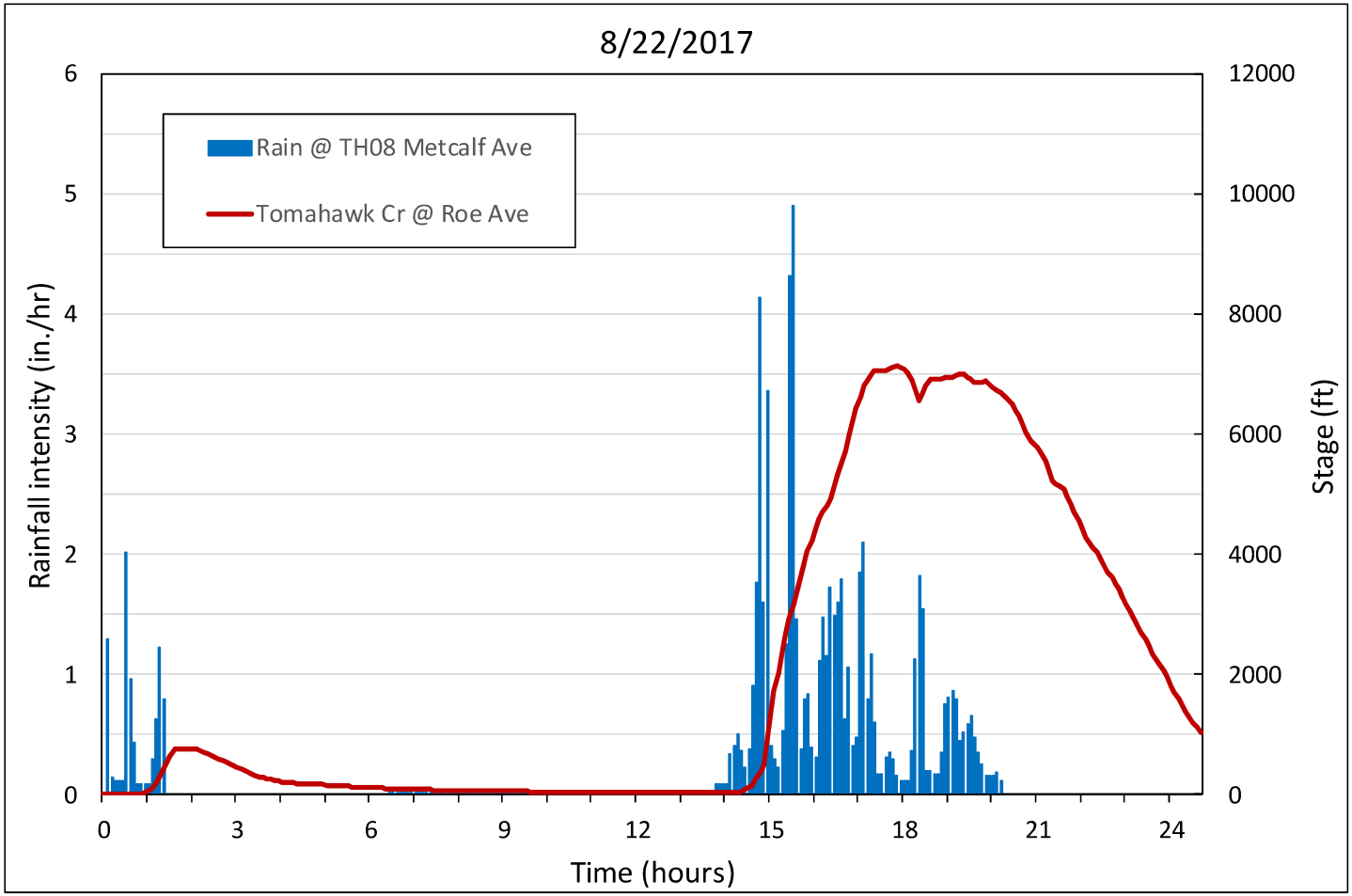
Figure A-18. Event of 8/09/2007

Appendix B – Significant Events on Tomahawk Creek



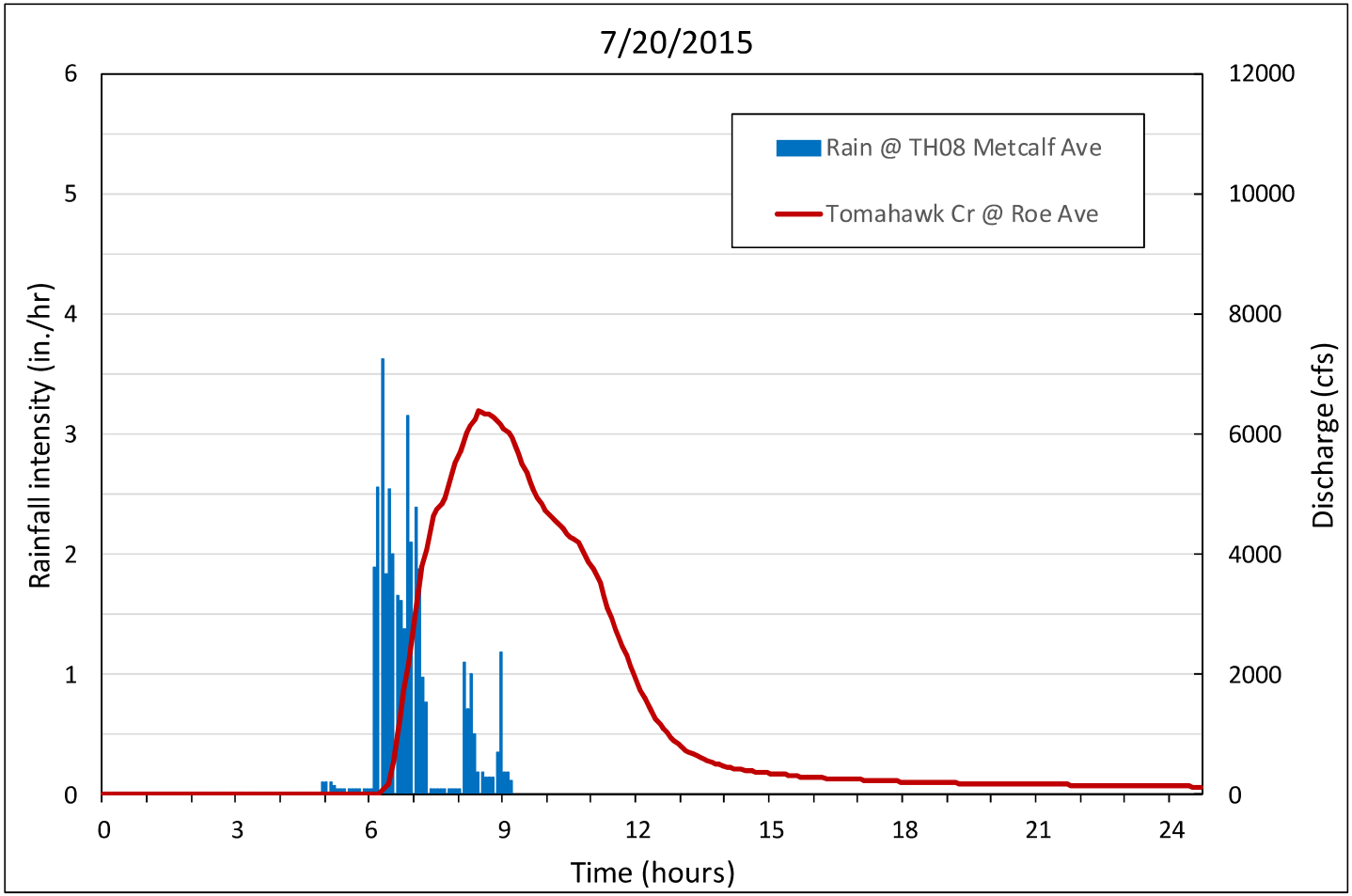
Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	5.52	10
12	5.46	17
6	5.15	30
3	4.46	41
2	3.92	42
1	2.57	16

Figure B-1. Event of 7/27/2017



Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	6.16	16
12	5.44	17
6	5.36	36
3	3.99	24
2	3.11	14
1	2.02	6

Figure B-2. Event of 8/22/2017



Rainfall duration (hours)	Max. depth (inches)	Recurr. interval (years)
24	3.19	<2
12	3.19	3
6	3.16	4
3	3.06	8
2	2.57	7
1	2.23	9

Figure B-3. Event of 7/20/2015

Appendix C – Development of Nested Storm Hyetographs

This appendix presents a method for developing a hyetograph for a nested storm with a desired duration and time-to-peak.

First, the depth-duration data for the desired annual exceedance probability (AEP) must be fitted with a continuous relationship to facilitate interpolation between tabulated durations and smooth out any irregularities. This is best done by fitting a polynomial equation to the base-10 logarithms of the depth and duration data. The process is demonstrated here using the 1%-AEP data for Indian Creek at Marty Street from Atlas 14. Table C1 shows the Atlas 14 depth-duration data and the base-10 logarithms of those values.

Table C-1. 1%-AEP rainfall depth-duration data for Indian Creek at Marty Street

Duration	Depth (inches)	log of duration in hours	log of depth in inches
5 min	1.06	-1.079	0.0253
10 min	1.56	-0.778	0.193
15 min	1.90	-0.602	0.279
30 min	2.70	-0.301	0.431
60 min	3.65	0.000	0.562
2 hr	4.60	0.301	0.663
3 hr	5.30	0.477	0.724
6 hr	6.56	0.778	0.817
12 hr	7.82	1.079	0.893
24 hr	8.86	1.380	0.947

Figure C-1 shows a plot of log(depth) vs. log(duration) and the trendline and corresponding equation fitted to those data with Excel's Add Trendline feature. A polynomial equation was chosen and its order was varied to achieve the best fit. In this example, a second-order polynomial equation fits the data well. In the displayed equation, x is the base-10 logarithm of duration in hours and y is the base-10 logarithm of depth in inches. The corresponding equation relating depth D in inches to duration z in hours is:

$$D = 10^{-0.08420 [\log(z)]^2 + 0.4008 \log(z) + 0.5556}$$

Excel allows the user to select the display mode for the coefficients in the fitted equation. The coefficients should be displayed in scientific notation with three decimal places. If the coefficients are rounded to fewer significant figures, the displayed equation might not fit the data as well as the trendline on the graph, which is computed with unrounded coefficients.

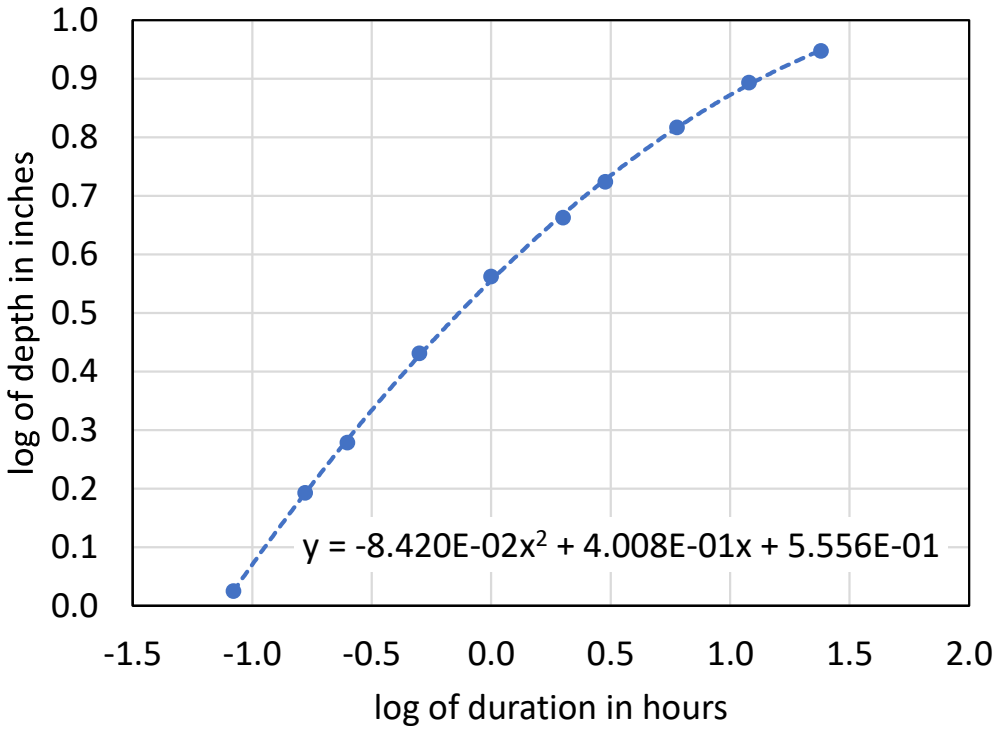


Figure C-1. Log-log plot of 1%-chance depth-duration data with fitted equation

Table C-2 compares the 1%-AEP rainfall depths from Atlas 14 and depth values from Atlas 14 with those computed with the fitted equation. The differences minor.

Table C-2. Comparison of 1%-AEP rainfall depths from Atlas 14 and fitted equation

Duration	Depth (inches)	
	Atlas 14	Fitted equation
5 min	1.06	1.06
10 min	1.56	1.56
15 min	1.90	1.92
30 min	2.70	2.67
60 min	3.65	3.59
2 hr	4.60	4.66
3 hr	5.30	5.34
6 hr	6.56	6.55
12 hr	7.82	7.76
24 hr	8.86	8.88

A hyetograph for a nested design storm can be developed from the fitted depth-duration equation in a spreadsheet table. Three hyetograph characteristics must be specified: the storm duration, the time-to-peak, and the time interval for the rainfall data. The storm duration and the time-to-peak must be whole multiples of the specified time interval. The time-to-peak is defined as the time to the start of the time interval with highest intensity. The equations used to develop the hyetograph incorporate the following notation:

- t = time from start of storm
- Δt = time interval for rainfall data (input)
- t_d = duration of storm (input)
- t_p = time-to-peak = time to start of time interval with greatest intensity (input)
- t_c = pivot time for distribution of n%-AEP rainfalls of different durations
- z = duration of n%-AEP rainfall starting at time t (for $t < t_c$) or ending at time t (for $t > t_c$)
- $D(z)$ = depth of n%-AEP rainfall for duration z, from fitted depth-duration equation
- P = cumulative rainfall depth at time t
- ΔP = incremental rainfall depth over time interval ending at time t
- R = average rainfall intensity over time interval ending at time t

The n%-AEP rainfall depths for all durations from Δt to t_d are distributed around a pivot time, t_c, located within the interval of greatest intensity as follows:

$$t_c = t_p + \left(\frac{t_p}{t_d}\right)\Delta t$$

The n%-AEP rainfall depths for all durations are distributed so that the fraction of the depth occurring before time t_c equals t_c/t_d and the fraction occurring after time t_c equals the $1 - t_c/t_d$.

The following equations are used to compute the cumulative depths for times from 0 to t_d at intervals of Δt. Incremental depths and intensities follow directly from the cumulative depths.

$$z = \begin{cases} \left(1 - \dfrac{t}{t_c}\right) t_d, & t < t_c \\ \left(\dfrac{t - t_c}{t_d - t_c}\right) t_d, & t > t_c \end{cases}$$

$$R = \begin{cases} \dfrac{t_c}{t_d}[D(t_d) - D(z)], & t < t_c \\ \dfrac{t_c}{t_d} D(t_d) + \left(1 - \dfrac{t_c}{t_d}\right) D(z), & t > t_c \end{cases}$$

Example

Develop a hyetograph table at 0.2-hour intervals for a nested storm with duration of 6 hours and a time-to-peak of 2.4 hours.

Δt = 0.2 hours
t_d = 6.0 hours
t_p = 2.4 hours

$$t_c = 2.4 + \left(\frac{2.4}{6.0}\right) 0.2 = 2.44$$

Applying the equations for z and R for times from 0 to 6 hours at 0.2-hour intervals yields the cumulative depths, incremental depths, and intensities shown in Table C-3. Note that the maximum values of incremental depth and intensity occur in the 0.2-hour time interval from 2.4 to 2.6 hours.

Table C-3. Incremental quantities for example nested hyetograph

Time t (hr)	z (hr)	D (in.)	Cum. depth P (in.)	Incr. depth ΔP (in.)	Intensity R (in./hr)
0.0	6.00	6.55	0.00		
0.2	5.51	6.40	0.06	0.06	0.31
0.4	5.02	6.24	0.13	0.07	0.34
0.6	4.52	6.06	0.20	0.07	0.37
0.8	4.03	5.85	0.28	0.08	0.41
1.0	3.54	5.63	0.38	0.09	0.46
1.2	3.05	5.37	0.48	0.10	0.52
1.4	2.56	5.07	0.60	0.12	0.61
1.6	2.07	4.72	0.75	0.14	0.72
1.8	1.57	4.28	0.93	0.18	0.89
2.0	1.08	3.71	1.16	0.23	1.16
2.2	0.59	2.88	1.49	0.34	1.68
2.4	0.10	1.17	2.19	0.70	3.49
2.6	0.27	2.00	3.85	1.66	8.29
2.8	0.61	2.92	4.39	0.55	2.73
3.0	0.94	3.51	4.75	0.35	1.77
3.2	1.28	3.96	5.01	0.27	1.33
3.4	1.62	4.32	5.23	0.21	1.07
3.6	1.96	4.63	5.41	0.18	0.90
3.8	2.29	4.89	5.56	0.16	0.78
4.0	2.63	5.12	5.70	0.14	0.68
4.2	2.97	5.32	5.82	0.12	0.61
4.4	3.30	5.51	5.93	0.11	0.55
4.6	3.64	5.68	6.03	0.10	0.50
4.8	3.98	5.83	6.12	0.09	0.46
5.0	4.31	5.97	6.21	0.08	0.42
5.2	4.65	6.10	6.29	0.08	0.39
5.4	4.99	6.23	6.36	0.07	0.37
5.6	5.33	6.34	6.43	0.07	0.34
5.8	5.66	6.45	6.49	0.06	0.32
6.0	6.00	6.55	6.55	0.06	0.30

JOHNSON COUNTY KANSAS
Public Works

To: **Dawn Livingston**
Project Manager - Mitigation Division
FEMA Region 7

From: **Sarah Smith, PE**
Lead Program Engineer
Johnson County Stormwater Management Program

Subject: CTP – Alternate Hydrology
Response to FEMA Comments from 2/13/2024

Date: **2/21/2024**

This memo presents responses to the four storm-related comments in FEMA's 2/13/2024 review of Johnson County's Alternative Hydrology Memorandum. The italicized bullet points are FEMA's comments.

- *In Appendix A of Dr. McEnroe's report, the 8/22/2017 and 7/27/2017 events each show significant rainfall amounts in the first 1-2 hours of the 24-hour storm events. The majority of the rainfall occurs between hours 12 and 24 of the events, supporting the use of the 12-hour duration. However, it could be argued that the presence of significant rainfall in the initial hours of the event may have resulted in saturated conditions creating larger peak discharges than would have resulted with use of 12-hour design storm.*

We agree that the wetter soil conditions caused by antecedent rainfall can increase runoff volumes and peak discharges. However, we think the proposed 12-hour storm duration adequately accounts for that concern. The 8/22/17 event had a total duration of 20 hours, but the duration of active rainfall was less than 8 hours. The 12-hour break between the initial burst of heavy rainfall in the early morning and the much larger event in the afternoon gave the soil's infiltration capacity a change to recover slightly. The event of 7/27/2017 also had a total duration of almost 20 hours, but the duration of active rainfall was only 6 hours.

- *Although the report describes in detail the NOAA temporal distributions, the discussion of why these temporal distributions are not appropriate is limited to consideration of the five cases of the 24-hour NOAA temporal distributions. Would not a more convincing case be made by considering the 12-hour NOAA temporal distributions and evaluating the 50th percentile cases for each of the four quartile storm types?*

We agree that an analysis of the NOAA's 50th-percentile temporal distributions for the 12-hour duration would be helpful. The analysis of the 10th-percentile and 90th-percentile distributions is repeated here for the 50th-percentile distributions from the four quartile groups.

Figure 1 shows 12-hour hyetographs with the 1%-ACE, 12-hour rainfall of 7.82 inches (the Atlas

14 value for Indian Creek at Marty Street) distributed according to the median (50%) patterns for the four quartiles and for the entire dataset. (The first-quartile storms comprise 56% of the events in the dataset.) The median distributions for the four quartile groups exhibit much less temporal variability than is seen in the hyetographs of the actual storms in Appendices A and B of the research report. The temporal averaging process smooths out the peaks on the individual storm hyetographs because those peaks occur at different times.

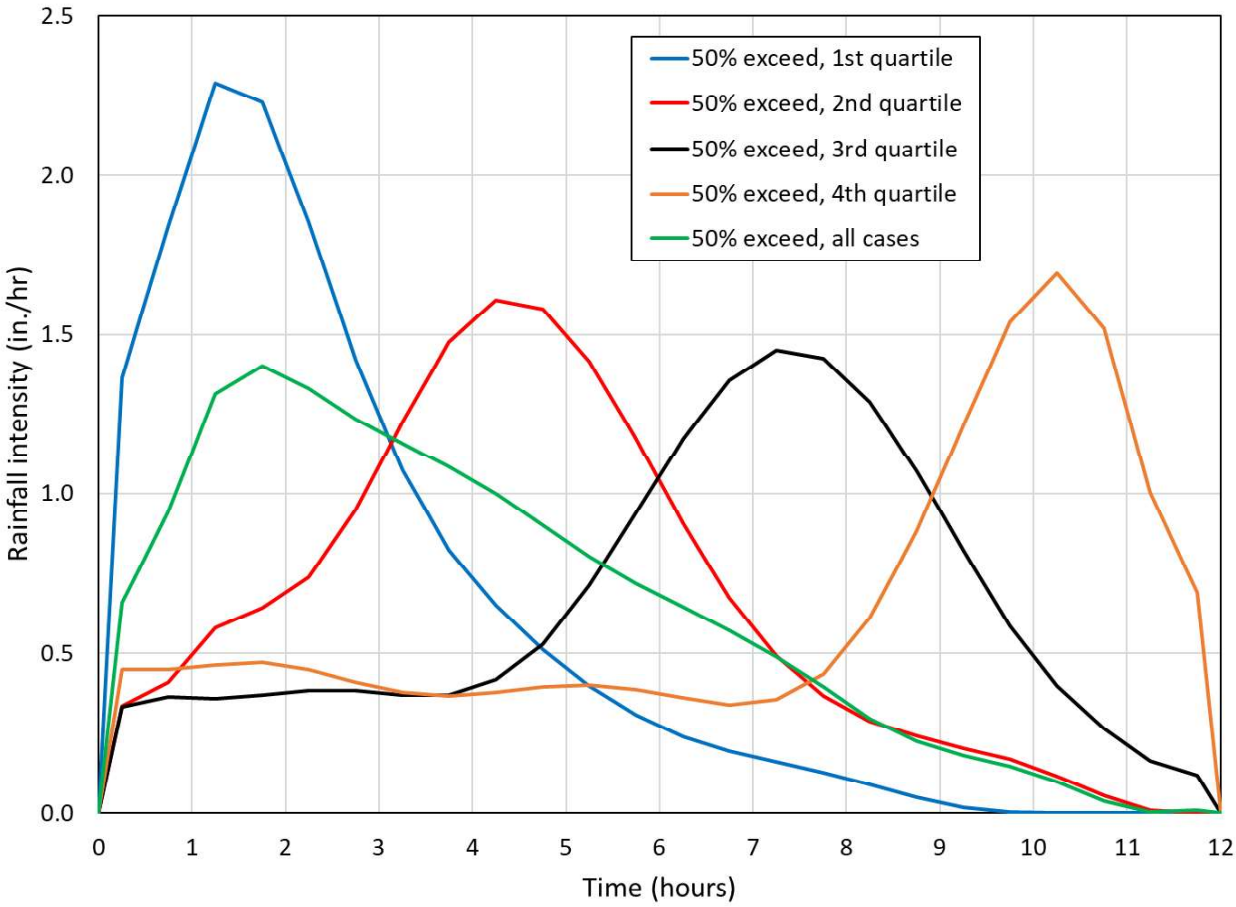


Figure 1. 1%, 12-hour hyetographs for Indian Creek with five NOAA temporal distributions

Table 1 shows the maximum depths of rainfall for durations of 30 minutes and 1, 2, 3, 6, and 12 hours for the five hyetographs. Included and highlighted for comparison are the maximum depths for the 12-hour nested storm, which are the 1%-chance depths from Atlas 14. Table 2 shows the recurrence intervals of the maximum depths in Table 1. The five hyetographs derived from NOAA's median distributions for the four quartile groups and the entire dataset contain maximum rainfall depths for durations shorter than 12 hours that differ greatly from the 1%-chance depths from Atlas 14. The maximum depths for durations of three hours and shorter are much too low for the 2nd-, 3rd-, and 4th-quartile groups and all cases. Distributing the synthetic storm rainfall in any of these patterns would result in substantial underestimation of the 1%-ACE discharges throughout the Indian and Tomahawk Creek watersheds. The

maximum depths for the 1st-quartile group are too low for the shorter durations and too high for the longer durations. Distributing the synthetic storm rainfall in that pattern would result in underestimation of the 1%-ACE discharges at locations with times of concentration of one hour or less.

Table 1. 1%-chance 12-hour synthetic storms for Indian Creek: maximum depth vs. duration

Temporal distribution for 1%-ACE 12-hr storm	Maximum rainfall depth of stated duration (inches)					
	30 min	1	2 hr	3 hr	6 hr	12 hr
Nested	**2.70**	**3.65**	**4.60**	**5.30**	**6.56**	**7.82**
NOAA 50%, 1st quartile	1.14	2.26	4.10	5.50	7.38	7.82
NOAA 50%, 2nd quartile	0.80	1.59	3.04	4.24	6.48	7.82
NOAA 50%, 3rd quartile	0.72	1.44	2.76	3.88	5.88	7.82
NOAA 50%, 4th quartile	0.85	1.61	2.98	3.93	5.32	7.82
NOAA 50%, all events	0.70	1.37	2.64	3.76	6.27	7.82

Table 2. 1%-chance, 12-hour synthetic storms for Indian Creek: recurrence intervals of maximum depths

Temporal distribution for 1%-ACE 12-hr storm	Recurrence interval of max. depth of stated duration (yr)					
	30 min	1	2 hr	3 hr	6 hr	12 hr
Nested	**100**	**100**	**100**	**100**	**100**	**100**
NOAA 50%, 1st quartile	2	9	54	122	192	100
NOAA 50%, 2nd quartile	<2	3	13	32	94	100
NOAA 50%, 3rd quartile	<2	2	9	21	57	100
NOAA 50%, 4th quartile	<2	3	12	22	35	100
NOAA 50%, all events	<2	2	7	19	79	100

- *Although the justification for use of the 12-hour, 3-hour to peak storm is reasonable, the overall result of using the shorter storm duration instead of the 24-hour, 12-hour to peak storm is to reduce the magnitude of the modeled 1% annual chance of exceedance (ACE) discharge at the 06893300 Indian Creek at Overland Park (at Marty Street) location by about 13%, from 15,700 cfs to 13,730 cfs. The 1% ACE discharge computed by frequency analysis at the 06893300 gage location is 15020 cfs. The modeled 1% ACE discharge using the 24-hour/12-hour to peak storm differs from the gage discharge estimate by 4.5%, while the modeled 1% ACE discharge using the 12-hour/3-hour to peak storm differs from the gage discharge estimate by -8.6%. Although both of the modeled estimates are within the standard error (68% confidence interval) of the 1% ACE gage estimate, the modeled 1% ACE estimate using the 12-hour/3-hour to peak storm differs from the gage estimate by more than the modeled 1% ACE estimate using the 24-hour/12-hour to peak storm. Dr. McEnroe's report presents a summary of several key points from FEMA's guidance regarding rainfall inputs to flood models and addresses the presented points in the report; however, FEMA's guidance also states that "The storm duration must also be large enough to provide reasonable runoff…". Please include*

clarification of the reasonableness of the alternative 1% ACE estimate resulting from the proposed 12-hour/3-hour to peak storm in comparison to the standard 24-hour storm duration.

Our calibration efforts are focused on flood stage rather than discharge. Our primary calibration point on Indian Creek is USGS gage 6893300, located on the downstream side of the Marty Street bridge. The recorded flood stages at that gage are reliable. However, as the research report explains, the USGS-reported discharges appear to be unreliable for floods with peak stages over 870 feet (discharges over 5000 cfs). The USGS rating curve for stages over 862 feet is a simple power function fitted to field-measurement data for stages only up to 869.32 feet. The rating curve appears to overestimate discharges for flood stages over 870 feet because it does not account for the substantial backwater effects from the bridge at 103rd Street, 650 feet downstream, with its waterway capacity exceeded.

The controlling low chord elevation of the Marty Street Bridge is approximately 877.66 feet, which is located at the downstream end of bridge. The stage-discharge curve is expected to stagnate at this elevation, with discharges steadily increasing after the bridge begins to overtop (at approximately 880.80 feet, not factoring in railing). The USGS stage-discharge curve does not reflect these breakpoints, which further attests to the unreliability of the USGS rating curve for high discharges and shows that stage is more appropriate for calibration purposes at this gage location.

Our frequency analysis of annual flood peaks 1977-2022 indicates that the 1%-ACE flood stage at the Marty Street gage is 875.70 feet. In the HEC-RAS 2D model as currently calibrated, the 12-hour nested storm with the 3-hour time-to-peak produces a peak stage at that is much closer to this target value than the 24-hour nested storm with the 12-hour time-to-peak.

- *The resulting 1% ACE modeled elevations are also compared in the report by Dr. McEnroe. As would be expected, the 12-hour duration/3-hour to peak storm yields the lowest 1% ACE estimated elevation of 877.34', which is lower by about 0.8' than the 1% ACE modeled elevation using the 24-hour/12-hour to peak storm. The 1% ACE modeled elevation using the 24-hour/6-hour to peak storm, however, is 877.51', less than 0.2' greater than 1% ACE modeled elevation using the 12-hour duration/3-hour to peak storm. This confirms the conclusion noted in Dr. McEnroe's report that "…that the amount of rain that precedes the peak matters much more than the amount of rain that falls after the peak." Please include clarification, given this comparison, of the preference for the 12-hour/3-hour to peak storm over a standard 24-hour storm duration using a 6-hour to peak distribution.*

It is true that the 12-hour nested storm with the 3-hour time-to-peak produces flood stages that are not much lower than the stages for the 24-hour storm with the 6-hour time-to-peak. Olsson's comparisons at six locations showed an average difference of 0.3 feet and a maximum difference of 0.4 feet as shown in the attachment. At the Marty Street gage, the stage for the 12/3 storm is closer to target value than the stage for the 24/6 storm but the difference is only 0.17 feet.

Feel free to reach out to us with any questions, comments, or additional documentation that is required.

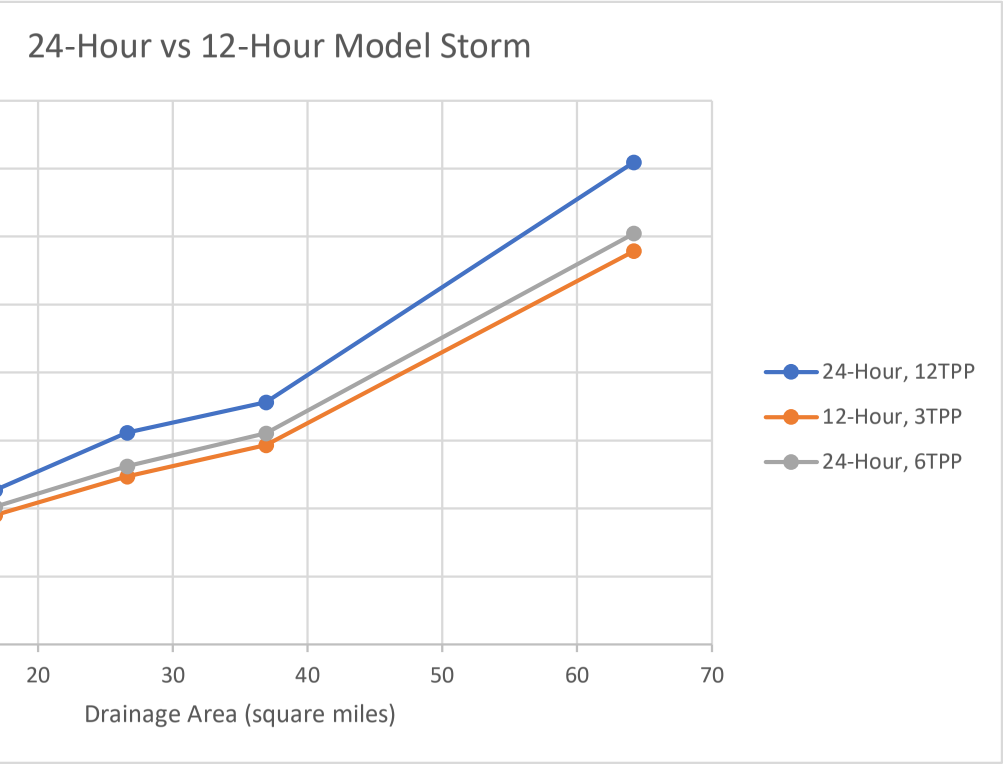
Sincerely,



Sarah Smith

mi^2)	24-Hour, 12 TPP* Peak Discharge	24-Hour, 6 TPP* Peak Discharge	12-Hour, 3 TPP* Peak Discharge	24-Hour, 12 TPP* Peak Elevation	24-Hour, 6 TPP* Peak Elevation	12-H
1.5	2950	2370	2260	1036.4	1035.7	
5.3	7460	6200	5870	994.7	993.9	
16.8	11340	10110	9500	908.2	907.3	
26.6	15570	13120	12350	878	877.4	
36.9	17820	15530	14670	857.6	856.8	
64.2	35480	30240	28940	819	818	

annel section



APPENDIX M.2 – PRECIPITATION TIME SERIES

THEORETICAL PRECIPITATION EVENTS

JULY 2017 HISTORIC EVENT

AUGUST 2017 HISTORIC EVENT

MAY 2020 HISTORIC EVENT

JULY 2024 HISTORIC EVENT

JULY 2025 HISTORIC EVENT

M INTERIOR DRAINAGE

M.2 PRECIPITATION TIME SERIES

The 0.1 AEP, 0.01 AEP, July 2017, August 2017, May 2020, July 2024, and July 2025 hydrograph data are documented in this section.

For theoretical annual exceedance probability events, one precipitation hyetograph was applied for the entire watershed. For historical events, a grouping of gage locations used the Thiessen polygon method to implement recorded precipitation hyetographs across the watershed. See Figure M.2.1 for Stormwatch gage locations.

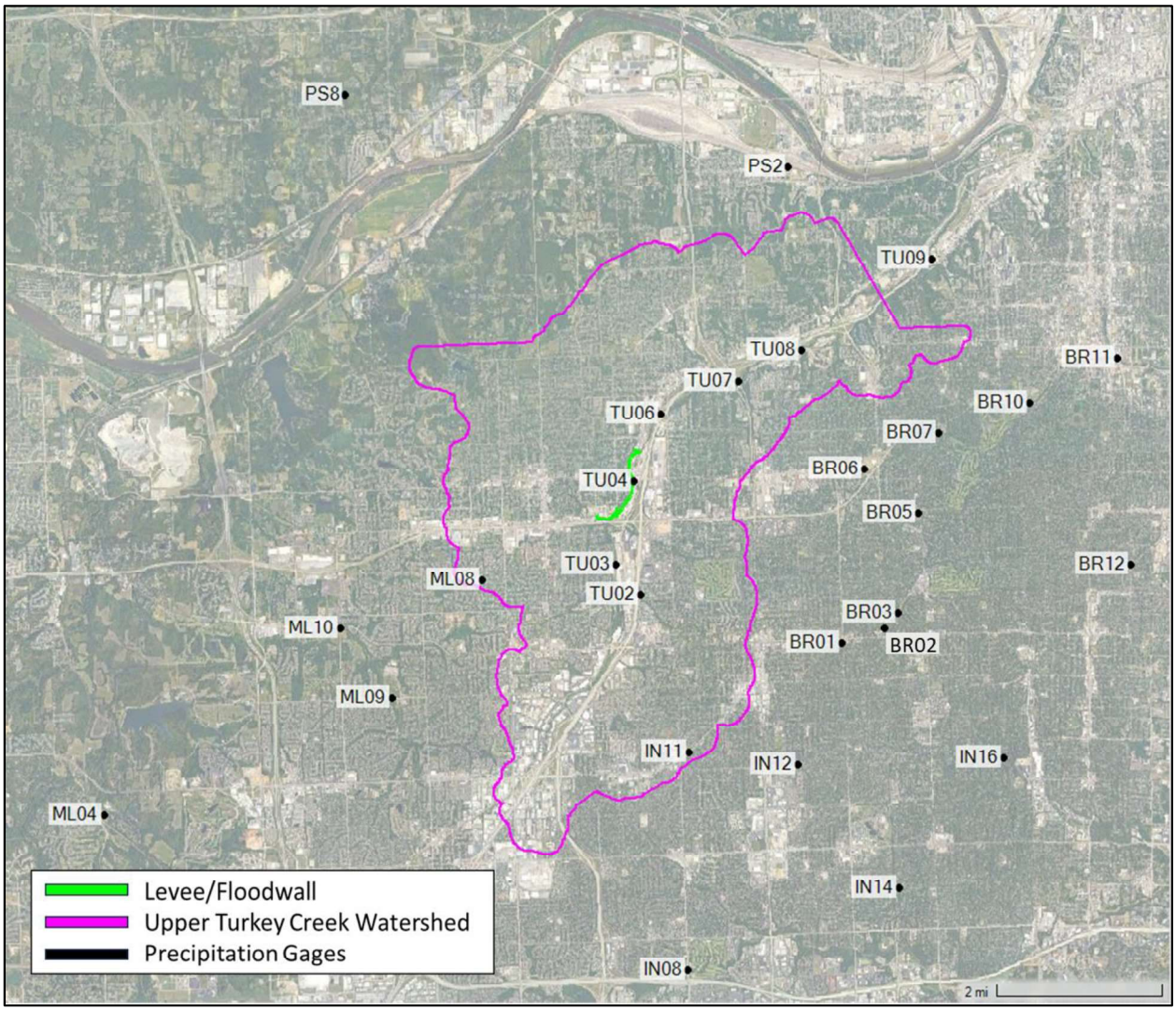


Figure M.2.1. Stormwatch Gage Locations.

M.2.1 THEORETICAL PRECIPITATION EVENTS

The theoretical precipitation events are 24-hour events, with a 5-minute time step. See Section 5.1.5 for more information.

Table M.2.1. Theoretical AEP Event Hyetograph Data.

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
0	0	0
5	0.004	0.006
10	0.004	0.006
15	0.004	0.006
20	0.006	0.009
25	0.006	0.009
30	0.006	0.009
35	0.006	0.009
40	0.006	0.009
45	0.006	0.009
50	0.006	0.009
55	0.006	0.009
60	0.006	0.009
65	0.006	0.009
70	0.006	0.009
75	0.006	0.009
80	0.006	0.009
85	0.006	0.009
90	0.006	0.009
95	0.006	0.009
100	0.006	0.009
105	0.006	0.009
110	0.006	0.009
115	0.006	0.009
120	0.006	0.009
125	0.006	0.009
130	0.006	0.009
135	0.006	0.009
140	0.006	0.009
145	0.006	0.009

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
150	0.006	0.009
155	0.006	0.009
160	0.006	0.009
165	0.006	0.009
170	0.006	0.009
175	0.006	0.009
180	0.006	0.009
185	0.006	0.009
190	0.006	0.009
195	0.006	0.009
200	0.006	0.009
205	0.006	0.009
210	0.006	0.009
215	0.006	0.009
220	0.007	0.012
225	0.008	0.012
230	0.008	0.012
235	0.008	0.012
240	0.008	0.012
245	0.008	0.012
250	0.008	0.012
255	0.008	0.012
260	0.008	0.012
265	0.008	0.012
270	0.008	0.012
275	0.008	0.012
280	0.008	0.012
285	0.008	0.012
290	0.008	0.012
295	0.008	0.012
300	0.008	0.012
305	0.008	0.012
310	0.008	0.012
315	0.008	0.012

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
320	0.008	0.012
325	0.008	0.012
330	0.008	0.012
335	0.008	0.012
340	0.008	0.012
345	0.008	0.012
350	0.009	0.015
355	0.01	0.015
360	0.01	0.015
365	0.01	0.015
370	0.01	0.015
375	0.01	0.015
380	0.01	0.015
385	0.01	0.015
390	0.01	0.015
395	0.01	0.015
400	0.01	0.015
405	0.01	0.015
410	0.01	0.015
415	0.01	0.015
420	0.01	0.015
425	0.01	0.015
430	0.01	0.015
435	0.01	0.015
440	0.01	0.015
445	0.01	0.015
450	0.01	0.015
455	0.01	0.015
460	0.01	0.015
465	0.011	0.018
470	0.012	0.018
475	0.012	0.018
480	0.013	0.021
485	0.013	0.021

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
490	0.013	0.021
495	0.013	0.021
500	0.013	0.021
505	0.013	0.021
510	0.013	0.021
515	0.013	0.021
520	0.014	0.022
525	0.015	0.024
530	0.015	0.024
535	0.015	0.024
540	0.015	0.024
545	0.015	0.024
550	0.016	0.026
555	0.017	0.028
560	0.017	0.028
565	0.017	0.028
570	0.017	0.028
575	0.017	0.028
580	0.019	0.03
585	0.019	0.031
590	0.019	0.031
595	0.023	0.036
600	0.023	0.037
605	0.023	0.037
610	0.029	0.046
615	0.029	0.046
620	0.03	0.047
625	0.035	0.055
630	0.035	0.055
635	0.036	0.058
640	0.04	0.064
645	0.04	0.064
650	0.044	0.07
655	0.05	0.079

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
660	0.05	0.079
665	0.128	0.203
670	0.199	0.318
675	0.199	0.318
680	0.41	0.654
685	0.529	0.843
690	0.529	0.843
695	0.191	0.305
700	0.084	0.134
705	0.084	0.134
710	0.057	0.091
715	0.054	0.086
720	0.054	0.086
725	0.044	0.07
730	0.044	0.07
735	0.043	0.068
740	0.035	0.055
745	0.035	0.055
750	0.033	0.053
755	0.029	0.046
760	0.029	0.046
765	0.027	0.044
770	0.025	0.04
775	0.025	0.04
780	0.023	0.037
785	0.021	0.034
790	0.021	0.034
795	0.02	0.032
800	0.019	0.031
805	0.019	0.031
810	0.018	0.028
815	0.017	0.028
820	0.017	0.028
825	0.016	0.025

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
830	0.015	0.024
835	0.015	0.024
840	0.013	0.022
845	0.013	0.021
850	0.013	0.021
855	0.013	0.021
860	0.013	0.021
865	0.013	0.021
870	0.013	0.021
875	0.013	0.021
880	0.013	0.02
885	0.012	0.018
890	0.012	0.018
895	0.012	0.018
900	0.012	0.018
905	0.012	0.018
910	0.012	0.018
915	0.012	0.018
920	0.012	0.018
925	0.012	0.018
930	0.012	0.018
935	0.012	0.018
940	0.012	0.018
945	0.012	0.018
950	0.012	0.018
955	0.01	0.016
960	0.01	0.015
965	0.01	0.015
970	0.01	0.015
975	0.01	0.015
980	0.01	0.015
985	0.01	0.015
990	0.01	0.015
995	0.01	0.015

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
1000	0.01	0.015
1005	0.01	0.015
1010	0.01	0.015
1015	0.01	0.015
1020	0.01	0.015
1025	0.009	0.014
1030	0.008	0.012
1035	0.008	0.012
1040	0.008	0.012
1045	0.008	0.012
1050	0.008	0.012
1055	0.008	0.012
1060	0.008	0.012
1065	0.008	0.012
1070	0.008	0.012
1075	0.008	0.012
1080	0.008	0.012
1085	0.008	0.012
1090	0.008	0.012
1095	0.008	0.012
1100	0.008	0.012
1105	0.008	0.012
1110	0.006	0.009
1115	0	0
1120	0	0
1125	0.003	0.004
1130	0.008	0.012
1135	0.008	0.012
1140	0.008	0.012
1145	0.008	0.012
1150	0.008	0.012
1155	0.007	0.01
1160	0.006	0.009
1165	0.006	0.009

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
1170	0.006	0.009
1175	0.006	0.009
1180	0.006	0.009
1185	0.006	0.009
1190	0.006	0.009
1195	0.006	0.009
1200	0.006	0.009
1205	0.006	0.009
1210	0.006	0.009
1215	0.006	0.009
1220	0.006	0.009
1225	0.006	0.009
1230	0.006	0.009
1235	0.006	0.009
1240	0.006	0.009
1245	0.006	0.009
1250	0.006	0.009
1255	0.006	0.009
1260	0.006	0.009
1265	0.006	0.009
1270	0.006	0.009
1275	0.006	0.009
1280	0.006	0.009
1285	0.006	0.009
1290	0.006	0.009
1295	0.006	0.009
1300	0.006	0.009
1305	0.006	0.009
1310	0.006	0.009
1315	0.006	0.009
1320	0.006	0.009
1325	0.006	0.009
1330	0.006	0.009
1335	0.006	0.009

Simulation Time (min)	0.1 AEP Precipitation (in)	0.01 AEP Precipitation (in)
1340	0.006	0.009
1345	0.006	0.009
1350	0.006	0.009
1355	0.006	0.009
1360	0.006	0.009
1365	0.006	0.009
1370	0.006	0.009
1375	0.006	0.009
1380	0.006	0.009
1385	0.005	0.008
1390	0.004	0.006
1395	0.004	0.006
1400	0.001	0.002
1405	0	0
1410	0	0
1415	0	0
1420	0	0
1425	0	0
1430	0	0
1435	0	0
1440	0	0

M INTERIOR DRAINAGE

M.2.2 JULY 2017 HISTORIC EVENT

Table M.2.2. July 2017 Historic Event Hyetograph Data.

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 18:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:40	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
7/26/201	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7 18:49																									
7/26/2017 18:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 18:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 18:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 18:53	0	0	0.01	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0
7/26/2017 18:54	0	0	0.01	0.01	0	0	0	0	0	0	0.01	0	0.04	0	0	0	0	0	0	0	0	0	0	0.03	0
7/26/2017 18:55	0	0	0.01	0.01	0	0	0	0	0	0	0.01	0	0.05	0	0	0	0	0	0	0	0	0	0	0.02	0
7/26/2017 18:56	0.01	0	0.01	0.01	0	0	0	0	0	0	0.01	0.04	0.05	0	0	0	0	0	0	0	0	0	0	0.02	0
7/26/2017 18:57	0.01	0	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 18:58	0.01	0	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0
7/26/2017 18:59	0.02	0	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 19:00	0.02	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 19:01	0.02	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0.04	0	0	0	0.02	0
7/26/2017 19:02	0.03	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 19:03	0.03	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 19:04	0.03	0	0	0	0	0	0	0	0	0.03	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:05	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:06	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 19:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
7/26/2017 19:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
7/26/2017 19:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
7/26/2017 19:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0
7/26/2017 19:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0
7/26/2017 19:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 19:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:39	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:40	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:41	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 19:42	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/26/2017 19:43	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:44	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:45	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:46	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:47	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:48	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:49	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:54	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 19:55	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:56	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:57	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:58	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 19:59	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:00	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:02	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:03	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 20:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 20:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 20:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 20:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 21:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 21:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 21:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 21:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 22:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
7/26/2017 22:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
7/26/2017 22:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0.01	0	0	0	0	0	0
7/26/2017 22:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0.01	0	0	0.03	0	0	0
7/26/2017 22:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0.01	0.04	0	0.05	0	0	0
7/26/2017 22:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0.03	0	0.07	0	0	0
7/26/2017 22:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.05	0.07	0.03	0	0.08	0	0	0
7/26/2017 22:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.07	0.05	0.07	0	0.05	0	0	0
7/26/2017 22:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0.11	0.04	0.05	0	0.04	0	0	0
7/26/2017 22:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.03	0	0.08	0.06	0.04	0	0.05	0	0	0
7/26/2017 22:19	0	0.06	0	0	0.01	0	0	0	0	0	0	0	0	0.05	0	0.07	0	0.05	0.06	0.05	0.03	0.09	0	0	0
7/26/2017 22:20	0	0.07	0	0	0.01	0	0	0	0	0	0	0	0	0.06	0.03	0.08	0	0	0.06	0.08	0.04	0.06	0	0	0
7/26/2017 22:21	0	0.04	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.09	0.04	0.12	0	0	0.06	0.07	0.05	0.06	0	0	0
7/26/2017 22:22	0	0.05	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.09	0.06	0.06	0	0	0.1	0.03	0.04	0.05	0.09	0	0
7/26/2017 22:23	0	0.05	0	0	0.02	0.03	0.04	0	0	0	0	0.01	0.01	0.1	0.07	0.04	0.02	0	0.08	0.05	0.12	0.02	0.07	0	0
7/26/2017 22:24	0	0.05	0	0	0.02	0.04	0.04	0	0	0	0	0.01	0.01	0.08	0.08	0	0.02	0	0.01	0.01	0.04	0.02	0.08	0	0
7/26/2017 22:25	0	0.08	0	0	0.02	0.04	0.04	0	0	0	0	0.01	0.01	0.06	0.06	0	0.04	0	0.01	0.01	0.04	0.03	0.04	0	0
7/26/2017 22:26	0	0.07	0	0.03	0.02	0.04	0.09	0.07	0.06	0	0.02	0.01	0.01	0.01	0.02	0	0.05	0	0.01	0.01	0.09	0.01	0.06	0	0
7/26/2017 22:27	0	0.06	0.01	0.03	0.02	0.03	0.04	0.07	0.06	0	0.02	0.01	0.01	0.01	0.02	0	0.03	0	0.01	0.01	0.04	0.01	0.09	0	0
7/26/2017 22:28	0	0.04	0.01	0.03	0.02	0.03	0.04	0.05	0.05	0	0.02	0.01	0.01	0.01	0	0	0.03	0	0.01	0	0.04	0.01	0.1	0.01	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 22:29	0	0.05	0.01	0.02	0.03	0.03	0.09	0.08	0.06	0	0.05	0.01	0.01	0	0	0	0.02	0	0	0	0.04	0.01	0.06	0.01	0
7/26/2017 22:30	0	0.04	0.01	0.02	0.03	0.03	0.04	0.05	0.07	0.02	0.07	0.02	0.01	0	0	0	0.02	0	0	0	0.03	0.01	0.05	0.01	0
7/26/2017 22:31	0	0.03	0.03	0.02	0.03	0.04	0.04	0.08	0.07	0.02	0.08	0.02	0.01	0	0	0	0	0	0	0	0.03	0.01	0.06	0.02	0
7/26/2017 22:32	0	0.01	0.03	0.04	0.04	0.04	0.04	0.03	0.06	0.02	0.08	0.05	0.01	0	0	0	0	0	0	0	0.04	0.01	0.05	0.05	0
7/26/2017 22:33	0	0.01	0.03	0.05	0.09	0.1	0.04	0.04	0.03	0.03	0.05	0.09	0.04	0	0	0	0	0	0	0	0	0	0.07	0.04	0
7/26/2017 22:34	0.02	0.01	0.04	0.08	0.09	0.04	0.04	0.04	0.04	0.03	0.02	0.06	0.03	0	0	0	0	0	0	0	0	0	0.03	0.05	0
7/26/2017 22:35	0.02	0.01	0.03	0.07	0.08	0.09	0.13	0.05	0.04	0.03	0.02	0.06	0.04	0	0	0	0	0	0	0	0	0	0.02	0.03	0
7/26/2017 22:36	0.02	0.01	0	0.23	0.09	0.04	0.04	0.06	0.04	0.02	0.01	0.03	0.06	0	0	0	0	0	0	0	0	0	0.02	0.01	0
7/26/2017 22:37	0.02	0.01	0	0.04	0.1	0.02	0.04	0.05	0.07	0.02	0.01	0.03	0.06	0	0	0	0	0	0	0	0	0	0.02	0	0
7/26/2017 22:38	0.03	0.01	0	0.04	0.08	0.02	0.04	0.07	0.05	0.05	0.01	0.02	0.08	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:39	0	0.01	0	0	0.01	0.02	0.01	0.05	0.04	0.08	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:40	0	0.01	0	0	0.01	0.02	0.01	0.05	0.04	0.08	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:41	0	0.01	0	0	0.01	0	0.01	0.01	0.09	0.1	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:42	0.03	0.01	0	0	0	0	0.01	0.01	0.06	0.07	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:43	0	0.01	0	0	0	0	0.01	0.01	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:44	0	0.01	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:45	0	0.01	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:46	0	0.01	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/26/2017 22:47	0	0.01	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:48	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:49	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:50	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 22:51	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 22:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 23:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 23:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/26/2017 23:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/26/2017 23:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/26/2017 23:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/26/2017 23:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/26/2017 23:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/26/2017 23:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03
7/26/2017 23:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04
7/26/2017 23:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.04	0	0	0	0	0	0	0.03
7/26/2017 23:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.03	0	0	0	0	0	0	0.02
7/26/2017 23:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0.02
7/26/2017 23:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.08	0	0.01	0	0	0	0	0	0	0
7/26/2017 23:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.04	0	0.01	0	0	0	0	0	0	0
7/26/2017 23:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0	0.01	0	0	0	0	0	0	0
7/26/2017 23:51	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0.03	0.05	0.03	0	0	0	0	0	0	0	0	0
7/26/2017 23:52	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0.03	0.03	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:53	0	0	0	0.03	0	0	0	0.03	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
7/26/2017 23:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/26/2017 23:57	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0
7/26/2017 23:58	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0.02	0
7/26/2017 23:59	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0.02	0	0	0	0	0	0	0.02	0
7/27/2017 0:00	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0.02	0	0	0	0	0	0	0.03	0
7/27/2017 0:01	0	0	0	0	0	0	0	0	0	0	0.03	0	0.04	0	0	0	0	0	0	0	0	0	0	0.02	0
7/27/2017 0:02	0	0	0	0	0	0	0	0	0	0	0.03	0.04	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 0:03	0	0	0	0	0.04	0	0	0	0	0	0.05	0.04	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 0:04	0.01	0	0.01	0.02	0.02	0	0	0	0	0	0	0.04	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 0:05	0.02	0	0.01	0.02	0.01	0	0	0	0	0	0	0.03	0.03	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:06	0.02	0	0.01	0.01	0.01	0	0	0	0	0	0	0.02	0.03	0	0	0	0	0	0	0	0	0	0	0	0.04
7/27/2017 0:07	0.01	0	0	0.01	0.01	0	0	0	0	0.03	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.03
7/27/2017 0:08	0.01	0	0	0.01	0.01	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03
7/27/2017 0:09	0.01	0	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:10	0.01	0	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:11	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:12	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:13	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:14	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 0:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 0:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 0:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 0:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:30	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:31	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:32	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:33	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 0:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 0:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 1:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 1:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/27/2017 1:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/27/2017 1:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/27/2017 1:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.02
7/27/2017 1:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.02
7/27/2017 1:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.02
7/27/2017 1:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.02	0	0	0	0	0	0	0.02
7/27/2017 1:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.02	0	0	0	0	0	0	0
7/27/2017 1:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.02	0.01	0	0	0	0	0	0
7/27/2017 1:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0	0.02	0.01	0	0	0	0	0	0
7/27/2017 1:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0	0.02	0.01	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 1:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0	0.03	0.01	0	0	0	0	0	0
7/27/2017 1:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0	0	0.02	0	0	0	0	0	0
7/27/2017 1:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0.01	0.01	0	0	0.02	0	0	0	0	0	0
7/27/2017 1:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0.01	0	0	0.03	0	0	0	0	0	0
7/27/2017 1:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.01	0	0	0.05	0.02	0	0	0	0	0
7/27/2017 1:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0.01	0	0	0.04	0.02	0	0	0	0	0
7/27/2017 1:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0.02	0.04	0	0.03	0	0	0
7/27/2017 1:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0.02	0.05	0	0.04	0	0	0
7/27/2017 1:33	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0.01	0.05	0	0.05	0	0	0
7/27/2017 1:34	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0.01	0.04	0	0.04	0	0	0
7/27/2017 1:35	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0.01	0	0	0.05	0	0	0
7/27/2017 1:36	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0.01	0.05	0	0.03	0	0	0
7/27/2017 1:37	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.03	0.01	0	0	0
7/27/2017 1:38	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0.01	0.01	0.04	0.01	0	0	0
7/27/2017 1:39	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0.01	0.01	0.03	0.01	0	0	0
7/27/2017 1:40	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0.01	0.01	0.03	0.01	0	0	0
7/27/2017 1:41	0	0.01	0.01	0.01	0	0.02	0	0	0	0	0	0	0.01	0	0	0	0	0.02	0.01	0.01	0.04	0.02	0	0	0
7/27/2017 1:42	0	0.01	0.01	0.01	0.02	0.02	0	0	0	0	0	0	0.01	0	0	0	0	0.02	0.01	0.01	0.03	0.02	0	0	0
7/27/2017 1:43	0	0.01	0.01	0.01	0.02	0.02	0.03	0	0.01	0	0	0	0.01	0	0	0	0	0.02	0.01	0.01	0.03	0.02	0	0	0
7/27/2017 1:44	0	0.01	0	0.01	0.02	0.02	0.02	0	0.01	0	0	0	0.01	0	0	0	0	0.02	0.01	0.01	0.02	0.02	0.04	0	0
7/27/2017 1:45	0	0.03	0	0.01	0.01	0.01	0.02	0	0.01	0	0	0	0	0	0	0.01	0	0.02	0.01	0.01	0.02	0.02	0.05	0	0
7/27/2017 1:46	0	0.05	0	0.01	0.01	0.01	0.02	0.03	0.01	0	0	0	0	0.01	0	0.01	0	0.02	0.01	0.01	0.01	0.02	0.04	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 1:47	0	0.05	0	0.01	0.01	0.01	0.02	0.02	0.01	0	0	0	0	0.01	0	0.01	0	0.01	0.03	0.01	0.01	0.02	0.03	0	0.02
7/27/2017 1:48	0	0.06	0	0.01	0.01	0.01	0.03	0.02	0.04	0	0	0	0	0.01	0.01	0.02	0	0.01	0.03	0.05	0.01	0.06	0.03	0	0.02
7/27/2017 1:49	0	0.03	0	0.01	0.01	0.01	0.02	0.02	0.05	0	0	0	0	0.02	0.01	0.02	0	0.01	0.03	0.04	0.01	0.06	0.03	0	0.01
7/27/2017 1:50	0	0.03	0	0.01	0.01	0.01	0.02	0.02	0.03	0	0	0	0	0.02	0.01	0.02	0.02	0.01	0.04	0.04	0.03	0.05	0.03	0	0.01
7/27/2017 1:51	0	0.03	0	0.01	0.01	0.01	0.02	0.02	0.02	0	0	0	0	0.02	0.02	0.02	0.02	0.01	0.04	0.05	0.01	0.05	0.02	0	0.01
7/27/2017 1:52	0	0.03	0	0.01	0.01	0.01	0.02	0.03	0.02	0.01	0	0	0	0.02	0.02	0.01	0.02	0.01	0.02	0.04	0.01	0.06	0.02	0	0.01
7/27/2017 1:53	0	0.03	0	0.01	0.01	0.01	0.01	0.02	0.04	0.01	0	0	0	0.02	0.02	0.01	0.02	0.01	0.02	0.04	0.01	0.05	0.02	0	0.01
7/27/2017 1:54	0	0.05	0	0.01	0.01	0.01	0.01	0.02	0.03	0.01	0	0	0	0.02	0.02	0.01	0.02	0.01	0.02	0	0.04	0.05	0.03	0	0.01
7/27/2017 1:55	0	0.02	0.03	0.01	0.01	0.02	0.01	0.04	0.04	0.01	0.04	0	0.02	0.01	0.01	0.01	0.03	0.01	0.02	0.04	0.04	0.04	0.04	0.02	0.01
7/27/2017 1:56	0	0.07	0.03	0.03	0.01	0.02	0.02	0.02	0.04	0.02	0.04	0	0.02	0.01	0.01	0.01	0.03	0.01	0.02	0.02	0.08	0.02	0.04	0.02	0.01
7/27/2017 1:57	0	0.05	0.01	0.03	0.05	0.02	0.02	0.02	0.05	0.02	0.02	0	0.04	0.01	0.01	0.01	0.03	0.01	0.02	0.02	0.05	0.01	0.05	0.01	0.02
7/27/2017 1:58	0.01	0.04	0.01	0.03	0.05	0.04	0.04	0.02	0.03	0.02	0.02	0.03	0.05	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.01	0.04	0.01	0.02
7/27/2017 1:59	0.03	0.04	0.01	0.02	0.05	0.04	0.04	0.02	0.04	0.02	0.02	0.04	0.03	0.01	0.01	0.01	0.01	0.09	0.01	0.01	0.07	0.01	0.09	0.01	0.05
7/27/2017 2:00	0.04	0.04	0.07	0.02	0.04	0.04	0.04	0.05	0.02	0.02	0.02	0.03	0.04	0.01	0.01	0.01	0.01	0.04	0.01	0.01	0.05	0.02	0.06	0.02	0.05
7/27/2017 2:01	0.03	0.06	0.05	0.05	0.04	0.04	0.04	0.05	0.02	0.06	0.03	0.03	0.04	0.01	0.01	0.01	0.01	0.09	0.01	0.02	0.04	0.02	0.05	0.01	0.04
7/27/2017 2:02	0.03	0.03	0.05	0.07	0.04	0.1	0.04	0.06	0.09	0.06	0.02	0.04	0.02	0.01	0.01	0.06	0.01	0.13	0.05	0.02	0.03	0.02	0.05	0.01	0.03
7/27/2017 2:03	0.03	0.04	0.05	0.05	0.06	0.04	0.04	0.06	0.06	0.05	0.02	0.03	0.02	0.01	0.1	0.05	0.01	0.11	0.1	0.09	0.03	0.04	0.06	0.01	0.05
7/27/2017 2:04	0.02	0.01	0.04	0.07	0.05	0.04	0.08	0.06	0.07	0.06	0.02	0.02	0.03	0.01	0.03	0.06	0.01	0.11	0.1	0.09	0.03	0.09	0.06	0.01	0.06
7/27/2017 2:05	0.03	0.01	0.02	0.05	0.04	0.04	0.04	0.07	0.06	0.06	0.01	0.02	0.05	0.09	0.04	0.03	0.01	0.06	0.05	0.1	0	0.1	0.11	0.01	0.07
7/27/2017 2:06	0.01	0.01	0.02	0.04	0.04	0.04	0.04	0.06	0.06	0.05	0.01	0.02	0.04	0.05	0.04	0.05	0.01	0.05	0.08	0.09	0	0.11	0.04	0	0.05
7/27/2017 2:07	0.01	0.02	0.04	0.03	0.03	0.03	0.04	0.05	0.06	0.05	0.01	0.01	0.03	0.04	0.05	0.1	0.01	0.05	0.11	0.05	0.03	0.1	0.02	0	0.06
7/27/2017 2:08	0.02	0.02	0.03	0.04	0.03	0.02	0.04	0.05	0.08	0.06	0.01	0.01	0.02	0.03	0.08	0.1	0.01	0.03	0.1	0.06	0.03	0.08	0.03	0	0.06

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 2:09	0.01	0.02	0.02	0.03	0.02	0.02	0.04	0.05	0.04	0.05	0.04	0.02	0.02	0.11	0.1	0.1	0.08	0.03	0.07	0.05	0.03	0.07	0.03	0	0.06
7/27/2017 2:10	0.01	0.05	0.02	0.03	0.02	0.02	0.01	0.04	0.05	0.04	0.04	0.02	0.07	0.12	0.07	0.05	0.06	0.01	0.05	0.03	0.04	0.05	0.03	0	0.06
7/27/2017 2:11	0.01	0.05	0.02	0.02	0.02	0.02	0.01	0.03	0.04	0.03	0.07	0.02	0.16	0.14	0.03	0.04	0.04	0.01	0.05	0.03	0.09	0.02	0.03	0.1	0.04
7/27/2017 2:12	0.01	0.05	0.02	0.02	0.02	0.02	0.01	0.02	0.04	0.03	0.04	0.05	0.1	0.13	0.05	0	0.05	0.01	0.06	0.03	0.08	0.03	0.03	0.07	0.03
7/27/2017 2:13	0.04	0.04	0.1	0.04	0.02	0.04	0.04	0.02	0.04	0.03	0.05	0.1	0.09	0.12	0.06	0.07	0.06	0.01	0.03	0.04	0.08	0.05	0.05	0.05	0.05
7/27/2017 2:14	0.05	0.02	0.06	0.08	0.04	0.02	0.04	0.02	0.03	0.03	0.09	0.05	0.09	0.04	0.05	0.06	0.12	0.02	0.01	0.03	0.1	0.06	0.11	0.04	0.09
7/27/2017 2:15	0.05	0.02	0.06	0.1	0.09	0.02	0.18	0.02	0.03	0.06	0.11	0.09	0.09	0.06	0.03	0.05	0.07	0.02	0.01	0	0.1	0.03	0.11	0.05	0.12
7/27/2017 2:16	0.09	0.02	0.07	0.05	0.15	0.02	0.15	0.05	0.03	0.06	0.06	0.11	0.05	0.03	0.04	0.03	0.05	0.01	0.01	0	0.04	0.02	0.11	0.06	0.1
7/27/2017 2:17	0.05	0.02	0.1	0.02	0.05	0.02	0.15	0.05	0.04	0.04	0.05	0.08	0.05	0.02	0.04	0	0.06	0.01	0.01	0	0.03	0.02	0.11	0.04	0.04
7/27/2017 2:18	0.05	0.01	0.07	0.08	0.05	0.07	0.18	0.1	0.04	0.04	0.05	0.04	0.04	0.02	0.03	0.03	0.03	0.01	0.01	0	0.05	0.01	0.06	0.04	0.04
7/27/2017 2:19	0.05	0.01	0.04	0.08	0.05	0.07	0.15	0.21	0.06	0.07	0.06	0.05	0.04	0.01	0.04	0.03	0.01	0.01	0.01	0	0.04	0.01	0.05	0.03	0.02
7/27/2017 2:20	0.05	0.01	0.04	0.05	0.05	0.04	0.11	0.21	0.09	0.1	0.04	0.05	0.01	0.01	0.04	0.03	0.01	0.01	0.01	0	0.04	0.01	0.03	0.04	0.02
7/27/2017 2:21	0.05	0.02	0.04	0.03	0.04	0.03	0.04	0.1	0.12	0.1	0.02	0.04	0.01	0.01	0.04	0.02	0.01	0.01	0.01	0	0.05	0.01	0.02	0.03	0.01
7/27/2017 2:22	0.05	0.02	0.03	0.03	0.03	0.04	0.09	0.04	0.08	0.16	0.02	0.03	0.01	0.02	0.04	0.02	0.01	0.01	0.01	0	0.04	0.01	0.02	0.03	0.01
7/27/2017 2:23	0.05	0.03	0.01	0.04	0.03	0.04	0.04	0.04	0.08	0.18	0.06	0.03	0.01	0.02	0.03	0.02	0.01	0.01	0.01	0	0.04	0.02	0.02	0.04	0.01
7/27/2017 2:24	0.05	0.01	0.01	0.03	0.05	0.04	0.03	0.04	0.04	0.08	0.03	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.03	0.01
7/27/2017 2:25	0.03	0.01	0.01	0.03	0.04	0	0.03	0.05	0.04	0.1	0.05	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.04	0.01
7/27/2017 2:26	0.03	0.01	0.01	0.01	0.01	0	0.01	0.03	0.03	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.01
7/27/2017 2:27	0.03	0.01	0.01	0.01	0.01	0	0.01	0.03	0.05	0.03	0.03	0.02	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.03	0.01	0.01	0.02	0.04	0.01
7/27/2017 2:28	0.03	0.02	0.01	0.01	0.01	0	0.01	0.03	0.05	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.01	0.03	0.02	0.03	0.01
7/27/2017 2:29	0.03	0.02	0.01	0.01	0.01	0	0.01	0.03	0.04	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.01	0.03	0.02	0.05	0.01
7/27/2017 2:30	0.06	0.02	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.04	0.01	0.03	0.03	0.07	0.01

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 2:31	0.03	0.02	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.03	0.01	0.04	0.02	0.05	0.01
7/27/2017 2:32	0.02	0.02	0.02	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.03	0.03	0.01	0.01	0.01	0.01	0.03	0.01	0.02	0.01	0.03	0.02	0.05	0.03
7/27/2017 2:33	0.03	0.02	0.02	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.02	0.01	0.07	0.01	0.02	0.01	0.03	0.02	0.04	0.02
7/27/2017 2:34	0.03	0.02	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.02	0.03	0.01	0.07	0.02	0.03	0.01	0.03	0.02	0.01	0.02
7/27/2017 2:35	0.03	0.03	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.01	0.08	0.02	0.03	0.03	0.01	0.02	0.01	0
7/27/2017 2:36	0.04	0.03	0.01	0.01	0.01	0	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.01	0.05	0.04	0.03	0.04	0.01	0.02	0.01	0
7/27/2017 2:37	0.03	0.04	0.01	0.01	0.01	0	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.02	0.03	0.03	0.01	0.03	0.07	0.06	0.04	0.01	0.02	0.01	0
7/27/2017 2:38	0.02	0.05	0.01	0.01	0.01	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.03	0.03	0.01	0.01	0.06	0.07	0.04	0.02	0.02	0.01	0
7/27/2017 2:39	0.03	0.04	0.01	0.01	0.01	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.03	0.03	0.02	0.01	0.07	0.08	0.03	0.02	0.03	0.01	0
7/27/2017 2:40	0.01	0.06	0.01	0.01	0.01	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.03	0.08	0.02	0.01	0.07	0.09	0.03	0.01	0.02	0	0
7/27/2017 2:41	0	0.05	0.01	0.01	0.01	0	0.02	0.02	0.01	0.01	0.02	0.01	0.01	0.03	0.05	0.07	0.06	0.02	0.05	0.03	0.03	0.01	0.02	0	0
7/27/2017 2:42	0	0.01	0.01	0.01	0.01	0	0.01	0.02	0.02	0.01	0.02	0.01	0.01	0.07	0.04	0.07	0.06	0.02	0.02	0.03	0.05	0.01	0.03	0	0
7/27/2017 2:43	0	0.01	0.01	0.01	0.01	0	0.01	0.03	0.02	0.01	0.02	0.01	0.01	0.05	0.02	0.06	0.06	0.04	0.02	0.01	0.04	0.01	0.03	0.01	0
7/27/2017 2:44	0	0.01	0.01	0.01	0.01	0	0.01	0.02	0.02	0.01	0.02	0.01	0.01	0.04	0.02	0.05	0.06	0.04	0.01	0.01	0.04	0.01	0.04	0.03	0
7/27/2017 2:45	0	0.01	0.01	0.01	0.01	0	0.03	0.02	0.02	0.01	0.02	0.01	0.01	0.04	0.02	0.05	0.08	0.05	0.01	0.01	0.04	0.01	0.04	0.01	0
7/27/2017 2:46	0.01	0.01	0.02	0.02	0.03	0.04	0.04	0.02	0.02	0.01	0.01	0.01	0.03	0.06	0.02	0	0.04	0.07	0.01	0.01	0.02	0.01	0.01	0.01	0
7/27/2017 2:47	0.02	0.01	0.02	0.02	0.04	0.04	0.04	0.02	0.02	0.01	0.01	0.01	0.03	0.05	0.01	0	0.03	0.08	0.02	0.01	0.02	0.01	0.01	0.01	0
7/27/2017 2:48	0.02	0.01	0.07	0.05	0.04	0.04	0.04	0.01	0.03	0.01	0.01	0.01	0.03	0.05	0.01	0.02	0.01	0.09	0.02	0.01	0.02	0.02	0.01	0	0.01
7/27/2017 2:49	0.01	0.01	0.07	0.09	0.05	0.04	0.04	0.01	0.03	0.01	0.01	0.01	0.04	0.06	0.01	0.02	0.01	0.08	0.02	0.01	0.02	0.02	0	0	0.01
7/27/2017 2:50	0	0.01	0.06	0.08	0.06	0.08	0.03	0.01	0.03	0.03	0.01	0.01	0.02	0.06	0.01	0.03	0.01	0.09	0.01	0.01	0.02	0.02	0	0	0.01
7/27/2017 2:51	0	0.01	0.03	0.05	0.06	0.02	0.04	0.01	0.03	0.03	0.01	0.01	0.02	0.06	0.02	0.03	0.01	0.1	0.01	0.03	0.01	0.02	0	0	0.04
7/27/2017 2:52	0	0.01	0.02	0.03	0.03	0.02	0.02	0.01	0.03	0.03	0.01	0.01	0.03	0.08	0.02	0.09	0.02	0.07	0.04	0.03	0.01	0.02	0	0	0.04

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 2:53	0	0.01	0.02	0.05	0.02	0.01	0.02	0.01	0.02	0.06	0.01	0.01	0.02	0.07	0.09	0.09	0.02	0.04	0.05	0.03	0.01	0.03	0	0	0.01
7/27/2017 2:54	0	0.01	0.01	0.02	0.02	0.01	0.02	0.01	0.02	0.05	0.01	0.01	0.02	0.09	0.08	0.08	0.03	0.04	0.05	0.03	0.01	0.04	0	0	0.01
7/27/2017 2:55	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0	0.05	0.01	0.01	0.03	0.09	0.08	0.19	0.05	0.05	0.07	0.07	0.01	0.02	0	0	0.01
7/27/2017 2:56	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0.03	0.01	0.02	0.02	0.09	0.05	0.1	0.05	0.1	0.1	0.08	0.02	0.02	0	0	0.03
7/27/2017 2:57	0	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0.02	0.01	0.02	0.02	0.08	0.07	0.13	0.07	0.1	0.12	0.09	0.02	0.07	0	0	0.07
7/27/2017 2:58	0	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0.02	0.02	0.01	0.03	0.1	0.08	0.11	0.06	0.1	0.08	0.08	0.02	0.06	0	0	0.09
7/27/2017 2:59	0	0.04	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0.04	0.01	0.03	0.1	0.05	0.07	0.1	0.1	0.06	0.08	0.03	0.04	0.03	0	0.06
7/27/2017 3:00	0	0.03	0.03	0.02	0.01	0.01	0.01	0	0	0.01	0.06	0.04	0.04	0.06	0.04	0.07	0.07	0.08	0.08	0.05	0.03	0.05	0.03	0.01	0.02
7/27/2017 3:01	0	0.04	0.03	0.02	0.01	0.01	0.01	0	0	0.01	0.07	0.05	0.05	0.04	0.05	0.07	0.09	0.05	0.08	0.05	0.03	0.06	0.02	0.01	0.01
7/27/2017 3:02	0.02	0.03	0.03	0.02	0.01	0.01	0.01	0	0	0.01	0.08	0.05	0.05	0.05	0.07	0.1	0.06	0.02	0.09	0.05	0.03	0.06	0.02	0.02	0.01
7/27/2017 3:03	0.02	0.04	0.03	0.02	0.01	0.04	0.01	0	0	0.01	0.09	0.07	0.05	0.06	0.06	0.09	0.06	0.02	0.06	0.04	0.05	0.04	0.02	0.05	0.01
7/27/2017 3:04	0	0.02	0.05	0.02	0.04	0.04	0.03	0	0	0.01	0.09	0.07	0.07	0.12	0.07	0.08	0.04	0.02	0.06	0.03	0.04	0.02	0.02	0.04	0.01
7/27/2017 3:05	0	0.02	0.06	0.02	0.04	0.11	0.04	0	0.02	0.01	0.07	0.06	0.06	0.13	0.08	0.08	0.04	0.01	0.05	0.04	0.03	0.02	0.03	0.02	0.01
7/27/2017 3:06	0	0.02	0.07	0.05	0.06	0.09	0.04	0.02	0.02	0.01	0.1	0.08	0.09	0.1	0.07	0.06	0.05	0.01	0.06	0	0.12	0.02	0.03	0.02	0.01
7/27/2017 3:07	0	0.02	0.1	0.06	0.06	0.13	0.1	0.04	0.03	0.01	0.1	0.11	0.07	0.09	0.05	0.04	0.08	0.01	0.05	0.02	0.04	0.02	0.03	0.01	0.01
7/27/2017 3:08	0.05	0.02	0.06	0.07	0.09	0.11	0.04	0.03	0.03	0.01	0.07	0.07	0.08	0.05	0.05	0.04	0.06	0.01	0.04	0.02	0.03	0.03	0.04	0.02	0.01
7/27/2017 3:09	0.05	0.02	0.06	0.05	0.1	0.04	0.15	0.03	0.02	0.01	0.08	0.07	0.09	0.04	0.06	0.11	0.07	0.01	0.02	0.02	0.03	0.03	0.06	0.01	0.13
7/27/2017 3:10	0.04	0.02	0.07	0.05	0.05	0.08	0.09	0.03	0.02	0.02	0.08	0.09	0.11	0.01	0.06	0.1	0.08	0.01	0.02	0.02	0.03	0.03	0.04	0.01	0.13
7/27/2017 3:11	0.02	0.02	0.05	0.05	0.05	0.04	0.09	0.07	0.03	0.02	0.08	0.09	0.07	0.01	0.08	0.06	0.06	0.01	0.02	0.02	0.04	0.04	0.03	0.02	0.12
7/27/2017 3:12	0.02	0.03	0.07	0.22	0.06	0.04	0.04	0.07	0.05	0.04	0.09	0.08	0.05	0.01	0.06	0.05	0.05	0.03	0.02	0.03	0.02	0.02	0.04	0.01	0.08
7/27/2017 3:13	0.02	0.03	0.1	0.09	0.07	0.04	0.04	0.03	0.04	0.05	0.07	0.09	0.07	0.01	0.09	0.02	0.03	0.04	0.02	0.04	0.02	0.05	0.05	0.02	0.03
7/27/2017 3:14	0.06	0.04	0.14	0.1	0.03	0.04	0.04	0.04	0.07	0.07	0.05	0.08	0.08	0.01	0.05	0.02	0.03	0.03	0.02	0.04	0.03	0.03	0.04	0.04	0.07

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 3:15	0.06	0.04	0.08	0.14	0.03	0.04	0.04	0.04	0.04	0.05	0.08	0.08	0.08	0.01	0.04	0.06	0.02	0.03	0.02	0.03	0.03	0.03	0.02	0.05	0.07
7/27/2017 3:16	0.04	0.02	0.06	0.13	0.03	0.04	0.04	0.04	0.05	0.05	0.08	0.05	0.09	0.01	0.03	0.04	0.02	0.02	0.02	0.02	0.03	0.03	0.02	0.09	0.03
7/27/2017 3:17	0.04	0.02	0.1	0.06	0.03	0.04	0.04	0.05	0.06	0.05	0.06	0.06	0.06	0.01	0.03	0.01	0.04	0.02	0.02	0.02	0.04	0.02	0.03	0.03	0.1
7/27/2017 3:18	0.06	0.02	0.1	0.03	0.03	0.02	0.04	0.03	0.03	0.05	0.04	0.06	0.08	0.01	0.01	0.01	0.03	0.01	0.02	0.02	0.03	0.02	0.05	0.01	0.09
7/27/2017 3:19	0.06	0.03	0.09	0.12	0.03	0.02	0.02	0.02	0.03	0.04	0.05	0.06	0.06	0.01	0.01	0.01	0.01	0.01	0.03	0.02	0.03	0.03	0.04	0.03	0.07
7/27/2017 3:20	0.08	0.03	0.07	0.26	0.02	0.02	0.02	0.02	0.02	0.04	0.04	0.06	0.02	0.01	0.01	0.02	0.01	0.01	0.04	0.02	0.04	0.03	0.04	0.03	0.06
7/27/2017 3:21	0	0.03	0.04	0.05	0.02	0.02	0.02	0.02	0.02	0.06	0.06	0.05	0.02	0.02	0.01	0.02	0.01	0	0.03	0.02	0.03	0.03	0.04	0.03	0.05
7/27/2017 3:22	0	0.02	0.03	0.04	0.03	0.04	0.02	0.02	0.02	0.04	0.02	0.04	0.01	0.02	0.01	0.03	0.01	0	0.04	0.03	0.03	0.04	0.02	0.04	0.05
7/27/2017 3:23	0	0.03	0.02	0.03	0.03	0.04	0.02	0.04	0.02	0.04	0.02	0.04	0.01	0.01	0.01	0.03	0.01	0	0.04	0.02	0.03	0.03	0.02	0.03	0.05
7/27/2017 3:24	0	0.03	0.02	0.02	0.05	0.04	0.02	0.03	0.02	0.03	0.16	0.03	0.01	0.01	0.02	0.02	0.03	0	0.03	0.02	0.02	0.04	0.02	0.03	0.02
7/27/2017 3:25	0	0.05	0.01	0.02	0.05	0.04	0.03	0.03	0.02	0.05	0.01	0.03	0.01	0.01	0.02	0.02	0.02	0	0.01	0.01	0.02	0.05	0.02	0.04	0.02
7/27/2017 3:26	0	0.04	0.01	0.04	0.05	0.02	0.03	0.03	0.03	0.05	0.01	0.02	0.01	0.01	0.02	0.02	0.02	0	0.01	0.01	0.02	0.03	0.02	0.04	0.02
7/27/2017 3:27	0	0.05	0.01	0.03	0.02	0.02	0.02	0.02	0.03	0.02	0.01	0.02	0.03	0.01	0.01	0.02	0.03	0	0.01	0.01	0.02	0.02	0.02	0.04	0.02
7/27/2017 3:28	0	0.05	0.02	0.03	0.02	0.03	0.02	0.02	0.03	0.02	0.01	0.01	0.03	0.01	0.01	0.02	0.01	0	0.01	0.01	0.02	0.02	0.02	0.03	0.03
7/27/2017 3:29	0	0.03	0.02	0.03	0.04	0.03	0.02	0.02	0.02	0.03	0.01	0.01	0.03	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.02	0.05	0.03	0.05
7/27/2017 3:30	0.02	0.05	0.04	0.03	0.05	0.02	0.02	0.02	0.02	0.03	0.01	0.01	0.01	0	0.01	0.01	0.01	0	0.01	0.01	0.04	0.01	0.06	0.05	0.08
7/27/2017 3:31	0.01	0.03	0.03	0.03	0.04	0.02	0.02	0.03	0.02	0.03	0.01	0.01	0.01	0	0.02	0.01	0.01	0	0.01	0.01	0.03	0.01	0.06	0.05	0.05
7/27/2017 3:32	0.01	0.01	0.02	0.02	0.02	0.03	0.02	0.03	0.02	0.04	0.02	0.01	0.01	0	0.02	0.05	0.01	0	0.01	0.01	0.02	0.01	0.05	0.03	0.06
7/27/2017 3:33	0.01	0.01	0.02	0.02	0.02	0.03	0.02	0.03	0.02	0.02	0.02	0.01	0.01	0	0.03	0.06	0.01	0	0.01	0.01	0.02	0.01	0.04	0.03	0.03
7/27/2017 3:34	0.01	0.01	0.02	0.04	0.03	0.02	0.03	0.03	0.02	0.02	0.01	0.01	0.03	0.02	0.05	0.06	0.01	0	0.01	0.01	0.02	0.01	0.03	0.01	0.01
7/27/2017 3:35	0.01	0.01	0.02	0.03	0.02	0.02	0.03	0.04	0.03	0.02	0.01	0.01	0.02	0.02	0.05	0.01	0.01	0	0.01	0.01	0.01	0.01	0.03	0.01	0.01
7/27/2017 3:36	0.01	0.01	0.02	0.06	0.02	0.01	0.03	0.03	0.03	0.02	0.01	0.01	0.02	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.03	0.01	0.01

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 3:37	0.01	0.01	0	0.01	0.01	0.01	0.03	0.03	0.03	0.02	0.01	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.03	0.01	0.01
7/27/2017 3:38	0.01	0.01	0	0.01	0.01	0.01	0.03	0.03	0.03	0.02	0.01	0.01	0	0	0	0.01	0.01	0.04	0.01	0.01	0.01	0.02	0.01	0.03	0.01
7/27/2017 3:39	0.01	0.01	0	0.01	0.01	0.01	0.03	0.03	0.03	0.02	0.01	0.01	0	0	0	0.01	0	0.05	0.01	0.01	0.01	0.01	0.01	0.04	0.04
7/27/2017 3:40	0.01	0.03	0	0.01	0.01	0.01	0.01	0.03	0.01	0.02	0.01	0.01	0	0	0	0.01	0	0.05	0.01	0.01	0.01	0.01	0.01	0.02	0.07
7/27/2017 3:41	0.01	0.03	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0.01	0	0.04	0.01	0.01	0.01	0.01	0.01	0.03	0.08
7/27/2017 3:42	0.01	0.03	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0	0	0	0.01	0	0.03	0.04	0.01	0.01	0.01	0.01	0.02	0.04
7/27/2017 3:43	0.01	0.05	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0	0	0	0.01	0	0.02	0.03	0.01	0.02	0.01	0.02	0.01	0.02
7/27/2017 3:44	0.01	0.05	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0	0	0.01	0.01	0	0.02	0.02	0.01	0.02	0.01	0.02	0.01	0.02
7/27/2017 3:45	0.01	0.03	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0	0	0.01	0.01	0	0.05	0.02	0.01	0.03	0.01	0.02	0	0.13
7/27/2017 3:46	0.03	0.03	0	0	0.01	0.01	0.01	0.02	0.01	0.01	0	0.01	0	0	0.01	0.01	0	0.05	0.03	0.01	0.02	0.01	0.02	0	0.01
7/27/2017 3:47	0.03	0.02	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0	0.01	0	0	0.01	0.03	0	0.06	0.03	0.01	0.02	0.01	0.02	0	0.01
7/27/2017 3:48	0.03	0.02	0.01	0	0.01	0.01	0.03	0.01	0.03	0.01	0	0.01	0	0.01	0.01	0.06	0	0.04	0.03	0.01	0.01	0.01	0.02	0	0.02
7/27/2017 3:49	0	0.01	0.01	0	0.03	0.01	0.02	0.01	0.03	0.01	0	0.01	0	0.01	0.03	0.02	0.04	0.01	0.03	0.03	0.01	0.01	0.02	0	0.02
7/27/2017 3:50	0	0.01	0.01	0	0.02	0.01	0.02	0.01	0.03	0.01	0	0.01	0	0.01	0.03	0.02	0.05	0.01	0.02	0.03	0.01	0.01	0.02	0.04	0.02
7/27/2017 3:51	0	0.01	0.01	0	0.02	0.01	0.01	0.01	0.03	0.01	0.01	0.01	0	0.01	0.02	0.01	0.04	0.01	0.02	0.01	0.01	0.01	0.02	0.1	0.02
7/27/2017 3:52	0	0.02	0.01	0.02	0.02	0.01	0.01	0.01	0.03	0.01	0.01	0.01	0.03	0.01	0.02	0.01	0.05	0.01	0	0.01	0.01	0.01	0.02	0.07	0.02
7/27/2017 3:53	0	0.02	0.04	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.08	0.01	0.01	0.01	0.03	0.01	0	0.01	0.01	0.01	0.02	0.04	0.03
7/27/2017 3:54	0	0.02	0.05	0.05	0.01	0.01	0.01	0.01	0.01	0.04	0.03	0.04	0.08	0.01	0.01	0.01	0.02	0.01	0	0	0.01	0	0.03	0.02	0.03
7/27/2017 3:55	0	0	0.05	0.06	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.02	0.09	0.03	0.01	0.01	0.02	0.01	0	0	0.01	0	0.03	0.03	0.02
7/27/2017 3:56	0	0	0.05	0.06	0.01	0.01	0.01	0.01	0.01	0.02	0.05	0.02	0.1	0.01	0.02	0.01	0.04	0.01	0	0	0.01	0	0.03	0.04	0.02
7/27/2017 3:57	0	0	0.03	0.04	0.01	0.01	0.01	0.02	0.01	0.02	0.03	0.02	0.08	0.01	0.02	0.01	0.04	0	0	0	0.01	0	0.01	0.02	0.03
7/27/2017 3:58	0.05	0	0.04	0.04	0.03	0.01	0.02	0.02	0.01	0.02	0.03	0.06	0.05	0.01	0.02	0.01	0.05	0	0	0	0.01	0	0.01	0.01	0.05

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 3:59	0.04	0	0.05	0.04	0.02	0.01	0.02	0.02	0.01	0.01	0.01	0.04	0.05	0.01	0.02	0.03	0.03	0	0	0	0.01	0	0.01	0.01	0.05
7/27/2017 4:00	0.04	0	0.06	0.02	0.02	0.01	0.02	0.03	0.01	0.01	0.01	0.05	0.05	0.01	0.02	0.03	0.03	0	0	0	0.01	0	0.01	0.01	0.01
7/27/2017 4:01	0.04	0	0.06	0.04	0.03	0.02	0.02	0.03	0.01	0.01	0.01	0.03	0.06	0	0.02	0.02	0.01	0	0	0	0.01	0	0.01	0.02	0.01
7/27/2017 4:02	0.05	0	0.06	0.05	0.03	0.04	0.02	0.02	0.01	0.02	0.01	0.05	0.07	0	0.02	0.02	0.01	0	0	0	0.01	0	0.01	0.01	0
7/27/2017 4:03	0.05	0	0.04	0.06	0.05	0.02	0.01	0.02	0.01	0.02	0.01	0.03	0.06	0	0.02	0.01	0.01	0	0	0	0.01	0	0.01	0.02	0
7/27/2017 4:04	0.04	0.01	0.03	0.06	0.04	0.02	0.01	0.02	0.01	0.02	0.01	0.01	0.04	0	0.02	0.01	0.01	0	0	0	0.01	0	0.01	0.01	0
7/27/2017 4:05	0.04	0.01	0.03	0.06	0.02	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.02	0	0.05	0.01	0.01	0	0	0	0.01	0	0.01	0.01	0
7/27/2017 4:06	0.01	0.01	0	0.04	0.02	0.01	0.01	0.01	0.02	0.03	0.01	0.01	0	0	0.06	0.01	0.01	0	0	0	0.01	0	0.01	0.02	0
7/27/2017 4:07	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.02	0.04	0.01	0.01	0	0	0.03	0.01	0	0	0	0	0.01	0	0.01	0.01	0
7/27/2017 4:08	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.05	0	0.01	0	0	0.03	0	0	0	0	0	0	0	0.01	0.01	0
7/27/2017 4:09	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0.03	0.01	0
7/27/2017 4:10	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0.02	0	0	0	0	0.01	0	0	0	0	0	0	0	0.03	0.02	0
7/27/2017 4:11	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0.02	0	0	0	0	0.01	0	0	0	0	0	0	0	0.02	0.02	0
7/27/2017 4:12	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0.02	0.05	0
7/27/2017 4:13	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.05	0
7/27/2017 4:14	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0
7/27/2017 4:15	0	0	0	0.01	0.01	0	0.01	0.02	0.02	0.01	0	0	0.02	0	0	0	0	0	0	0	0	0	0.01	0.04	0
7/27/2017 4:16	0	0	0	0.01	0.01	0	0.01	0.02	0.02	0.01	0	0	0.02	0	0	0	0	0	0	0	0	0	0.01	0.02	0
7/27/2017 4:17	0	0	0	0.01	0.01	0	0.01	0.01	0.02	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0.01	0
7/27/2017 4:18	0	0	0	0.01	0	0	0.01	0.01	0.02	0.03	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0.02	0
7/27/2017 4:19	0	0	0	0.01	0	0	0	0.01	0.01	0.03	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0.03	0
7/27/2017 4:20	0	0	0	0.02	0	0	0	0	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0.05	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 4:21	0	0	0	0.02	0	0	0	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0.08	0
7/27/2017 4:22	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/27/2017 4:23	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:24	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 4:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 4:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
7/27/2017 4:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 4:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 4:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 5:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:20	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/27/2017 5:21	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/27/2017 5:22	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/27/2017 5:23	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/27/2017 5:24	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/27/2017 5:25	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
7/27/2017 5:26	0	0	0	0	0	0	0.01	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	IN14	PS2	BR01	BR03	BR05	BR06	BR07	BR10	BR11	BR12	IN11	IN12	IN16	ML08	ML09	ML10	TU02	TU03	TU04	TU06	TU07	TU08	TU09	IN08	ML04
7/27/2017 5:27	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:28	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:29	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:30	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:31	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:32	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/27/2017 5:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M.2.3 AUGUST 2017 HISTORIC EVENT

Table M.2.3. August 2017 Historic Event Hyetograph Data.

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 8:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:28	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:29	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:30	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:31	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:32	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 8:33	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0
9/21/2017 8:34	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 8:35	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.04	0
9/21/2017 8:36	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.05	0
9/21/2017 8:37	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.05	0
9/21/2017 8:38	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0.01	0	0	0	0	0	0	0	0	0	0	0.04	0
9/21/2017 8:39	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0	0.04	0
9/21/2017 8:40	0	0.01	0	0	0	0	0	0	0	0	0	0	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0	0.03	0
9/21/2017 8:41	0	0.01	0	0	0	0	0	0	0	0	0	0	0.05	0.06	0.01	0	0	0	0	0	0	0	0	0	0	0.04	0
9/21/2017 8:42	0	0.02	0	0	0	0	0	0	0	0	0	0	0.02	0.06	0.02	0	0	0	0	0	0	0	0	0	0	0.02	0
9/21/2017 8:43	0	0.03	0	0	0	0	0	0	0	0	0	0	0.02	0.07	0.02	0	0	0	0	0	0	0	0	0	0	0.04	0
9/21/2017 8:44	0	0.03	0	0	0.02	0	0	0	0	0	0	0	0	0.05	0.02	0	0	0	0	0	0	0	0	0	0	0.09	0
9/21/2017 8:45	0	0.03	0	0.01	0.02	0	0	0	0	0	0	0	0	0.03	0.02	0	0	0	0	0	0	0	0	0	0	0.05	0
9/21/2017 8:46	0	0.04	0	0.01	0.02	0.02	0	0	0	0	0	0	0.02	0.03	0.02	0	0	0	0	0	0	0	0	0	0	0.04	0
9/21/2017 8:47	0	0.04	0	0.01	0.01	0.02	0	0	0	0	0	0	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.05	0
9/21/2017 8:48	0	0.02	0	0.01	0.01	0.02	0	0	0	0	0	0	0.06	0.01	0.04	0	0	0	0.01	0	0	0	0	0	0	0.02	0
9/21/2017 8:49	0	0.02	0	0	0.01	0.02	0	0	0	0	0	0	0.04	0.01	0.03	0	0	0	0.01	0	0	0	0	0.02	0	0.04	0
9/21/2017 8:50	0	0.02	0	0	0.01	0.01	0	0	0	0	0	0	0.04	0.01	0.03	0	0	0	0.01	0	0	0	0	0.01	0	0.06	0
9/21/2017 8:51	0	0.05	0	0	0.01	0.01	0	0	0	0	0	0	0.05	0.01	0.02	0	0	0	0.04	0	0	0	0	0.01	0	0.04	0
9/21/2017 8:52	0	0.03	0	0	0.01	0.01	0	0	0	0	0	0	0.08	0.01	0.02	0	0	0	0.04	0	0	0	0	0.01	0	0.04	0
9/21/2017 8:53	0	0.02	0	0	0.01	0.01	0	0	0	0	0	0	0.08	0.04	0.04	0	0	0	0.04	0	0	0	0	0.03	0	0.04	0
9/21/2017 8:54	0	0.02	0	0	0.01	0.01	0	0.03	0	0	0	0	0.09	0.06	0.1	0	0	0	0.01	0	0	0	0	0	0	0.03	0
9/21/2017 8:55	0	0.03	0	0	0.01	0.01	0	0.03	0	0	0	0	0.08	0.05	0.05	0	0	0	0.01	0	0	0	0	0	0	0.04	0
9/21/2017 8:56	0	0.08	0	0	0.01	0.01	0	0.03	0	0	0	0	0.06	0.07	0.09	0	0	0	0.01	0	0	0	0	0	0	0.02	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 8:57	0	0.05	0	0.04	0.04	0.03	0	0.02	0	0	0	0	0.07	0.04	0.05	0	0	0	0.01	0	0	0	0.01	0	0	0.01	0
9/21/2017 8:58	0	0.04	0	0.06	0.04	0.03	0.03	0.02	0	0	0	0	0.05	0.03	0.06	0	0	0	0.01	0	0	0	0.01	0	0	0.01	0
9/21/2017 8:59	0	0.04	0	0.04	0.03	0.05	0.02	0.02	0	0	0	0	0.03	0.03	0.11	0	0	0	0.01	0	0	0	0.01	0	0	0	0
9/21/2017 9:00	0	0.06	0	0.04	0.03	0.03	0.02	0.04	0.02	0	0	0.02	0.03	0.03	0.13	0	0	0	0.01	0	0	0	0.01	0	0	0	0
9/21/2017 9:01	0	0.05	0	0.03	0.04	0.08	0.02	0.08	0.02	0	0	0.02	0.09	0.05	0.08	0	0	0	0.01	0	0	0	0.01	0	0	0	0
9/21/2017 9:02	0	0.05	0	0.03	0.06	0.07	0.04	0.09	0.04	0	0	0.02	0.07	0.07	0.08	0	0	0	0.01	0	0	0	0	0	0.01	0	0
9/21/2017 9:03	0	0.08	0	0.04	0.06	0.07	0.05	0.04	0.04	0	0	0.01	0.04	0.09	0.08	0	0	0	0.01	0	0	0	0	0	0.01	0	0
9/21/2017 9:04	0	0.06	0	0.03	0.03	0.04	0.09	0.04	0.1	0	0	0.01	0	0.1	0.07	0	0	0	0.01	0	0	0	0	0	0.01	0	0
9/21/2017 9:05	0	0.07	0	0.03	0.03	0.03	0.11	0.04	0.08	0.02	0	0.01	0	0.03	0.07	0	0	0	0	0	0	0	0	0	0.01	0	0
9/21/2017 9:06	0	0.02	0	0.1	0.05	0.03	0.03	0.03	0.04	0.02	0	0.03	0	0.03	0.04	0	0	0	0	0	0	0	0	0	0.01	0	0
9/21/2017 9:07	0	0.01	0	0.04	0.08	0.04	0.04	0.03	0.04	0.03	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/21/2017 9:08	0	0.01	0	0	0.05	0.05	0.03	0	0.02	0.05	0	0.02	0	0	0.12	0	0	0	0	0	0	0	0	0	0.03	0	0
9/21/2017 9:09	0	0.02	0	0.03	0.05	0.06	0.03	0	0.02	0.03	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0	0.04	0	0
9/21/2017 9:10	0	0.01	0	0.03	0.03	0.06	0.04	0	0.03	0.03	0	0.05	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:11	0	0.02	0	0.04	0.01	0.04	0.04	0	0.03	0.03	0	0.04	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:12	0	0.01	0	0.01	0.01	0.04	0.03	0	0.03	0.02	0.01	0.04	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:13	0	0.01	0	0.01	0.01	0.01	0.03	0	0	0.02	0.01	0.03	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:14	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0.01	0.04	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:15	0	0	0	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0.03	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:16	0	0	0	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0.02	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:17	0	0	0	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:18	0	0	0	0	0.01	0.01	0.01	0	0	0.01	0.01	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 9:19	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:20	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:21	0	0.01	0	0	0	0	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:22	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:23	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:24	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:25	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:26	0	0	0	0	0	0	0	0	0	0.01	0.02	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:27	0	0	0	0	0	0	0	0	0	0.01	0.02	0.04	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:28	0	0	0	0	0	0	0	0	0	0.01	0.02	0.04	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:29	0	0	0	0	0	0	0	0	0	0.01	0.02	0.04	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:30	0	0	0	0	0	0	0	0	0	0.01	0.02	0.02	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:31	0	0	0	0.04	0.03	0.04	0	0	0	0.01	0.02	0.02	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:32	0	0	0	0.03	0.05	0.05	0	0	0	0.01	0.01	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:33	0	0.01	0	0	0.05	0.06	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:34	0	0	0	0	0.02	0.1	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0
9/21/2017 9:35	0	0	0	0	0.02	0.03	0.05	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0
9/21/2017 9:36	0	0	0	0	0.01	0.02	0.05	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0
9/21/2017 9:37	0	0	0	0	0.01	0.02	0.04	0.01	0.01	0.01	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:38	0	0	0	0	0.01	0.02	0.05	0.01	0.02	0.01	0.01	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:39	0	0	0	0	0.01	0	0.05	0.01	0.02	0.07	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:40	0	0	0	0	0	0	0.09	0	0.03	0.07	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 9:41	0	0	0	0	0	0	0.03	0	0.03	0.03	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:42	0	0	0	0	0	0	0	0	0.03	0.03	0.04	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:43	0	0	0	0	0	0	0	0	0.02	0.06	0.03	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:44	0	0	0	0	0	0	0	0	0.02	0.06	0.05	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:45	0	0	0	0	0	0	0	0	0	0.06	0.06	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:46	0	0	0	0	0	0	0	0	0	0.05	0.06	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:47	0	0	0	0	0	0	0	0	0	0.03	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0.06	0	0
9/21/2017 9:48	0	0	0	0	0	0	0	0	0	0	0.06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:49	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 9:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 10:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:10	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:11	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:12	0	0	0	0.03	0.03	0.03	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:13	0	0	0	0.01	0.03	0.02	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:14	0	0	0	0.01	0.01	0.02	0.02	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:15	0	0	0	0.01	0.01	0.02	0.02	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:16	0	0	0	0.01	0.01	0	0.02	0	0	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:17	0	0	0	0.01	0.01	0	0	0	0	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:18	0	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:19	0	0	0	0.01	0	0	0.08	0	0	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:20	0	0	0	0	0	0	0.02	0.02	0	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:21	0	0	0	0	0	0	0.02	0.02	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:22	0	0	0	0	0	0	0.01	0	0.03	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:23	0	0	0	0	0	0	0.01	0	0.04	0.04	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:24	0	0	0	0	0	0	0.01	0	0	0.04	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 10:25	0	0	0	0	0	0	0	0	0	0.04	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:26	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:27	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:28	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:29	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:30	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 10:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 15:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:44	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:45	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:46	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:47	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:48	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:49	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:50	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:51	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:52	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 15:53	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 15:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 16:15	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:16	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:17	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:18	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:19	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:20	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:21	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 16:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 21:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/21/2017 21:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0
9/21/2017 21:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0
9/21/2017 21:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0
9/21/2017 21:27	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0.02	0	0	0
9/21/2017 21:28	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0.02	0	0	0
9/21/2017 21:29	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0.02	0	0	0
9/21/2017 21:30	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0.01	0	0.02	0	0	0
9/21/2017 21:31	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0.03	0	0.02	0	0	0
9/21/2017 21:32	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0.03	0.03	0	0.04	0	0	0
9/21/2017 21:33	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.03	0.02	0	0.03	0	0	0
9/21/2017 21:34	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.03	0.02	0	0.04	0	0	0
9/21/2017 21:35	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.02	0.03	0	0.03	0	0	0
9/21/2017 21:36	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.02	0	0.04	0.01	0.03	0.02	0.04	0	0	0
9/21/2017 21:37	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.02	0	0.07	0.01	0.03	0.02	0.04	0.02	0	0.01
9/21/2017 21:38	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.04	0	0.06	0.01	0.02	0.01	0.03	0.03	0	0.01
9/21/2017 21:39	0.02	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.03	0	0.05	0.04	0.02	0.01	0.03	0.05	0	0.01
9/21/2017 21:40	0.02	0	0.02	0	0	0	0	0.02	0.01	0.01	0.01	0	0	0	0	0.01	0.01	0.03	0.01	0.04	0.05	0.03	0.01	0.02	0.06	0	0
9/21/2017 21:41	0.04	0	0.03	0	0	0	0	0.02	0.01	0.01	0.01	0	0	0	0	0.01	0.01	0.02	0.01	0.02	0.05	0.05	0.03	0.02	0.07	0	0
9/21/2017 21:42	0.05	0	0.03	0	0	0	0	0.03	0.01	0.01	0.01	0	0	0	0	0.04	0.04	0.02	0.01	0.02	0.06	0.05	0.03	0.06	0.03	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 21:43	0.05	0	0.04	0	0	0	0.02	0.03	0.04	0.01	0.01	0	0	0	0.01	0.04	0.02	0.02	0.01	0	0.06	0.07	0.03	0.08	0.04	0	0
9/21/2017 21:44	0.05	0	0.04	0.01	0	0	0.02	0.04	0.04	0.01	0.02	0	0	0	0.01	0.04	0.04	0.02	0.01	0	0.06	0.08	0.03	0.08	0.04	0	0
9/21/2017 21:45	0.03	0	0.04	0.01	0	0	0.02	0.04	0.04	0.03	0.02	0.02	0	0	0.01	0.05	0.02	0	0.02	0	0.05	0.07	0.03	0.09	0.04	0.01	0
9/21/2017 21:46	0.02	0	0.04	0.01	0.01	0	0.02	0.04	0.04	0.03	0.04	0.02	0.02	0	0.03	0.03	0.02	0	0.02	0	0.04	0.06	0.02	0.08	0.03	0.01	0
9/21/2017 21:47	0.02	0	0.03	0.01	0.01	0	0.03	0.04	0.04	0.06	0.05	0.01	0.02	0	0.03	0.03	0	0	0.04	0	0.02	0.04	0.02	0.09	0.06	0	0
9/21/2017 21:48	0.02	0	0.05	0.01	0.01	0.02	0.02	0.02	0.04	0.06	0.04	0.01	0.03	0.02	0.04	0.03	0	0	0.04	0	0.02	0.03	0.02	0.06	0.06	0	0
9/21/2017 21:49	0.02	0.02	0.04	0.02	0.02	0.02	0.03	0.02	0.04	0.05	0.05	0.01	0.02	0.02	0.01	0.03	0	0	0.04	0	0.01	0	0.02	0.03	0.06	0	0
9/21/2017 21:50	0.01	0.02	0.03	0.02	0.02	0.01	0.03	0.02	0.03	0.05	0.04	0.02	0.02	0.02	0.01	0	0	0	0.03	0	0.01	0.01	0.04	0.01	0.06	0	0
9/21/2017 21:51	0.01	0.02	0.04	0.03	0.02	0.01	0.03	0.04	0.03	0.05	0.05	0.02	0.02	0.01	0.01	0	0	0	0.03	0.02	0.01	0.01	0.11	0.01	0.04	0	0
9/21/2017 21:52	0.01	0.01	0.06	0.03	0.03	0.01	0.04	0.04	0.03	0.03	0.04	0.02	0	0.01	0.01	0	0	0	0.03	0.02	0.01	0.01	0.04	0.01	0.05	0	0
9/21/2017 21:53	0.01	0.01	0.04	0.03	0.04	0.04	0.05	0.04	0.03	0.05	0.05	0.03	0	0.01	0.01	0	0	0	0.03	0.06	0.01	0.01	0.14	0.01	0.05	0	0
9/21/2017 21:54	0.01	0	0.07	0.01	0.01	0.03	0.06	0.09	0.04	0.04	0.02	0.04	0	0.01	0.01	0	0	0	0	0.04	0.01	0.01	0.04	0.01	0.07	0	0
9/21/2017 21:55	0.01	0	0.07	0.01	0.01	0.01	0.07	0.04	0.04	0.06	0.02	0.04	0	0	0.01	0	0	0.01	0	0.03	0.01	0.01	0.08	0.02	0.08	0	0
9/21/2017 21:56	0.01	0	0.04	0.01	0.01	0.01	0.05	0.09	0.04	0.06	0.05	0.04	0	0	0.01	0	0	0.01	0	0.05	0.01	0.01	0.04	0.02	0.08	0	0.04
9/21/2017 21:57	0.01	0	0.03	0.01	0.01	0.01	0.04	0.04	0.17	0.06	0.05	0.04	0	0	0.01	0	0	0.01	0	0.04	0.01	0.02	0.03	0.02	0.06	0	0.01
9/21/2017 21:58	0.01	0	0.03	0.01	0.01	0.01	0.02	0.04	0.04	0.05	0.06	0.04	0	0	0.01	0	0	0.01	0	0.05	0.01	0.02	0.03	0.02	0.11	0	0.01
9/21/2017 21:59	0.01	0	0.03	0.01	0.01	0.01	0.02	0.04	0.1	0.06	0.06	0.04	0	0	0.01	0.01	0	0.01	0	0.05	0.04	0.04	0.01	0.02	0.11	0	0.01
9/21/2017 22:00	0.05	0	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.05	0.08	0.03	0	0	0.01	0.01	0.06	0.07	0	0.05	0.04	0.04	0.01	0.02	0.09	0	0.02
9/21/2017 22:01	0.04	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.05	0.04	0.03	0	0	0.01	0.01	0.05	0.07	0	0.04	0.03	0.03	0.01	0.02	0.11	0	0.02
9/21/2017 22:02	0.03	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.04	0.03	0	0	0.01	0.07	0.05	0.08	0.02	0.04	0.04	0.03	0.01	0.02	0.01	0	0.03
9/21/2017 22:03	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.04	0.03	0	0	0.03	0.04	0.07	0.09	0.02	0.02	0.04	0.03	0.01	0.02	0.01	0	0.05
9/21/2017 22:04	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.04	0.03	0	0	0.04	0.06	0.06	0.09	0.04	0.02	0.03	0.03	0.01	0.02	0.01	0	0.04

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 22:05	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0	0	0.06	0.05	0.03	0	0.04	0.02	0.04	0.03	0.01	0.02	0.02	0.01	0.03
9/21/2017 22:06	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.04	0	0.11	0.06	0.03	0.03	0.04	0.02	0.05	0.04	0.05	0.02	0.02	0.02	0.03
9/21/2017 22:07	0.02	0	0.01	0.02	0.02	0.02	0.01	0.01	0.02	0.02	0.02	0.02	0.04	0	0.09	0.08	0.02	0.03	0.04	0.02	0.03	0.03	0.02	0.02	0.02	0.01	0.03
9/21/2017 22:08	0.02	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.02	0.02	0.02	0.02	0.07	0.05	0.07	0.06	0.08	0.05	0.04	0.02	0.03	0.02	0.03	0.02	0.02	0.02	0.03
9/21/2017 22:09	0.01	0.01	0.03	0.06	0.03	0.02	0.03	0.03	0.02	0.02	0.02	0.05	0.09	0.06	0.06	0.05	0.07	0.02	0.06	0.03	0.03	0.02	0.03	0.02	0.02	0.02	0.03
9/21/2017 22:10	0.01	0.03	0.01	0.09	0.03	0.05	0.03	0.03	0.02	0.02	0.02	0.04	0.07	0.08	0.04	0.03	0.03	0.02	0.06	0.03	0.03	0.02	0.03	0.02	0.02	0.07	0.03
9/21/2017 22:11	0.01	0.06	0.01	0.06	0.08	0.06	0.04	0.03	0.02	0.02	0.02	0.02	0.07	0.06	0	0.05	0.03	0.03	0.04	0.02	0.02	0.02	0.03	0.01	0.02	0.04	0.02
9/21/2017 22:12	0.01	0.05	0.01	0.07	0.06	0.07	0.04	0.04	0.03	0.02	0.02	0.02	0.05	0.06	0.3	0.04	0.04	0.04	0.04	0.02	0.02	0.01	0.03	0.01	0.02	0.07	0.02
9/21/2017 22:13	0.01	0.05	0.01	0.07	0.06	0.05	0.07	0.04	0.03	0.02	0.02	0.02	0.04	0.04	0.04	0.02	0.03	0.05	0.02	0.02	0.02	0.01	0.02	0.01	0.02	0.03	0.03
9/21/2017 22:14	0.01	0.01	0.01	0.05	0.08	0.05	0.05	0.04	0.04	0.02	0.02	0.02	0.06	0.04	0.03	0.02	0.02	0.03	0.02	0.02	0.02	0.01	0.03	0.01	0.02	0.01	0.04
9/21/2017 22:15	0.01	0.01	0.01	0.04	0.04	0.04	0.05	0.04	0.04	0.02	0.02	0.04	0.02	0.06	0.05	0.03	0.02	0.03	0.02	0.03	0.03	0.01	0.03	0.01	0.02	0.02	0.03
9/21/2017 22:16	0.01	0.3	0.02	0.03	0.04	0.04	0.05	0.04	0.03	0.03	0.04	0.04	0.02	0.06	0.06	0.03	0.02	0.03	0.02	0.04	0.03	0.02	0.02	0.01	0.03	0.03	0.03
9/21/2017 22:17	0.01	0.04	0.02	0.03	0.02	0.03	0.04	0.03	0.03	0.04	0.03	0.04	0.05	0.03	0.07	0.03	0.03	0.04	0.01	0.03	0.02	0.02	0.02	0.01	0.03	0.03	0.03
9/21/2017 22:18	0.01	0.05	0.01	0.03	0.02	0.03	0	0.02	0.03	0.04	0.05	0.04	0.05	0.04	0.05	0.03	0.03	0.14	0.01	0.03	0.02	0.02	0.02	0.01	0.02	0.02	0.02
9/21/2017 22:19	0.01	0.05	0.01	0.02	0.02	0.02	0.13	0.02	0.02	0.04	0.03	0.04	0.05	0.04	0.04	0.05	0.04	0.04	0.01	0	0.02	0.01	0.02	0.01	0.02	0.05	0.02
9/21/2017 22:20	0.01	0.04	0.01	0.02	0.02	0.01	0.03	0.02	0.02	0.02	0.03	0.02	0.04	0.04	0.04	0.04	0.04	0.04	0	0	0.02	0.01	0.01	0.01	0.01	0.03	0
9/21/2017 22:21	0.01	0.04	0.01	0.03	0.02	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.05	0.06	0.04	0.03	0.03	0	0	0	0.01	0.01	0.01	0.01	0.01	0.02	0
9/21/2017 22:22	0.01	0.05	0.01	0.04	0.03	0.01	0.02	0.02	0.02	0.01	0.02	0.03	0.05	0.05	0.05	0.02	0.04	0.04	0	0	0.01	0.01	0.01	0.01	0.01	0.03	0
9/21/2017 22:23	0.01	0.05	0.01	0.04	0.04	0.01	0.02	0.02	0.02	0.01	0.02	0.03	0.04	0.08	0.05	0.02	0.04	0.04	0	0	0.01	0.01	0.02	0.01	0.01	0.02	0
9/21/2017 22:24	0	0.04	0.01	0.06	0.04	0.04	0.02	0.02	0.02	0.01	0.01	0.02	0.05	0.07	0.07	0.03	0.01	0.03	0	0	0.02	0.01	0.02	0.01	0.01	0.03	0
9/21/2017 22:25	0	0.04	0.01	0.09	0.08	0.03	0.03	0.02	0.02	0.01	0.01	0.02	0.03	0.07	0.04	0.03	0.01	0.03	0	0	0	0.01	0.02	0	0.01	0.02	0
9/21/2017 22:26	0	0.06	0.01	0.07	0.08	0.06	0.03	0.02	0.02	0.01	0.01	0.02	0.03	0.03	0.03	0.03	0.01	0.02	0	0	0	0	0.02	0	0.01	0.04	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 22:27	0	0.03	0.01	0.05	0.08	0.06	0.04	0.02	0.03	0.01	0.01	0.04	0.03	0.03	0.02	0.01	0.01	0.02	0	0	0	0	0.02	0	0.01	0.02	0
9/21/2017 22:28	0	0.03	0.01	0.04	0.06	0.06	0.05	0.02	0.03	0.03	0.01	0.03	0.02	0.04	0.02	0.01	0.01	0	0	0	0	0	0.02	0	0.01	0.02	0
9/21/2017 22:29	0	0.05	0.01	0.03	0.03	0.03	0.04	0.02	0.03	0.04	0.01	0.03	0.02	0.03	0.02	0.01	0	0	0	0	0	0	0.01	0	0.01	0.03	0
9/21/2017 22:30	0	0.02	0.01	0.04	0.04	0.03	0.04	0.02	0.02	0.04	0.03	0.03	0.05	0.03	0.03	0	0	0	0	0	0	0	0.01	0	0.01	0.02	0
9/21/2017 22:31	0	0.02	0.01	0.04	0.04	0.04	0.04	0.02	0.02	0.01	0.03	0.03	0	0.03	0.06	0	0	0	0	0	0	0	0.01	0	0.01	0.02	0
9/21/2017 22:32	0	0.01	0	0.04	0.04	0.04	0.04	0.02	0.02	0.01	0.02	0.03	0	0.05	0.01	0	0	0	0	0	0	0	0.01	0	0.01	0.02	0
9/21/2017 22:33	0	0.02	0	0.03	0.04	0.04	0.04	0.02	0.02	0.02	0.02	0.03	0	0.05	0.01	0	0	0	0	0	0	0	0.01	0	0	0.02	0
9/21/2017 22:34	0	0.01	0	0.01	0.02	0.03	0.02	0.02	0.02	0.02	0.02	0.03	0	0.04	0.01	0	0	0	0	0	0	0	0.01	0	0	0.04	0
9/21/2017 22:35	0	0.02	0	0.01	0.02	0	0.02	0.02	0.02	0.02	0.02	0.03	0	0.04	0.02	0	0	0	0	0	0	0	0.01	0	0	0.05	0
9/21/2017 22:36	0	0.02	0	0.01	0.02	0	0	0	0	0.02	0.02	0.02	0	0.01	0.02	0	0	0	0	0	0	0	0.01	0	0	0.05	0
9/21/2017 22:37	0	0.04	0	0.01	0.02	0	0	0	0	0.02	0.02	0.02	0	0.01	0.02	0	0	0	0	0.01	0	0	0.01	0	0	0.02	0
9/21/2017 22:38	0	0.03	0	0.01	0	0	0	0	0	0.02	0.02	0.02	0	0.01	0.02	0	0	0	0	0.01	0	0	0	0	0	0.02	0
9/21/2017 22:39	0.01	0.05	0	0.01	0	0	0	0	0	0	0.01	0	0.04	0.01	0.02	0	0	0	0	0.01	0	0	0	0.01	0	0.03	0
9/21/2017 22:40	0.01	0.03	0	0	0	0	0	0	0	0	0.01	0.04	0.04	0.01	0.02	0	0	0	0	0.01	0.01	0	0	0.01	0	0.01	0
9/21/2017 22:41	0.01	0.02	0	0	0	0	0	0	0	0	0.01	0.04	0.04	0.01	0.02	0	0	0	0	0.01	0.01	0	0	0.01	0	0	0.01
9/21/2017 22:42	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.04	0.01	0.02	0	0	0	0	0.04	0.01	0	0	0.01	0	0	0.01
9/21/2017 22:43	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.02	0.02	0.02	0	0	0	0	0.03	0.01	0	0	0.01	0	0.02	0.01
9/21/2017 22:44	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0.02	0.01	0	0	0	0	0.03	0.01	0	0	0.01	0	0.05	0.01
9/21/2017 22:45	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0.02	0.01	0	0	0.01	0	0.02	0.02
9/21/2017 22:46	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0.02	0.01	0	0	0.01	0	0.01	0.02
9/21/2017 22:47	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0.02	0	0	0.01	0	0.02	0.01	0	0	0.01	0.01	0.02	0.01
9/21/2017 22:48	0.03	0.01	0	0	0	0.02	0	0	0	0	0	0	0.01	0.02	0.02	0	0.03	0.01	0	0.02	0.01	0.01	0	0.01	0.01	0.1	0.01

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 22:49	0.03	0.01	0	0.01	0	0.02	0	0	0	0	0	0	0.01	0.02	0.03	0.01	0.02	0.01	0	0.02	0.01	0.01	0	0.01	0.01	0.09	0.01
9/21/2017 22:50	0.02	0.01	0	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.01	0.02	0.05	0.01	0.02	0.02	0	0.02	0.01	0.01	0	0.01	0.01	0.09	0.01
9/21/2017 22:51	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.02	0.03	0.08	0.01	0.02	0.02	0.02	0.02	0.01	0.05	0.01	0.01	0.01	0.1	0.01
9/21/2017 22:52	0.03	0.02	0.04	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.02	0.03	0.09	0.03	0.02	0.02	0.02	0.07	0.01	0	0.01	0.01	0.02	0.05	0.01
9/21/2017 22:53	0.03	0.02	0.03	0.01	0.01	0.01	0.01	0.01	0	0.01	0	0.01	0.03	0.05	0.06	0.02	0.02	0.01	0.02	0.07	0.04	0.04	0.01	0.02	0.02	0.07	0.06
9/21/2017 22:54	0.03	0.03	0.03	0.01	0.01	0.01	0.01	0.01	0	0.01	0	0.01	0.03	0.06	0.05	0.02	0.02	0.01	0.02	0.04	0.05	0.05	0.01	0.02	0.03	0.07	0.09
9/21/2017 22:55	0.04	0.05	0.04	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.04	0.08	0.03	0.03	0.01	0.02	0.03	0.06	0.05	0.01	0.03	0.01	0.09	0.12
9/21/2017 22:56	0.03	0.12	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.06	0.02	0.03	0.01	0.02	0.03	0.02	0.03	0.01	0.03	0.01	0.08	0.09
9/21/2017 22:57	0.04	0.12	0.02	0.02	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.07	0.08	0.02	0.03	0.01	0.01	0.04	0.02	0.03	0.01	0.06	0.01	0.09	0.03
9/21/2017 22:58	0.02	0.15	0.03	0.01	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.06	0.05	0.02	0.03	0.02	0.01	0.03	0.04	0	0.02	0.06	0.01	0.09	0.03
9/21/2017 22:59	0.02	0.08	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.04	0.08	0.05	0.02	0.03	0.02	0.01	0.01	0.03	0.04	0.02	0.05	0.01	0.02	0.02
9/21/2017 23:00	0.01	0.05	0.01	0.01	0.01	0.02	0.02	0.03	0.01	0.01	0.01	0.01	0.07	0.06	0.04	0.04	0.04	0.04	0.01	0.01	0.04	0.04	0.05	0.04	0.01	0.08	0.02
9/21/2017 23:01	0.01	0.09	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.01	0.01	0.04	0.06	0.05	0.04	0.04	0.04	0.06	0.01	0.01	0.04	0.04	0.02	0.07	0.01	0.05	0
9/21/2017 23:02	0.01	0.08	0.03	0.03	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.02	0.07	0.06	0.04	0.03	0.04	0.08	0.04	0.01	0.03	0.03	0.02	0.06	0.01	0.05	0.01
9/21/2017 23:03	0.01	0.08	0.03	0.03	0.03	0.01	0.02	0.01	0.01	0.02	0.01	0.02	0.08	0.07	0.05	0.04	0.03	0.05	0.04	0.01	0.03	0.03	0.03	0.05	0.01	0.1	0.01
9/21/2017 23:04	0.01	0.04	0.02	0.06	0.03	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.08	0.08	0.17	0.04	0.03	0.05	0.04	0.01	0.03	0.03	0.03	0.03	0.02	0.09	0.01
9/21/2017 23:05	0.01	0.04	0.02	0.05	0.05	0.05	0.01	0.01	0.01	0.02	0.02	0.02	0.09	0.08	0.1	0.04	0.03	0.05	0.04	0.01	0.03	0.02	0.04	0.02	0.02	0.12	0.01
9/21/2017 23:06	0.01	0.03	0.02	0.08	0.06	0.06	0.01	0.01	0.01	0.02	0.02	0.04	0.08	0.08	0.08	0.04	0.03	0.04	0.04	0.03	0.01	0.02	0.03	0.02	0.02	0.09	0.01
9/21/2017 23:07	0.01	0.09	0.02	0.07	0.05	0.06	0.01	0.01	0.01	0.01	0.01	0.03	0.06	0.08	0.04	0.04	0.03	0	0.04	0.04	0.01	0.01	0.03	0.01	0.02	0.1	0.01
9/21/2017 23:08	0.01	0.1	0.01	0.07	0.07	0.22	0.01	0.01	0.01	0.01	0.01	0.02	0.05	0.07	0	0.02	0.01	0.01	0.04	0.06	0.01	0.01	0.04	0.01	0.02	0.02	0.01
9/21/2017 23:09	0.01	0.07	0.01	0.09	0.05	0.06	0.03	0.01	0.01	0.01	0.01	0.02	0.02	0.05	0.02	0.02	0.01	0.01	0.04	0.05	0.02	0.01	0.04	0.01	0.02	0.01	0.01
9/21/2017 23:10	0.04	0.07	0.01	0.06	0.06	0.07	0.06	0.04	0.01	0.01	0.01	0.03	0.01	0.03	0.02	0.01	0.01	0.01	0.01	0.06	0.02	0.02	0.04	0.01	0.02	0.01	0.01

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 23:11	0.05	0.05	0.01	0.05	0.05	0.19	0.05	0.04	0.03	0.01	0.01	0.03	0.01	0.03	0	0.01	0.01	0.02	0.01	0.05	0.03	0.02	0.04	0.01	0.04	0.01	0.01
9/21/2017 23:12	0.03	0.02	0.01	0.02	0.04	0.03	0.05	0.04	0.04	0.01	0.01	0.03	0.01	0.01	0	0.01	0.01	0.02	0.01	0.04	0.03	0.02	0.04	0.01	0.06	0	0.01
9/21/2017 23:13	0.03	0.01	0.01	0.01	0.01	0.03	0.04	0.04	0.04	0.03	0.01	0.04	0.01	0.01	0	0.01	0.02	0.02	0.01	0.04	0.03	0.02	0.04	0.01	0.04	0	0.01
9/21/2017 23:14	0.04	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.03	0.04	0.01	0.05	0.01	0.01	0	0.01	0.02	0.03	0.01	0.03	0.04	0.05	0.03	0.04	0.05	0	0.01
9/21/2017 23:15	0.04	0	0.01	0.01	0.01	0.01	0.02	0.03	0.02	0.03	0.01	0.07	0.01	0.01	0	0.02	0.02	0.04	0.01	0.03	0.05	0.04	0.01	0.04	0.05	0	0.01
9/21/2017 23:16	0.03	0	0.01	0.01	0.01	0.01	0.01	0.03	0.02	0.02	0.02	0.06	0.01	0.01	0	0.02	0.02	0.05	0.01	0.02	0.05	0.06	0.01	0.05	0.04	0	0.01
9/21/2017 23:17	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.04	0.01	0.01	0	0.04	0.03	0.07	0.01	0.02	0.05	0.05	0.01	0.04	0.03	0	0.04
9/21/2017 23:18	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0	0.04	0.03	0.05	0.04	0.01	0.04	0.05	0.01	0.04	0.03	0	0.02
9/21/2017 23:19	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0	0.06	0.04	0.04	0.04	0.01	0.03	0.03	0.01	0.01	0.03	0	0.02
9/21/2017 23:20	0.02	0	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.06	0.04	0.04	0.04	0.01	0.01	0	0.01	0.01	0.01	0	0.01
9/21/2017 23:21	0.01	0	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.06	0.03	0	0.04	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01
9/21/2017 23:22	0.01	0	0.02	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.06	0.03	0.03	0.02	0.01	0.01	0.02	0.04	0.01	0.01	0	0.02
9/21/2017 23:23	0.01	0	0.02	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.05	0.03	0.03	0.02	0.01	0.01	0.01	0.04	0.01	0.01	0	0.02
9/21/2017 23:24	0.01	0	0.03	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.03	0.03	0.04	0.01	0.01	0.01	0.04	0.01	0.01	0	0.02
9/21/2017 23:25	0.01	0	0.03	0.01	0.01	0.02	0.02	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.03	0.02	0.01	0.01	0.01	0.04	0.01	0.01	0	0.02
9/21/2017 23:26	0.01	0	0.05	0.01	0.01	0.02	0.02	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.03	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.08	0.01	0.01	0.01	0.02
9/21/2017 23:27	0.01	0	0.03	0.01	0.01	0.03	0.02	0.03	0.03	0.03	0.02	0.01	0.01	0.01	0.03	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.02	0.01	0.04	0.02	0.03
9/21/2017 23:28	0.01	0.01	0.03	0.01	0.01	0.03	0.02	0.03	0.02	0.03	0.02	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.02	0.01	0.04	0.01	0.02
9/21/2017 23:29	0.01	0.01	0.02	0.01	0.01	0.03	0.02	0.03	0.02	0.02	0.04	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.03	0.02
9/21/2017 23:30	0.01	0.01	0	0.01	0.01	0.01	0.02	0.01	0.03	0.02	0.03	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.03	0.02
9/21/2017 23:31	0.01	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.05	0.02
9/21/2017 23:32	0.01	0.02	0	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.04	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.03	0.05	0.04

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 23:33	0.01	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.06	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.1	0
9/21/2017 23:34	0.01	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.02	0.01	0.06	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.12	0.04
9/21/2017 23:35	0.01	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.03	0.03	0.05	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.1	0.02
9/21/2017 23:36	0.01	0.05	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.03	0.03	0.06	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.09	0.02
9/21/2017 23:37	0.01	0.05	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.03	0.03	0.06	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.06	0.01
9/21/2017 23:38	0.01	0.04	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.03	0.05	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.05	0.01
9/21/2017 23:39	0.01	0.04	0	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.03	0	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.01
9/21/2017 23:40	0.01	0.05	0	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.03	0.03	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.01
9/21/2017 23:41	0.01	0.03	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0	0.01	0.02	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01
9/21/2017 23:42	0.01	0.04	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
9/21/2017 23:43	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/21/2017 23:44	0	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02
9/21/2017 23:45	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02
9/21/2017 23:46	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02
9/21/2017 23:47	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.02
9/21/2017 23:48	0	0.02	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.02
9/21/2017 23:49	0	0.02	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.02
9/21/2017 23:50	0	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0	0.01	0	0	0.01	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.03
9/21/2017 23:51	0	0	0.01	0.01	0.01	0	0.01	0.01	0.01	0	0	0.01	0	0	0.01	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.03
9/21/2017 23:52	0	0	0.01	0.01	0.01	0.04	0.01	0.01	0.01	0	0	0.01	0	0	0.01	0.02	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.02
9/21/2017 23:53	0	0	0.01	0.01	0	0	0	0.01	0.01	0	0	0.01	0	0	0.01	0.02	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.03
9/21/2017 23:54	0	0	0.01	0	0	0	0	0.01	0.01	0	0	0.01	0	0	0.01	0.02	0.04	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.03

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/21/2017 23:55	0	0	0.01	0	0	0	0	0.01	0.01	0	0	0.01	0	0	0.01	0.02	0.04	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.02
9/21/2017 23:56	0	0	0.01	0	0	0	0	0.01	0.01	0	0	0	0	0	0.01	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.02
9/21/2017 23:57	0	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01
9/21/2017 23:58	0	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.02	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01
9/21/2017 23:59	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0.02	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01
9/22/2017 0:00	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0.02	0.01	0	0.01	0.02	0.02	0.02	0	0.01	0	0.01	0	0.01	0.01	0.01
9/22/2017 0:01	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0.02	0.01	0	0.02	0.01	0.02	0.02	0	0.01	0	0.01	0	0	0.01	0.01
9/22/2017 0:02	0	0.02	0	0	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0.02	0.01	0.02	0.02	0	0.01	0	0	0	0	0.01	0.01
9/22/2017 0:03	0	0.02	0	0.02	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0.02	0.01	0.01	0.02	0	0.01	0	0	0	0	0.01	0.01
9/22/2017 0:04	0	0.01	0	0.02	0.02	0.02	0.01	0.01	0	0	0	0	0.01	0.01	0	0.02	0.01	0.01	0.01	0	0.01	0	0	0	0	0.01	0.01
9/22/2017 0:05	0	0.01	0	0.01	0.02	0.02	0.01	0.01	0	0	0	0	0.01	0.01	0	0.02	0.01	0.01	0.01	0	0.01	0	0	0	0	0.03	0.01
9/22/2017 0:06	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0.01	0	0	0.02	0.01	0.02	0.01	0	0.01	0	0	0	0	0.01	0
9/22/2017 0:07	0	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0	0	0	0.01	0	0	0.01	0.01	0.02	0.01	0	0.01	0	0	0	0	0	0.01
9/22/2017 0:08	0	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0	0	0	0.01	0	0	0.01	0.01	0.01	0.01	0	0.01	0	0	0	0	0	0.01
9/22/2017 0:09	0	0	0	0.01	0	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0	0.01	0.01	0	0.01	0.01	0	0.01	0	0	0	0	0	0.01
9/22/2017 0:10	0	0	0	0.01	0	0	0.01	0.01	0.01	0	0	0.01	0.01	0	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0	0.01
9/22/2017 0:11	0	0	0	0.01	0	0	0.01	0.01	0.01	0	0	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:12	0	0	0	0.01	0	0	0.01	0.01	0.01	0	0	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:13	0	0	0	0.01	0	0	0.01	0	0.01	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.02
9/22/2017 0:14	0	0	0	0.01	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
9/22/2017 0:15	0	0	0	0.01	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:16	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 0:17	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:18	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:23	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:24	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:25	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:26	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:27	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:28	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0.01	0.01
9/22/2017 0:29	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0.02	0	0	0	0	0.01	0.01
9/22/2017 0:30	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0.02	0	0	0	0	0.02	0.01
9/22/2017 0:31	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0.01
9/22/2017 0:32	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0.01
9/22/2017 0:33	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.02	0.01
9/22/2017 0:34	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.02	0.01
9/22/2017 0:35	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.04	0.01
9/22/2017 0:36	0	0.01	0	0.01	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0.06	0.01
9/22/2017 0:37	0	0.02	0	0.01	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.1	0.03
9/22/2017 0:38	0	0.02	0	0.01	0.01	0.01	0	0	0	0	0	0	0.02	0.02	0.02	0	0	0	0	0	0	0	0	0	0	0.06	0.03

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 0:39	0	0.04	0	0	0.01	0.01	0	0	0	0	0	0	0.02	0.02	0.05	0	0	0	0	0	0	0	0	0	0	0.07	0
9/22/2017 0:40	0	0.04	0	0.01	0.01	0.01	0	0	0	0	0	0	0.02	0	0.05	0	0	0	0	0	0	0	0	0	0	0.05	0
9/22/2017 0:41	0	0.09	0	0.01	0.01	0.01	0	0	0	0	0	0	0.01	0	0.03	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 0:42	0	0.06	0	0.01	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 0:43	0	0.05	0	0.01	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.03	0
9/22/2017 0:44	0	0.04	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0
9/22/2017 0:45	0	0.02	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 0:46	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 0:47	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 0:48	0	0.11	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 0:49	0	0.02	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 0:50	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 0:51	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 0:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0.01
9/22/2017 0:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.03	0	0	0	0	0	0	0.01
9/22/2017 0:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.08	0	0	0	0	0	0	0.04
9/22/2017 0:57	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.07	0	0	0	0	0	0	0.05
9/22/2017 0:58	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.06	0	0	0	0	0.01	0	0.04
9/22/2017 0:59	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.07	0	0.05	0	0	0	0	0.01	0	0.02
9/22/2017 1:00	0.06	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0.03	0	0.04	0.09	0	0.05	0.05	0.04	0	0	0.01	0.01	0.02

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 1:01	0.08	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.03	0	0.05	0.07	0	0.04	0.06	0.04	0	0.05	0.01	0.03	0.02
9/22/2017 1:02	0.08	0	0	0	0	0	0	0	0	0	0	0	0.03	0.02	0.03	0	0.04	0.06	0.01	0.03	0.07	0.07	0	0.08	0	0.01	0.02
9/22/2017 1:03	0.07	0	0	0	0	0	0	0	0	0	0	0	0.03	0.02	0.04	0.07	0.03	0.06	0.01	0.03	0.08	0.1	0	0.09	0	0	0
9/22/2017 1:04	0.08	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0.01	0.08	0.04	0.07	0.01	0.01	0.08	0.1	0	0.08	0	0	0
9/22/2017 1:05	0.06	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0.01	0.09	0.04	0.05	0.01	0.01	0.07	0.06	0	0.08	0	0	0
9/22/2017 1:06	0.06	0.01	0	0.01	0.03	0	0	0	0	0	0	0	0.01	0.03	0.01	0.09	0.04	0.03	0.01	0.01	0.07	0.05	0	0.09	0	0.02	0
9/22/2017 1:07	0.03	0.01	0.03	0.01	0.02	0.03	0	0	0	0	0	0	0.06	0.02	0.03	0.08	0.03	0.03	0.04	0	0.05	0.05	0	0.09	0	0.03	0
9/22/2017 1:08	0.03	0.01	0.03	0.01	0.02	0.02	0	0	0	0	0	0	0.07	0.02	0.11	0.07	0.02	0.03	0.04	0	0.03	0.04	0	0.06	0	0.04	0
9/22/2017 1:09	0.02	0.01	0.02	0.01	0.02	0.02	0.03	0	0	0	0	0	0.07	0.06	0.07	0.05	0.02	0.03	0.04	0	0.01	0	0	0.04	0	0.03	0
9/22/2017 1:10	0.02	0.01	0.02	0.04	0.02	0.02	0.03	0	0	0	0	0	0.06	0.08	0.05	0.03	0.02	0.01	0.08	0	0.01	0.03	0.04	0.02	0	0.01	0
9/22/2017 1:11	0.02	0.04	0.05	0.08	0.02	0.02	0.02	0	0	0	0	0.03	0.05	0.09	0	0.03	0.01	0.01	0.04	0	0.01	0.01	0.04	0.02	0.01	0	0
9/22/2017 1:12	0.02	0.05	0.05	0.04	0.05	0.07	0.03	0.04	0	0.02	0	0.03	0.04	0.09	0.02	0.03	0.01	0.01	0.02	0	0.01	0.01	0.03	0	0.01	0	0
9/22/2017 1:13	0.02	0.04	0.04	0.03	0.07	0.08	0.03	0.04	0.03	0.02	0	0.04	0.04	0.09	0.02	0.01	0.01	0.01	0.02	0	0.01	0.01	0.03	0	0.01	0	0
9/22/2017 1:14	0.02	0.04	0.05	0.03	0.05	0.06	0.04	0.04	0.03	0.02	0	0.02	0.03	0.04	0.02	0.01	0	0.01	0.04	0	0.01	0.01	0.03	0	0.03	0.01	0
9/22/2017 1:15	0.02	0.02	0.05	0.05	0.05	0.06	0.05	0.03	0.04	0.02	0.02	0.02	0.02	0.02	0.02	0.01	0	0.01	0.02	0	0.01	0.01	0.06	0	0.03	0.01	0
9/22/2017 1:16	0.02	0.01	0.06	0.03	0.04	0.06	0.05	0.04	0.04	0.04	0.02	0.03	0.02	0.02	0.02	0.01	0	0.01	0.02	0	0.01	0.01	0.06	0	0.03	0.02	0
9/22/2017 1:17	0.01	0.01	0.07	0.03	0.04	0.05	0.04	0.04	0.04	0.04	0.02	0.03	0.01	0.02	0	0.01	0	0	0	0	0.01	0.01	0.04	0	0.03	0.01	0
9/22/2017 1:18	0.01	0.01	0.07	0.03	0.03	0.03	0.06	0.04	0.04	0.04	0.04	0.03	0.01	0.02	0	0.01	0	0	0	0	0.01	0	0.03	0	0.04	0.01	0
9/22/2017 1:19	0.01	0.01	0.03	0.02	0.02	0.03	0.04	0.04	0.04	0.05	0.04	0.07	0.01	0	0	0.01	0	0	0	0	0.01	0	0.03	0	0.06	0.01	0
9/22/2017 1:20	0.01	0.01	0.02	0.02	0.02	0.03	0.03	0.03	0.08	0.04	0.03	0.07	0.01	0	0	0.01	0	0	0	0	0.01	0	0.01	0	0.04	0.02	0
9/22/2017 1:21	0.01	0.01	0.02	0.01	0.02	0.02	0.03	0.02	0.03	0.04	0.04	0.06	0.01	0	0	0.01	0	0	0	0	0	0	0.01	0	0.03	0.04	0
9/22/2017 1:22	0.01	0.01	0.01	0.01	0.02	0.01	0.03	0.02	0.03	0.04	0.03	0.05	0.01	0	0	0.01	0	0	0	0	0	0	0.01	0	0.03	0.02	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 1:23	0.01	0.01	0.01	0.01	0.02	0.01	0.04	0.01	0.02	0.05	0.04	0.03	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0.02	0.01	0
9/22/2017 1:24	0	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.01	0.02	0.04	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0
9/22/2017 1:25	0	0.02	0.01	0	0.01	0.02	0.01	0.01	0.01	0.02	0.02	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/22/2017 1:26	0	0.02	0.01	0	0.01	0.02	0.01	0.01	0.01	0.02	0.02	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/22/2017 1:27	0	0.02	0.01	0	0.01	0.02	0.01	0	0.01	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/22/2017 1:28	0	0.01	0.01	0	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/22/2017 1:29	0	0	0.01	0	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/22/2017 1:30	0	0	0.01	0	0	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
9/22/2017 1:31	0	0	0.01	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:32	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:33	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:34	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 1:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
9/22/2017 1:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 1:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01
9/22/2017 1:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01
9/22/2017 1:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01
9/22/2017 1:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0.01
9/22/2017 1:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0.01
9/22/2017 1:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0.01
9/22/2017 1:58	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0.01
9/22/2017 1:59	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0.02
9/22/2017 2:00	0	0	0	0.01	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0	0.02
9/22/2017 2:01	0	0	0	0.01	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0	0.04
9/22/2017 2:02	0	0	0	0.01	0	0.01	0	0	0	0.01	0.01	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0	0.03
9/22/2017 2:03	0	0	0	0.01	0	0.01	0.01	0	0	0.01	0.01	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0	0.02
9/22/2017 2:04	0	0	0	0.01	0	0.01	0.01	0	0	0.01	0.01	0	0	0.01	0.01	0	0.01	0.01	0	0.02	0	0	0	0.01	0	0	0.02
9/22/2017 2:05	0	0	0	0	0	0.01	0.01	0	0	0.01	0.01	0	0	0.01	0	0	0.01	0.01	0	0.02	0	0	0	0.01	0	0	0.03
9/22/2017 2:06	0	0	0	0	0	0.01	0.01	0	0	0.01	0	0	0	0.01	0	0	0.03	0.01	0	0.04	0	0	0	0.01	0	0	0.03

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 2:07	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0.01	0	0	0.02	0.01	0	0.02	0	0	0	0.01	0	0	0.04
9/22/2017 2:08	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0.02	0.01	0	0.02	0	0.01	0	0.01	0	0	0.04
9/22/2017 2:09	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0.02	0.01	0	0.02	0	0.01	0.01	0.01	0	0	0.04
9/22/2017 2:10	0	0	0.01	0	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0	0.02	0.01	0	0.02	0	0.01	0.01	0.01	0	0	0.03
9/22/2017 2:11	0	0.01	0.01	0	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0.01	0.04	0.02	0	0.02	0	0.01	0.01	0.01	0	0	0
9/22/2017 2:12	0	0.01	0.01	0	0.01	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0.01	0	0.02	0	0.02	0.01	0.01	0.01	0.01	0	0	0.04
9/22/2017 2:13	0	0	0.01	0	0.01	0	0.01	0.01	0	0	0	0	0.01	0.01	0	0.01	0	0.03	0.01	0.02	0.01	0.01	0.01	0.01	0	0	0.04
9/22/2017 2:14	0	0	0.01	0	0.01	0	0.01	0.01	0.01	0	0	0	0.01	0.01	0.02	0.01	0.04	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.03
9/22/2017 2:15	0	0	0.01	0	0.01	0	0.01	0.01	0.01	0	0	0	0.02	0.01	0.02	0.01	0.05	0.05	0.01	0.01	0.02	0.01	0.01	0.03	0	0.02	0.05
9/22/2017 2:16	0	0	0.01	0	0.01	0	0.01	0.01	0.01	0	0.01	0	0.02	0.01	0.07	0.01	0.05	0.04	0.01	0.01	0.02	0.01	0.01	0.03	0	0.02	0.06
9/22/2017 2:17	0	0	0.01	0.02	0.01	0	0.01	0.01	0.01	0	0.01	0	0.02	0.01	0.04	0.06	0.04	0.04	0.04	0.01	0.01	0.01	0.01	0.02	0	0.02	0.06
9/22/2017 2:18	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0.02	0.03	0.06	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.02	0.01	0.03	0
9/22/2017 2:19	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.03	0.02	0.03	0.04	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.06
9/22/2017 2:20	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.02	0.02	0.01	0.01	0.05	0.02	0.04	0.03	0.04	0.02	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.07
9/22/2017 2:21	0	0.02	0.04	0.01	0.01	0.01	0.01	0	0.02	0.02	0.02	0.01	0.06	0.02	0.05	0.02	0.02	0.02	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.05
9/22/2017 2:22	0	0.02	0.02	0.01	0.01	0.01	0.01	0	0.02	0.02	0.01	0.02	0.05	0.02	0.05	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.04
9/22/2017 2:23	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0	0.01	0.02	0.01	0.02	0.05	0.04	0.03	0.02	0.03	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.02	0.04	0.03
9/22/2017 2:24	0.01	0.02	0.03	0.02	0.01	0.01	0.01	0.02	0.01	0.02	0.01	0.03	0.05	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.04	0.05	0.04
9/22/2017 2:25	0.01	0.04	0.03	0.02	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0.03	0.05	0.05	0.02	0.01	0.02	0.02	0.01	0.01	0	0.01	0.02	0.01	0.04	0.05	0.02
9/22/2017 2:26	0.01	0.02	0.03	0.02	0.01	0.01	0.01	0.03	0.01	0.01	0.02	0.03	0.04	0.05	0.02	0.01	0.02	0.02	0.01	0.01	0	0.01	0.02	0.01	0.03	0.05	0.02
9/22/2017 2:27	0.01	0.04	0.01	0.02	0.02	0.03	0.01	0.03	0.01	0.01	0.03	0.04	0.04	0.04	0.02	0.01	0.02	0.02	0.01	0.01	0	0.01	0.01	0.02	0.04	0.04	0.02
9/22/2017 2:28	0.01	0.04	0.01	0.02	0.02	0.03	0.03	0.02	0.03	0.01	0.03	0.03	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.04	0.06	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 2:29	0.02	0.04	0.01	0.02	0.02	0.03	0.03	0.02	0.03	0.04	0.04	0.01	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0	0.01	0.01	0.03	0.03	0.06	0
9/22/2017 2:30	0.04	0.05	0.01	0.02	0.02	0.01	0.03	0.02	0.02	0.04	0.04	0.01	0.02	0.03	0	0.02	0.01	0.01	0.01	0.01	0	0.01	0.01	0.03	0.03	0.03	0
9/22/2017 2:31	0.05	0.03	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.03	0.01	0.02	0.03	0.03	0.01	0.01	0.01	0.01	0.02	0	0.01	0.01	0.03	0.02	0.04	0
9/22/2017 2:32	0.05	0.03	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0.04	0.01	0.02	0.01	0.03	0.01	0	0.01	0.01	0.02	0	0.01	0.01	0.04	0.02	0.02	0
9/22/2017 2:33	0.06	0.03	0.03	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0.04	0.04	0.01	0.01	0.01	0.01	0	0	0.01	0.02	0.02	0.01	0.01	0.06	0.02	0.01	0
9/22/2017 2:34	0.04	0.02	0.03	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0.03	0	0.01	0.01	0.01	0.01	0	0	0.01	0.02	0.03	0.04	0.01	0.04	0.02	0.01	0
9/22/2017 2:35	0.04	0.02	0.03	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.02	0.03	0.01	0.01	0.01	0.01	0	0	0.01	0.03	0.03	0.06	0.01	0.03	0.02	0.01	0
9/22/2017 2:36	0.04	0.01	0.03	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.02	0.03	0	0.01	0.01	0.01	0	0	0.01	0.05	0.03	0.06	0.01	0.03	0.03	0.01	0
9/22/2017 2:37	0.01	0.01	0.03	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.02	0.02	0	0.01	0.01	0.01	0	0	0.01	0.05	0.03	0.06	0.01	0.05	0.05	0	0
9/22/2017 2:38	0.01	0.01	0.04	0.01	0.01	0.01	0.02	0.02	0.01	0.01	0.02	0.02	0	0.01	0.01	0.01	0	0	0.01	0.04	0.04	0.06	0.02	0.04	0.05	0	0
9/22/2017 2:39	0.01	0.01	0.04	0.01	0.01	0.01	0.02	0.02	0.01	0.03	0.02	0.02	0	0.01	0.01	0.01	0	0	0.01	0.03	0.03	0.05	0.02	0.03	0.04	0	0
9/22/2017 2:40	0.01	0.01	0.04	0.01	0.01	0.01	0.02	0.02	0.01	0.03	0.02	0.02	0	0.01	0.01	0.01	0	0	0.01	0.03	0.04	0.06	0.03	0.03	0.04	0	0
9/22/2017 2:41	0.01	0	0.03	0.01	0.01	0.01	0.02	0.02	0.01	0.03	0.02	0.02	0	0.01	0.03	0.01	0	0	0.01	0.02	0.03	0.05	0.03	0.02	0.03	0	0
9/22/2017 2:42	0.01	0	0.03	0.01	0.01	0.01	0.02	0.02	0.03	0.05	0.02	0.02	0	0.01	0.03	0.01	0	0	0.01	0.02	0.03	0.04	0.03	0.02	0.05	0	0.01
9/22/2017 2:43	0.01	0	0.03	0.01	0.01	0.01	0.02	0.02	0.03	0.04	0.03	0.02	0	0.01	0.04	0.01	0	0	0.01	0.01	0.03	0.04	0.03	0.02	0.08	0	0.01
9/22/2017 2:44	0.01	0	0.03	0.01	0.02	0.01	0.02	0.03	0.04	0.04	0.05	0.02	0	0.01	0.04	0.02	0.01	0.02	0.02	0.01	0.04	0.03	0.03	0.02	0.05	0	0.01
9/22/2017 2:45	0.01	0	0.05	0.02	0.02	0.02	0.02	0.03	0.04	0.04	0.05	0.02	0	0.01	0.04	0.02	0.01	0.02	0.02	0.01	0.02	0.03	0.05	0.03	0.04	0	0.01
9/22/2017 2:46	0.01	0	0.02	0.03	0.02	0.02	0.04	0.04	0.04	0.04	0.06	0.02	0	0.01	0.05	0.05	0.01	0.03	0.02	0.01	0.02	0.04	0.04	0.02	0.04	0	0.01
9/22/2017 2:47	0.01	0	0.02	0.03	0.02	0.02	0.04	0.04	0.09	0.03	0.05	0.03	0.01	0.02	0.04	0.04	0.01	0.03	0.02	0.01	0.02	0.02	0.03	0.02	0.04	0	0.01
9/22/2017 2:48	0.01	0.01	0.02	0.05	0.02	0.02	0.05	0.04	0.04	0.05	0.05	0.05	0.01	0.02	0.05	0.03	0.02	0.01	0.04	0.01	0.02	0.02	0.05	0.02	0.04	0	0.01
9/22/2017 2:49	0.01	0.01	0.01	0.07	0.04	0.03	0.05	0.04	0.04	0.05	0.03	0.06	0.01	0.02	0.06	0.02	0.02	0.01	0.04	0.01	0.02	0.02	0.04	0.02	0.04	0	0.01
9/22/2017 2:50	0.01	0.02	0.01	0.06	0.04	0.05	0.05	0.04	0.04	0.05	0.03	0.06	0.01	0.02	0.04	0.02	0.02	0.01	0.02	0.01	0.02	0.02	0.03	0.01	0.02	0	0.01

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 2:51	0.01	0.02	0.01	0.05	0.05	0.04	0.04	0.04	0.04	0.03	0.04	0.06	0.01	0.01	0.05	0.02	0.02	0.01	0.02	0.01	0.02	0.01	0.05	0.01	0.03	0.01	0.01
9/22/2017 2:52	0.01	0.01	0.02	0.04	0.05	0.04	0.05	0.04	0.04	0.03	0.03	0.05	0.01	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.01	0.03	0.01	0.01
9/22/2017 2:53	0.01	0.01	0.02	0.05	0.04	0.05	0.05	0.04	0.04	0.03	0.03	0.05	0.01	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.03	0.02	0.01
9/22/2017 2:54	0	0.01	0.02	0.04	0.04	0.04	0.04	0.04	0.04	0.03	0.02	0.06	0.01	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.05	0.01	0.04	0.01	0.01
9/22/2017 2:55	0	0.01	0.02	0.04	0.04	0.02	0.04	0.04	0.03	0.02	0.02	0.05	0.01	0.01	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.01	0.04	0.01	0.01
9/22/2017 2:56	0	0.01	0.02	0.04	0.04	0.02	0.03	0.03	0.03	0.02	0.02	0.04	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.04	0.01	0
9/22/2017 2:57	0	0.01	0.02	0.03	0.03	0.11	0.02	0.02	0.02	0.02	0.02	0.04	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0
9/22/2017 2:58	0	0.01	0.02	0.03	0.03	0.04	0.02	0.02	0.02	0.03	0.03	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0
9/22/2017 2:59	0	0.02	0.01	0.02	0.03	0.02	0.01	0.02	0.03	0.02	0.03	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:00	0	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:02	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0
9/22/2017 3:03	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0
9/22/2017 3:04	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0
9/22/2017 3:05	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:06	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:07	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:08	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:09	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0
9/22/2017 3:10	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.01	0.01	0.01	0.02	0.01	0.02	0
9/22/2017 3:11	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01	0.01	0.02	0.01	0.02	0.01	0.02	0.01	0.02	0
9/22/2017 3:12	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 3:13	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01
9/22/2017 3:14	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01
9/22/2017 3:15	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01
9/22/2017 3:16	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0.01	0.01	0.01
9/22/2017 3:17	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0.03	0.01	0.01	0.01
9/22/2017 3:18	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.02	0.03	0.02	0.03	0.01	0.01	0.01
9/22/2017 3:19	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0.02	0.01	0.01	0.02	0.02	0.02	0.03	0.01	0.01	0.01
9/22/2017 3:20	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0	0.02	0.01	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01
9/22/2017 3:21	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01
9/22/2017 3:22	0.01	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.01	0	0.01	0	0.01	0.01	0.01	0	0.01
9/22/2017 3:23	0.01	0	0.03	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0	0.01	0	0.02	0.01	0.01	0.01	0.01
9/22/2017 3:24	0.01	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0	0.01	0	0.02	0.01	0.01	0	0.01
9/22/2017 3:25	0	0.01	0.04	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0	0.01	0	0.02	0.01	0.01	0	0.01
9/22/2017 3:26	0	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0.02	0	0.01	0	0.03	0.01	0.01	0	0
9/22/2017 3:27	0	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0	0	0	0.04	0	0.03	0	0
9/22/2017 3:28	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0	0	0	0.01	0	0.03	0	0
9/22/2017 3:29	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0	0	0	0.01	0	0.03	0	0
9/22/2017 3:30	0	0.01	0.01	0.02	0.01	0.01	0.01	0.03	0.01	0.02	0.01	0.01	0.02	0.01	0.03	0	0	0.01	0.01	0	0	0	0.01	0	0.01	0	0
9/22/2017 3:31	0	0.01	0.01	0.02	0.01	0.01	0.03	0.03	0.03	0.02	0.02	0.02	0.02	0.01	0.03	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0
9/22/2017 3:32	0	0.01	0.01	0.03	0.01	0.03	0.03	0.01	0.03	0.02	0.02	0.02	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0
9/22/2017 3:33	0	0.01	0.01	0.02	0.02	0.03	0.01	0.01	0.03	0.01	0.02	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0.02	0
9/22/2017 3:34	0	0	0.01	0.02	0.02	0.03	0.01	0.01	0	0.01	0.02	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0.01	0

Date	PS8	IN 14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 10	BR 11	BR 12	IN 11	IN 12	IN 16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 08	TU 09	IN 08	ML 04
9/22/2017 3:35	0	0	0	0	0.02	0.01	0.01	0.01	0	0.01	0	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 3:36	0	0.01	0	0	0	0.01	0	0.01	0	0.01	0	0.02	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 3:37	0	0.01	0	0	0	0.01	0	0.01	0	0	0	0.02	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 3:38	0	0.03	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 3:39	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 3:40	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 3:41	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 3:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0
9/22/2017 3:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 3:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0
9/22/2017 3:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M.2.4 MAY 2020 HISTORIC EVENT

Table M.2.4. May 2020 Historic Event Hyetograph Data.

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 7:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:53	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:54	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:55	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:56	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:57	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:58	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 7:59	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:00	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:02	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:03	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:04	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 8:05	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:06	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:07	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:08	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:09	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:10	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:11	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:12	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:13	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:14	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:21	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:22	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:23	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:24	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:25	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:26	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 8:27	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:28	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:29	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:30	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:31	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:32	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:33	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:34	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:35	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:36	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:37	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:38	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:39	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:40	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:41	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:42	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:43	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:44	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:45	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:46	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:47	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:48	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 8:49	0	0	0	0	0	0	0	0	0	0	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:50	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:51	0	0	0	0	0	0	0.01	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:52	0	0	0	0	0.01	0	0.01	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:53	0	0	0	0	0.01	0	0.01	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:54	0	0	0	0	0.01	0	0.01	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:55	0	0	0	0	0.01	0.01	0.01	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:56	0	0	0	0	0.01	0.01	0.01	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 8:57	0	0	0	0	0.01	0.01	0.01	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0
5/27/2020 8:58	0	0	0	0	0.01	0.01	0.01	0	0.02	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.06	0
5/27/2020 8:59	0	0	0	0	0.01	0.01	0.01	0	0.02	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.08	0
5/27/2020 9:00	0	0	0	0	0.01	0.01	0.01	0	0.03	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.08	0
5/27/2020 9:01	0	0	0	0.02	0.01	0.01	0.01	0	0.03	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.07	0
5/27/2020 9:02	0	0	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.07	0
5/27/2020 9:03	0	0	0	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0.07	0
5/27/2020 9:04	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.1	0
5/27/2020 9:05	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0.03	0.1	0
5/27/2020 9:06	0	0	0	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0.03	0.03	0
5/27/2020 9:07	0	0	0	0.01	0.01	0.01	0.01	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0
5/27/2020 9:08	0	0	0	0	0.01	0.01	0.01	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0
5/27/2020 9:09	0	0	0	0	0	0.01	0.01	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:10	0	0	0	0	0	0.01	0.01	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 9:11	0	0	0	0	0	0.01	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0
5/27/2020 9:12	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:13	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:14	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:15	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:16	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:17	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:18	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:19	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:20	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0
5/27/2020 9:21	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 9:22	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 9:23	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 9:24	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 9:25	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 9:26	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:27	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:28	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:29	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:30	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:31	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:32	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 9:33	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 9:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 10:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 10:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 10:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 10:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 10:57	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 10:58	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 10:59	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 11:00	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 11:01	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 11:02	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:03	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:04	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:05	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:06	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:07	0	0.01	0	0	0	0	0.01	0.01	0	0.01	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:08	0	0.01	0	0	0	0	0.01	0.01	0	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:09	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 11:10	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.01	0	0
5/27/2020 11:11	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.01	0	0
5/27/2020 11:12	0	0.01	0	0.01	0	0.01	0.03	0.01	0.01	0.01	0.02	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.01	0	0
5/27/2020 11:13	0	0.01	0	0.01	0.02	0.01	0.02	0.01	0.01	0.02	0.02	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.01	0	0
5/27/2020 11:14	0	0.01	0	0.01	0.02	0.01	0.02	0.01	0.01	0.02	0.02	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.01	0	0
5/27/2020 11:15	0	0.01	0	0.02	0.02	0.02	0.02	0.01	0.02	0.04	0.02	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0.01	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 11:16	0	0	0	0.02	0.01	0.02	0.03	0.01	0.02	0.04	0.02	0.01	0.01	0	0	0	0	0.01	0	0	0	0.01	0.01	0	0
5/27/2020 11:17	0	0	0	0.01	0.01	0.02	0.02	0.01	0.02	0.04	0.01	0.01	0.01	0	0	0	0	0.01	0	0	0	0.01	0.01	0	0
5/27/2020 11:18	0	0	0	0.01	0.01	0.02	0.02	0.01	0.02	0.03	0.01	0.01	0.01	0	0	0	0	0.01	0	0	0	0.01	0.01	0	0
5/27/2020 11:19	0	0.01	0	0.01	0.02	0.02	0.03	0.01	0.02	0.03	0.01	0.01	0.01	0.02	0	0	0	0.01	0	0.01	0	0.01	0.01	0	0
5/27/2020 11:20	0	0.02	0	0.01	0.02	0.02	0.02	0.02	0.02	0.03	0.01	0.01	0.01	0.02	0	0	0	0.01	0	0.01	0	0.01	0.01	0	0
5/27/2020 11:21	0	0.02	0	0.01	0.02	0.02	0.02	0.02	0.02	0.03	0.01	0.01	0.01	0.01	0	0	0	0.01	0	0.01	0	0.01	0.01	0.01	0
5/27/2020 11:22	0	0.01	0	0.01	0.02	0.02	0.03	0.01	0.02	0.01	0.01	0	0.01	0.01	0	0	0	0.01	0	0.01	0	0.01	0.01	0	0
5/27/2020 11:23	0	0.01	0	0.02	0.02	0.02	0.03	0.01	0.02	0.01	0.01	0	0.01	0.01	0	0	0	0.01	0	0.01	0	0.01	0.01	0	0
5/27/2020 11:24	0	0.01	0	0.02	0.02	0.02	0.03	0.01	0.03	0.01	0.01	0	0.01	0.01	0	0	0	0.01	0	0.01	0	0.01	0.01	0	0
5/27/2020 11:25	0	0.01	0	0.02	0.02	0.02	0.02	0.02	0.03	0.01	0.01	0	0.01	0.01	0	0	0	0.01	0.01	0.01	0	0.01	0.01	0.01	0
5/27/2020 11:26	0	0.01	0	0.01	0.02	0.01	0.02	0.02	0.03	0.01	0.01	0	0.01	0.01	0	0	0	0.01	0.01	0.01	0	0.01	0.01	0.01	0
5/27/2020 11:27	0	0.01	0	0.01	0.02	0.01	0.01	0.01	0.03	0.01	0.01	0	0.02	0.01	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
5/27/2020 11:28	0	0.01	0	0.01	0.02	0.01	0.01	0.01	0.02	0.01	0.01	0	0.02	0.01	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0
5/27/2020 11:29	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0
5/27/2020 11:30	0	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0
5/27/2020 11:31	0	0.01	0	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01
5/27/2020 11:32	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.02	0	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01
5/27/2020 11:33	0	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0.01	0.02	0	0.01	0.02	0.01	0.01	0.01	0	0.01	0.02	0.01	0.01	0.01
5/27/2020 11:34	0	0.01	0	0.01	0.01	0.01	0.01	0	0.01	0.02	0	0.01	0.01	0	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0	0.01
5/27/2020 11:35	0	0.01	0	0.01	0.01	0.01	0.01	0	0.01	0.02	0	0.01	0.01	0	0.01	0.02	0	0.01	0.01	0.01	0.01	0.02	0.01	0	0.01
5/27/2020 11:36	0	0	0	0.01	0.01	0.01	0.01	0	0.02	0.02	0	0.01	0.01	0	0	0.02	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01
5/27/2020 11:37	0	0	0	0.01	0.01	0.01	0.01	0.04	0.02	0.03	0	0.01	0.01	0	0	0.02	0	0.01	0.01	0.01	0	0.01	0.01	0	0.01

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 11:38	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0.01	0.01	0	0	0.01	0	0.01	0.01	0.01	0	0.01	0.01	0	0.01
5/27/2020 11:39	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0.01	0	0	0	0.01	0	0.01	0	0.01	0	0.01	0.02	0	0.01
5/27/2020 11:40	0	0	0	0	0.01	0	0.01	0.01	0.01	0.02	0	0.01	0	0	0	0.01	0	0.01	0	0.01	0	0.01	0.02	0	0.01
5/27/2020 11:41	0	0	0	0	0	0	0.01	0.01	0.01	0.02	0	0.01	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0.01
5/27/2020 11:42	0	0	0	0	0	0	0.01	0.01	0.04	0.03	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.01
5/27/2020 11:43	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01
5/27/2020 11:44	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01
5/27/2020 11:45	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01
5/27/2020 11:46	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01
5/27/2020 11:47	0	0	0	0	0	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01
5/27/2020 11:48	0	0	0	0	0	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01
5/27/2020 11:49	0	0	0	0	0	0	0	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0.01
5/27/2020 11:50	0	0	0	0	0	0	0	0.01	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0.01
5/27/2020 11:51	0	0	0	0	0	0	0	0.01	0.01	0.03	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0.01
5/27/2020 11:52	0	0	0	0	0	0	0	0.01	0.01	0.03	0	0	0	0.01	0	0	0	0	0	0	0	0.01	0.01	0	0.01
5/27/2020 11:53	0	0	0	0	0	0	0	0.01	0.01	0.02	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0
5/27/2020 11:54	0	0	0	0	0	0	0.01	0.01	0.01	0.02	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:55	0	0	0	0	0	0	0.01	0.01	0.01	0.04	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:56	0	0	0	0	0	0	0.01	0.01	0.01	0.03	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:57	0	0	0	0	0	0	0.01	0.01	0.01	0.04	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:58	0	0	0	0	0	0	0.01	0.01	0.01	0.04	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 11:59	0	0	0	0	0	0	0.01	0.01	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 12:00	0	0	0	0	0	0	0.01	0.01	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:01	0	0	0	0	0	0	0.02	0.01	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:02	0	0	0	0	0	0	0.02	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:03	0	0	0	0	0	0	0.02	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:04	0	0	0	0	0	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:05	0	0	0	0	0	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:06	0	0	0	0	0	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:07	0	0	0	0	0	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:08	0	0	0	0	0	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:09	0	0	0	0	0	0	0.01	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:10	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:11	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:12	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:13	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:14	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:15	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:16	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:17	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:18	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:19	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:20	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 12:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 12:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 17:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0
5/27/2020 17:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 17:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0
5/27/2020 17:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0
5/27/2020 18:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0.01	0	0	0
5/27/2020 18:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0.01	0	0	0
5/27/2020 18:02	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0.01	0	0	0
5/27/2020 18:03	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0
5/27/2020 18:04	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0
5/27/2020 18:05	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0
5/27/2020 18:06	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0
5/27/2020 18:07	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.02	0.01	0	0	0	0	0
5/27/2020 18:08	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0.01	0	0	0	0	0
5/27/2020 18:09	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0	0	0	0	0	0
5/27/2020 18:10	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 18:11	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0
5/27/2020 18:12	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0	0	0.01	0	0	0	0	0	0
5/27/2020 18:13	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0	0	0.01	0	0	0	0	0	0
5/27/2020 18:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0
5/27/2020 18:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/27/2020 18:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/27/2020 18:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/27/2020 18:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0	0	0	0	0	0	0	0
5/27/2020 18:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0.02	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 18:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0
5/27/2020 18:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0
5/27/2020 18:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:40	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 18:41	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 18:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 18:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0.01	0	0	0	0
5/27/2020 19:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0.01	0	0	0	0
5/27/2020 19:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0.01	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 19:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0.01	0	0	0	0
5/27/2020 19:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0
5/27/2020 19:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 19:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 19:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 19:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:42	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:43	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:44	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:45	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 19:46	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 19:47	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 19:48	0	0	0	0.01	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:49	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:50	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:51	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:52	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:53	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:54	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:55	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:56	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:57	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:58	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 19:59	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:04	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:05	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 20:06	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 20:07	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:08	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:09	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 20:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 20:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 20:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:53	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 20:54	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:55	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:56	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 20:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 20:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 20:59	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:00	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:02	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0
5/27/2020 21:03	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0
5/27/2020 21:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 21:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:30	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:31	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:32	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:33	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:34	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:35	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:36	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:37	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 21:38	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:39	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:40	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 21:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:55	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:56	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:57	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:58	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 21:59	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 22:00	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:02	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:03	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 22:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 22:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 22:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.03	0.02	0	0	0
5/27/2020 22:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.02	0	0	0
5/27/2020 22:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0
5/27/2020 22:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0
5/27/2020 22:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0.01	0	0	0	0
5/27/2020 22:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0
5/27/2020 22:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0
5/27/2020 22:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:58	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 22:59	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:00	0	0	0	0	0	0	0	0	0	0	0	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 23:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:07	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:08	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:09	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:10	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:11	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:12	0.01	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:13	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:14	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/27/2020 23:15	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0.01
5/27/2020 23:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0	0	0.01
5/27/2020 23:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.02	0	0	0	0.01	0	0	0.01
5/27/2020 23:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.02	0	0	0	0.01	0	0	0.01
5/27/2020 23:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0.01	0.01	0	0	0.01
5/27/2020 23:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0.01	0.01	0	0	0.01
5/27/2020 23:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0.01	0.01	0.01	0	0	0.01
5/27/2020 23:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0.01	0.03	0.01	0.02	0	0.01
5/27/2020 23:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0.03	0.01	0.02	0	0.01
5/27/2020 23:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.02	0.03	0.02	0.06	0	0
5/27/2020 23:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0.02	0.02	0.01	0.02	0.05	0	0
5/27/2020 23:27	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0.03	0	0.01	0	0.02	0.02	0.01	0	0.03	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 23:28	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0.05	0	0.01	0	0.02	0.02	0.01	0	0.03	0	0
5/27/2020 23:29	0	0	0	0	0	0	0	0.03	0.04	0	0	0	0	0	0.02	0.02	0.01	0	0.03	0	0	0	0.03	0	0
5/27/2020 23:30	0	0	0	0	0	0	0.03	0.02	0.04	0.01	0	0	0	0	0.02	0.02	0.01	0	0.01	0	0	0	0	0	0
5/27/2020 23:31	0	0	0	0	0.03	0.03	0.05	0.02	0.04	0.01	0	0	0	0	0	0.02	0	0	0.01	0	0	0	0	0	0
5/27/2020 23:32	0	0	0	0	0.03	0.05	0.04	0	0.04	0.01	0	0.02	0.02	0	0	0.01	0	0	0.01	0	0	0	0	0	0
5/27/2020 23:33	0	0	0	0	0.03	0.03	0.05	0	0.04	0.02	0.01	0.02	0.02	0	0	0.01	0	0	0.01	0	0	0	0	0	0
5/27/2020 23:34	0	0	0	0	0.03	0.03	0.06	0	0.01	0.02	0.01	0.02	0.04	0.01	0	0.01	0	0	0.01	0	0	0	0	0	0
5/27/2020 23:35	0	0	0	0	0.03	0.04	0.01	0	0.01	0.04	0.01	0.02	0.04	0.01	0	0.01	0	0	0	0	0	0	0	0	0
5/27/2020 23:36	0	0	0	0	0.04	0.06	0.01	0	0.01	0.03	0.01	0.01	0.02	0.01	0	0.03	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:37	0	0	0	0	0.04	0.04	0.01	0.04	0.01	0.03	0.01	0.01	0.02	0.01	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:38	0	0.03	0	0	0.03	0.01	0	0	0	0	0.02	0.01	0.01	0.02	0	0	0	0	0	0	0	0	0	0.02	0
5/27/2020 23:39	0	0.03	0	0	0	0.01	0	0	0	0	0.02	0	0.01	0.02	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:40	0	0.02	0	0	0	0.01	0	0	0	0	0.02	0	0.01	0.03	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:41	0	0.02	0	0	0	0	0	0	0	0	0.02	0	0	0.03	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:42	0	0.03	0	0	0	0	0	0	0	0	0.02	0	0	0.02	0	0	0	0	0	0	0	0	0	0.01	0
5/27/2020 23:43	0	0.03	0	0	0	0	0	0	0	0	0.02	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:44	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:45	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:46	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:47	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:48	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01
5/27/2020 23:49	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/27/2020 23:50	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.06
5/27/2020 23:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.06
5/27/2020 23:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.03
5/27/2020 23:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/27/2020 23:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/28/2020 0:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/28/2020 0:01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/28/2020 0:02	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/28/2020 0:03	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/28/2020 0:04	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
5/28/2020 0:05	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:06	0.01	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:07	0.01	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0
5/28/2020 0:08	0.01	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0
5/28/2020 0:09	0.01	0	0	0	0	0.01	0.12	0	0.01	0.01	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:10	0.01	0	0	0	0	0.01	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:11	0.01	0	0	0	0.02	0.01	0	0	0.01	0.01	0	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0.01

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 0:12	0.01	0	0	0	0.02	0	0	0	0.01	0.01	0.01	0	0	0	0.01	0.01	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:13	0	0	0	0	0	0	0	0	0.01	0.01	0.01	0	0	0	0.01	0.01	0.03	0	0	0	0	0	0	0	0.01
5/28/2020 0:14	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0.01	0.03	0.02	0	0	0	0	0	0	0	0.01
5/28/2020 0:15	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0.01	0.02	0.02	0	0	0	0	0	0	0	0.01
5/28/2020 0:16	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0.01	0.02	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:17	0	0.01	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0	0.02	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:18	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:19	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:20	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0.01	0	0.01	0.01	0	0	0	0	0	0	0	0.01
5/28/2020 0:21	0	0.03	0	0	0	0	0	0	0	0.02	0.02	0	0	0.01	0	0.01	0.01	0.03	0	0	0.01	0	0	0	0.01
5/28/2020 0:22	0	0	0	0	0	0	0	0	0	0.02	0.02	0	0	0.01	0	0.01	0.01	0.03	0	0	0.01	0	0	0	0.01
5/28/2020 0:23	0	0	0	0	0	0	0	0	0	0.03	0.01	0	0	0.01	0	0.01	0.01	0.05	0	0	0.01	0	0	0.02	0.01
5/28/2020 0:24	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0.03	0	0	0.01	0	0	0.02	0.01
5/28/2020 0:25	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0.03	0	0.03	0.01	0	0	0.01	0.01
5/28/2020 0:26	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0.03	0	0.04	0.02	0	0	0	0.01
5/28/2020 0:27	0	0.01	0	0	0	0	0	0	0	0	0.02	0	0	0.01	0	0	0	0.02	0.04	0.03	0.02	0	0	0	0
5/28/2020 0:28	0	0.01	0	0	0	0	0	0	0	0	0.02	0	0	0.01	0.01	0	0	0.02	0.06	0.03	0.01	0	0	0	0
5/28/2020 0:29	0	0.01	0	0	0	0	0	0	0	0	0.04	0	0	0.01	0.01	0	0	0.05	0.03	0.04	0.01	0	0	0	0
5/28/2020 0:30	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0	0	0.01	0.04	0.02	0.01	0	0	0	0
5/28/2020 0:31	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0	0	0.01	0.01	0.02	0.01	0	0	0	0
5/28/2020 0:32	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0.01	0	0	0.01	0.01	0.03	0.01	0	0	0	0
5/28/2020 0:33	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0.01	0.01	0.01	0.01	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 0:34	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0.01	0.04	0.01	0.01	0	0	0	0
5/28/2020 0:35	0	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0.01	0	0	0	0.01	0.01	0.01	0.01	0	0	0.01	0
5/28/2020 0:36	0	0.01	0	0	0	0	0	0	0	0	0.01	0.01	0	0.01	0	0	0	0.01	0.01	0.01	0.01	0	0	0.01	0
5/28/2020 0:37	0	0.01	0	0	0	0	0	0	0	0	0.01	0.01	0	0.01	0	0	0	0	0.01	0.01	0.01	0	0	0	0
5/28/2020 0:38	0	0.01	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0
5/28/2020 0:39	0	0.01	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/28/2020 0:40	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0
5/28/2020 0:41	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0.01	0	0	0	0	0.03	0
5/28/2020 0:42	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 0:43	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 0:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02	0
5/28/2020 0:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 0:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 0:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 0:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 0:59	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:00	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:01	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:02	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:03	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:04	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:05	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:06	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:07	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:08	0	0	0	0	0	0	0	0	0	0	0.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:09	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:10	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:11	0	0	0	0	0	0	0	0	0	0	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:12	0	0	0	0	0	0	0	0	0	0	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:13	0	0	0	0	0	0	0	0	0	0	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:14	0	0	0	0	0	0	0	0	0	0	0.07	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:15	0	0	0	0	0	0	0	0	0	0	0.05	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:16	0	0	0	0	0	0	0	0	0	0.01	0.06	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:17	0	0	0	0	0	0	0	0	0	0.01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 1:18	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:19	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:20	0	0	0	0	0	0	0	0	0	0.01	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/28/2020 1:21	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/28/2020 1:22	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/28/2020 1:23	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0
5/28/2020 1:24	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:25	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:26	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:27	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:28	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:29	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:30	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:33	0	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:34	0	0.03	0	0	0.01	0.01	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 1:35	0	0.01	0	0	0.01	0.01	0	0	0.04	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0.02	0	0
5/28/2020 1:36	0	0.01	0	0	0.01	0.02	0.02	0	0.04	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0.02	0	0
5/28/2020 1:37	0	0.01	0	0	0.01	0.02	0.02	0	0.04	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.02	0	0
5/28/2020 1:38	0	0.01	0	0	0.01	0.02	0.05	0	0.04	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.02	0	0
5/28/2020 1:39	0	0.02	0	0	0.01	0.04	0.04	0	0.04	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.02	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 1:40	0	0.02	0	0	0.03	0.05	0.04	0	0.04	0.01	0	0	0	0.02	0	0	0	0	0	0	0	0	0.02	0	0
5/28/2020 1:41	0	0.03	0	0	0.03	0.05	0.04	0	0.01	0.01	0	0	0	0.02	0	0	0	0	0	0	0	0	0.02	0	0
5/28/2020 1:42	0	0.01	0	0	0.04	0.05	0.03	0.01	0.01	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0
5/28/2020 1:43	0	0.02	0	0	0.04	0.04	0.03	0.01	0.01	0.01	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0
5/28/2020 1:44	0	0.03	0	0	0.03	0.04	0.02	0.01	0.01	0.04	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0
5/28/2020 1:45	0	0.08	0	0	0.03	0.04	0.02	0.01	0.02	0.04	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01	0	0
5/28/2020 1:46	0	0.02	0	0	0.04	0.03	0.02	0.01	0.02	0.04	0	0	0	0.01	0	0	0	0.04	0	0	0	0	0.01	0	0
5/28/2020 1:47	0	0.02	0	0	0.03	0.05	0.02	0.01	0.03	0.04	0	0	0	0.01	0	0	0	0.03	0	0	0	0	0.01	0	0
5/28/2020 1:48	0	0.04	0	0	0.02	0.02	0.02	0.01	0.01	0.02	0.01	0	0	0.01	0	0	0	0.03	0	0	0	0	0.02	0	0
5/28/2020 1:49	0	0.03	0	0	0.02	0.02	0.02	0.01	0.01	0.02	0.01	0	0	0.01	0.04	0	0	0.04	0	0	0	0	0.02	0	0
5/28/2020 1:50	0	0	0	0	0.02	0.02	0.02	0.01	0.01	0.03	0.01	0	0	0.01	0.05	0	0	0.04	0	0.04	0.03	0	0.02	0	0
5/28/2020 1:51	0	0	0	0	0.02	0.02	0.02	0.01	0.02	0.03	0.01	0	0	0.01	0.05	0	0	0.1	0.05	0.08	0.05	0	0.09	0	0
5/28/2020 1:52	0	0.02	0	0	0.01	0.02	0.02	0.01	0.02	0.03	0.02	0	0	0.01	0.02	0	0	0.05	0.06	0.09	0.03	0	0.08	0	0
5/28/2020 1:53	0	0.01	0	0	0.01	0.02	0.02	0.01	0.02	0.04	0.02	0	0	0.03	0.04	0	0	0.06	0.11	0.06	0.06	0	0.06	0	0
5/28/2020 1:54	0	0.02	0	0	0.01	0.01	0.02	0.01	0.02	0.04	0.02	0	0.02	0.03	0.04	0	0	0.04	0.09	0.05	0.06	0.05	0.06	0	0.02
5/28/2020 1:55	0	0.04	0	0	0.01	0.01	0.03	0.04	0.04	0.04	0.02	0.02	0.02	0.03	0.03	0.01	0	0.04	0.06	0.08	0.07	0.03	0.04	0	0.02
5/28/2020 1:56	0	0.02	0	0	0.01	0.01	0.03	0.04	0.04	0.04	0.02	0.02	0.06	0.03	0.03	0.01	0	0.03	0.06	0.06	0	0.03	0.02	0	0.03
5/28/2020 1:57	0	0.02	0	0	0.01	0.01	0.05	0.04	0.03	0.04	0.05	0.08	0.03	0.04	0.03	0.01	0	0.03	0.09	0.07	0.02	0.03	0.02	0	0.02
5/28/2020 1:58	0	0.01	0	0	0.01	0.03	0.03	0.01	0.03	0.04	0.05	0.05	0.02	0.05	0.02	0.01	0	0.03	0.08	0.05	0.02	0.03	0.01	0	0.02
5/28/2020 1:59	0	0.01	0	0	0.04	0.04	0.03	0.01	0.01	0.04	0.05	0.04	0.02	0.05	0.02	0.01	0	0.03	0.04	0.05	0.02	0.02	0.01	0	0.02
5/28/2020 2:00	0	0.01	0	0	0.04	0.03	0.02	0.01	0.01	0.04	0.05	0.06	0.04	0.04	0.01	0.01	0	0.03	0.05	0.02	0.04	0.02	0.01	0.04	0.02
5/28/2020 2:01	0	0.02	0	0	0.04	0.03	0.02	0.01	0.01	0.02	0.05	0.05	0.04	0.03	0.01	0.01	0	0.03	0.03	0.02	0.02	0.02	0.01	0.09	0.04

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 2:02	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.03	0.05	0.05	0.06	0.04	0.01	0.01	0	0.03	0.03	0.02	0.02	0.02	0.01	0.08	0
5/28/2020 2:03	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.08	0.03	0.03	0.02	0.02	0	0.01	0.02	0.02	0.04	0.02	0.01	0.07	0
5/28/2020 2:04	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.04	0.01	0.03	0.02	0.02	0.05	0.01	0.05	0.02	0.04	0.02	0.01	0.04	0
5/28/2020 2:05	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.02	0.08	0.01	0.05	0.03	0.01	0.01	0.02	0.03	0
5/28/2020 2:06	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.04	0.05	0.01	0.05	0.01	0.01	0.01	0.02	0.05	0
5/28/2020 2:07	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.02	0	0.02	0.02	0.01	0.01	0.01	0.03	0.05	0
5/28/2020 2:08	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.02	0.02	0.01	0.01	0.01	0.05	0.05	0
5/28/2020 2:09	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.04	0.02	0.02	0.02	0.01	0.03	0.02	0
5/28/2020 2:10	0	0.01	0	0	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.02	0.04	0.02	0.01	0.02	0.02	0.02	0.02	0.01	0.03	0.01	0
5/28/2020 2:11	0	0.01	0	0	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.02	0.04	0.01	0.04	0.02	0.02	0.02	0.02	0.01	0.03	0.02	0
5/28/2020 2:12	0	0.01	0	0	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.04	0.02	0.01	0.02	0.04	0.01	0.01	0.01	0
5/28/2020 2:13	0	0.01	0	0	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0.01	0.02	0.03	0.01	0.01	0.01	0.01
5/28/2020 2:14	0	0	0	0	0.01	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0	0.01	0.04	0.01	0.01	0.01	0.01	0.01
5/28/2020 2:15	0	0	0	0	0	0.01	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.04	0	0.03	0.03	0.01	0.01	0.01	0	0.01
5/28/2020 2:16	0	0.01	0	0	0	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0	0.02	0.01	0.01	0.01	0.01	0	0.01
5/28/2020 2:17	0	0.01	0	0	0	0.01	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0	0.02	0.01	0.01	0.01	0.01	0	0.01
5/28/2020 2:18	0	0.01	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0	0	0.01	0.01	0.01	0.01	0.01	0.01
5/28/2020 2:19	0	0.01	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0	0	0.01	0.01	0.01	0.01	0.01	0.01
5/28/2020 2:20	0	0.01	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0	0	0.01	0.01	0.01	0.01	0.01	0.01
5/28/2020 2:21	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.02	0	0	0.01	0	0	0.01	0.01	0.01
5/28/2020 2:22	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.02	0	0	0	0	0	0.01	0.01	0.01
5/28/2020 2:23	0	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.06	0	0	0	0	0	0.01	0.01	0.01

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 2:24	0	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.04	0	0	0	0	0	0.01	0.01	0.01
5/28/2020 2:25	0	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.02	0	0	0	0	0	0.01	0.01	0.01
5/28/2020 2:26	0	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0.01
5/28/2020 2:27	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0.01
5/28/2020 2:28	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0.01
5/28/2020 2:29	0	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.02	0.01	0.01	0.01	0	0.01	0.01	0	0	0	0	0	0.01	0	0.01
5/28/2020 2:30	0	0.01	0	0	0	0.01	0.01	0	0.01	0.01	0.02	0.01	0	0.01	0.01	0.01	0.01	0	0	0	0	0	0.01	0	0.01
5/28/2020 2:31	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.02	0.01	0	0.01	0.01	0.01	0.01	0.03	0	0	0	0.01	0.01	0	0.01
5/28/2020 2:32	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0.01	0	0.01	0.01	0.01	0.02	0.04	0	0	0.01	0.01	0.01	0	0.01
5/28/2020 2:33	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0	0	0.01	0.01	0.02	0.02	0.02	0	0.02	0.01	0.01	0.01	0	0.01
5/28/2020 2:34	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0	0	0.01	0.03	0.02	0.02	0.02	0	0.02	0.01	0.01	0.01	0	0.01
5/28/2020 2:35	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0	0	0.01	0.03	0.03	0.01	0.01	0	0.02	0.01	0.01	0.01	0	0.01
5/28/2020 2:36	0	0	0	0	0	0.01	0.01	0	0.01	0.01	0.01	0	0	0.01	0.01	0.03	0.01	0.01	0.04	0.03	0.01	0.01	0.03	0	0.01
5/28/2020 2:37	0	0.01	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0	0	0.01	0.01	0.01	0	0.01	0.04	0.02	0.01	0.03	0.01	0	0.02
5/28/2020 2:38	0	0.01	0	0	0	0.01	0.01	0	0.01	0.02	0.01	0	0	0.01	0.01	0.01	0	0.02	0.04	0.02	0.04	0.03	0.01	0	0.02
5/28/2020 2:39	0	0.01	0	0	0	0.01	0.01	0.02	0.03	0.02	0.01	0	0	0.01	0.01	0	0	0.02	0.02	0.03	0.04	0.02	0.01	0	0.02
5/28/2020 2:40	0	0.01	0	0	0	0.01	0.03	0.02	0.03	0.02	0.01	0	0	0.01	0.02	0	0	0.02	0.02	0.03	0.02	0.02	0.01	0	0.02
5/28/2020 2:41	0	0.01	0	0	0	0.01	0.03	0.02	0.02	0.02	0.01	0	0	0.01	0.02	0	0	0.02	0.02	0.03	0.02	0.01	0.01	0	0
5/28/2020 2:42	0	0.01	0	0	0.02	0.03	0.03	0.02	0.02	0.01	0.03	0	0	0.01	0.01	0	0	0.03	0.03	0.02	0.03	0.01	0.01	0	0
5/28/2020 2:43	0	0.01	0	0	0.02	0.03	0.01	0.01	0.02	0.01	0.03	0.02	0.02	0.01	0.01	0	0	0.02	0.03	0.02	0.03	0.01	0.01	0	0
5/28/2020 2:44	0	0.02	0	0	0.02	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0.02	0.01	0.01	0	0	0.02	0.02	0.04	0.03	0.01	0.01	0	0
5/28/2020 2:45	0	0.02	0	0	0.02	0.01	0.01	0.01	0.02	0.01	0.02	0.02	0.05	0.01	0.01	0	0	0.04	0.02	0.03	0.02	0.01	0.01	0.01	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 2:46	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.03	0.01	0.02	0	0	0.02	0.03	0.04	0.02	0.01	0.03	0.01	0
5/28/2020 2:47	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0.01	0.02	0	0	0.02	0.03	0.03	0.02	0.01	0.01	0.02	0
5/28/2020 2:48	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.01	0.02	0	0	0.04	0.03	0.03	0.02	0.01	0.01	0.03	0
5/28/2020 2:49	0	0.01	0	0	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.05	0.01	0.01	0	0	0	0.04	0.02	0.02	0.03	0.01	0.01	0.02	0
5/28/2020 2:50	0	0.01	0	0	0.01	0.01	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0	0	0	0.04	0.02	0.02	0.03	0.01	0.01	0.02	0
5/28/2020 2:51	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0	0	0.01	0.02	0.01	0.02	0.04	0.01	0.01	0.02	0
5/28/2020 2:52	0	0.01	0	0	0.01	0.01	0.02	0.01	0.01	0.02	0.01	0.01	0.01	0.01	0	0	0.01	0.02	0.01	0.02	0.03	0.01	0.01	0.02	0
5/28/2020 2:53	0	0.01	0	0	0.01	0.01	0.02	0.01	0.01	0.02	0.02	0.03	0.01	0.01	0	0	0.01	0.03	0.01	0.04	0.03	0.01	0	0.01	0.03
5/28/2020 2:54	0	0.01	0	0	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0	0	0.01	0.03	0.03	0.03	0.03	0.01	0	0.01	0.03
5/28/2020 2:55	0	0.01	0	0	0.01	0.01	0.01	0.01	0	0.02	0.03	0.02	0.01	0.01	0	0	0.01	0.03	0.03	0.02	0.03	0.01	0	0.01	0.02
5/28/2020 2:56	0	0.01	0	0	0.01	0.01	0.01	0.01	0	0.02	0.04	0.02	0.01	0.01	0	0	0	0	0.02	0.02	0.03	0.01	0	0.01	0.02
5/28/2020 2:57	0	0.01	0	0	0.01	0.01	0.01	0.01	0	0.02	0.04	0.02	0.01	0.01	0.01	0	0	0	0.02	0.03	0	0.01	0	0.01	0
5/28/2020 2:58	0	0.01	0	0	0.01	0.01	0.01	0.01	0	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0	0	0.02	0.03	0	0	0	0.02	0
5/28/2020 2:59	0	0.01	0	0	0.01	0.01	0.01	0.01	0	0.03	0.02	0.02	0.01	0.03	0.01	0.01	0	0	0.02	0.03	0	0	0	0.02	0
5/28/2020 3:00	0	0.02	0	0	0.01	0.01	0	0	0	0.01	0.01	0.02	0.01	0.03	0.01	0.01	0	0	0.01	0.01	0	0	0	0.02	0
5/28/2020 3:01	0	0.01	0	0	0.01	0.01	0	0	0	0.01	0.01	0.02	0.01	0.02	0	0.01	0	0	0.01	0.01	0	0	0	0.02	0
5/28/2020 3:02	0	0.01	0	0	0.01	0	0	0	0	0.01	0.01	0.02	0.01	0.02	0	0.01	0	0	0.01	0.01	0	0	0	0.02	0
5/28/2020 3:03	0	0	0	0	0.01	0	0	0	0	0.01	0	0.02	0.01	0	0	0	0	0	0.01	0.01	0	0	0	0.01	0
5/28/2020 3:04	0	0	0	0	0.01	0	0	0	0	0.01	0	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0
5/28/2020 3:05	0	0	0	0	0.01	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 3:06	0	0	0	0	0	0	0	0	0	0.01	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0
5/28/2020 3:07	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 3:08	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.02	0
5/28/2020 3:09	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 3:10	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 3:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0
5/28/2020 3:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
5/28/2020 3:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0
5/28/2020 3:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0
5/28/2020 3:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0.01	0	0	0	0	0	0	0	0
5/28/2020 3:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0	0	0	0
5/28/2020 3:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.02	0.01	0	0	0	0	0	0	0	0
5/28/2020 3:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.03	0.03	0	0	0	0	0	0	0	0.01
5/28/2020 3:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0.01
5/28/2020 3:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01
5/28/2020 3:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.02
5/28/2020 3:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.02
5/28/2020 3:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01
5/28/2020 3:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01
5/28/2020 3:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0	0	0	0	0	0	0	0	0.01
5/28/2020 3:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 3:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 3:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5/28/2020 3:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	IN14	PS2	BR 01	BR 02	BR 03	BR 05	BR 06	BR 07	BR 11	BR 12	IN11	IN12	IN16	ML 08	ML 09	ML 10	TU 02	TU 03	TU 04	TU 06	TU 07	TU 09	IN08	ML 04
5/28/2020 3:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M.2.5 JULY 2024 HISTORIC EVENT

Table M.2.5. July 2024 Historic Event Hyetograph Data.

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 8:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 8:02	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:03	0	0.04	0	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0
7/1/2024 8:04	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 8:05	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 8:06	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:08	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/1/2024 8:09	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/1/2024 8:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:11	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:12	0	0	0.04	0	0	0	0	0.04	0	0.04	0	0	0	0.04	0.03	0
7/1/2024 8:13	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:14	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 8:15	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:16	0	0.04	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 8:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 8:19	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/1/2024 8:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:21	0	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0
7/1/2024 8:22	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/1/2024 8:23	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:24	0	0	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 8:26	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 8:27	0	0.04	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0
7/1/2024 8:29	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0
7/1/2024 8:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/1/2024 8:31	0	0	0	0	0	0.04	0	0	0	0	0	0	0.03	0	0	0.01
7/1/2024 8:32	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/1/2024 8:33	0	0	0	0	0.04	0	0	0	0	0.04	0	0	0	0	0	0
7/1/2024 8:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:36	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/1/2024 8:37	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/1/2024 8:38	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:39	0	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:40	0	0	0	0	0	0	0.04	0	0	0	0.03	0	0	0	0	0
7/1/2024 8:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:42	0	0	0.04	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/1/2024 8:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.01
7/1/2024 8:45	0	0	0	0	0	0	0	0	0	0.04	0	0	0.04	0	0	0
7/1/2024 8:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:47	0	0.04	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0	0
7/1/2024 8:48	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/1/2024 8:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:50	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:52	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0
7/1/2024 8:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:54	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 8:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 8:56	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:57	0	0	0	0	0	0.04	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 8:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 8:59	0	0	0	0.04	0	0	0	0.04	0	0.04	0	0.03	0	0	0	0
7/1/2024 9:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:04	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/1/2024 9:05	0	0	0	0	0	0.04	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 9:06	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0.01
7/1/2024 9:07	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:08	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:09	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:10	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/1/2024 9:11	0	0	0.08	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:12	0	0	0	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0
7/1/2024 9:13	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0.08	0
7/1/2024 9:14	0	0	0	0.04	0	0	0	0	0.04	0	0	0	0	0.07	0	0.01
7/1/2024 9:15	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:16	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:17	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:18	0	0.04	0	0	0	0	0	0.04	0	0.04	0.04	0.04	0	0	0	0
7/1/2024 9:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:21	0	0	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0
7/1/2024 9:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:23	0	0	0.04	0	0.04	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 9:24	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 9:25	0	0	0	0.03	0	0	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 9:26	0	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0.04	0
7/1/2024 9:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:28	0.08	0	0	0	0	0	0	0	0	0.04	0	0	0.04	0	0	0
7/1/2024 9:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:30	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0	0
7/1/2024 9:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:33	0	0	0.04	0.04	0.04	0	0	0.04	0	0	0	0	0	0	0	0.01
7/1/2024 9:34	0.04	0	0	0	0	0.03	0.04	0	0	0	0	0	0	0	0	0
7/1/2024 9:35	0	0	0	0	0.04	0	0	0	0	0.04	0	0.04	0	0	0	0
7/1/2024 9:36	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:37	0	0.04	0	0	0.04	0.04	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 9:38	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0
7/1/2024 9:39	0	0.04	0	0	0.04	0	0	0	0.03	0	0	0	0	0	0.04	0
7/1/2024 9:40	0	0	0	0	0	0.04	0	0.04	0	0.04	0	0	0	0	0	0
7/1/2024 9:41	0	0.04	0	0	0	0.08	0	0	0	0	0	0	0	0.04	0	0.01
7/1/2024 9:42	0.04	0	0	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:43	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0	0.04	0	0	0
7/1/2024 9:44	0	0.04	0	0.04	0	0.04	0	0	0	0	0	0.04	0	0	0	0
7/1/2024 9:45	0	0.08	0	0	0	0.04	0.04	0	0	0	0	0	0	0	0	0
7/1/2024 9:46	0	0.04	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:47	0	0.04	0	0	0.04	0	0	0	0.04	0.04	0.04	0	0	0	0	0
7/1/2024 9:48	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0.01
7/1/2024 9:49	0	0.04	0.12	0.04	0	0	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 9:50	0	0	0	0.04	0	0.04	0	0.04	0	0	0.04	0	0	0	0.04	0
7/1/2024 9:51	0	0	0.04	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/1/2024 9:52	0.04	0.04	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 9:53	0	0	0	0	0	0	0	0.04	0.04	0	0	0.04	0	0	0	0
7/1/2024 9:54	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0.01
7/1/2024 9:55	0	0	0.04	0	0.04	0.04	0	0	0.08	0	0	0	0	0	0	0
7/1/2024 9:56	0	0	0	0	0	0	0	0.04	0	0.04	0	0	0.04	0	0.04	0
7/1/2024 9:57	0	0	0	0.04	0	0	0.04	0	0.04	0	0	0	0	0.08	0	0
7/1/2024 9:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 9:59	0	0	0	0	0.03	0	0	0.04	0	0	0	0	0	0	0	0
7/1/2024 10:00	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0.01
7/1/2024 10:01	0	0	0	0	0	0	0.03	0.08	0	0.04	0	0	0	0	0	0
7/1/2024 10:02	0	0.04	0	0	0.04	0	0	0.04	0	0	0	0.04	0	0	0	0
7/1/2024 10:03	0	0	0.04	0	0	0.04	0	0.03	0.12	0	0.04	0	0	0	0	0
7/1/2024 10:04	0	0	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0
7/1/2024 10:05	0.04	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0.04	0
7/1/2024 10:06	0	0	0	0	0	0	0.04	0	0.04	0.04	0.04	0.04	0.04	0	0	0.01
7/1/2024 10:07	0	0	0	0	0	0	0	0.04	0	0	0	0	0.04	0	0	0
7/1/2024 10:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0.08	0	0
7/1/2024 10:09	0	0.03	0	0	0	0	0	0	0.04	0	0	0.04	0	0	0	0
7/1/2024 10:10	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0
7/1/2024 10:11	0	0	0	0	0.04	0.04	0	0.04	0	0	0	0	0	0	0	0
7/1/2024 10:12	0	0	0	0.04	0	0	0	0	0	0.03	0	0	0	0	0	0.01
7/1/2024 10:13	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0
7/1/2024 10:14	0.03	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0
7/1/2024 10:15	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 10:16	0	0.04	0	0	0.04	0.04	0	0	0	0	0	0	0	0.04	0	0
7/1/2024 10:17	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/1/2024 10:18	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0.01
7/1/2024 10:19	0.04	0	0	0.04	0	0	0.04	0.04	0	0	0	0.04	0	0	0	0
7/1/2024 10:20	0	0.04	0	0	0.04	0	0	0	0	0	0.04	0	0	0	0	0
7/1/2024 10:21	0	0	0.08	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 10:22	0	0	0	0	0	0.04	0	0	0.12	0.04	0	0	0	0	0	0
7/1/2024 10:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 10:24	0.04	0	0	0	0	0	0.04	0	0.03	0	0	0	0	0	0.04	0.01
7/1/2024 10:25	0	0	0	0	0	0	0	0.04	0.04	0	0	0.04	0	0	0	0
7/1/2024 10:26	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0
7/1/2024 10:27	0	0	0	0	0	0	0	0	0.04	0	0.03	0	0	0	0	0
7/1/2024 10:28	0	0.04	0	0.04	0	0	0	0.04	0	0	0	0	0	0.04	0	0
7/1/2024 10:29	0	0	0.04	0	0	0	0.04	0	0	0	0.04	0.04	0.04	0	0	0
7/1/2024 10:30	0	0	0	0	0	0	0	0.04	0	0.04	0.04	0.04	0	0	0	0.01
7/1/2024 10:31	0.04	0	0	0	0	0	0	0.04	0	0	0	0.04	0.04	0	0	0
7/1/2024 10:32	0	0	0	0	0.04	0.04	0.04	0.04	0	0	0	0	0	0	0	0
7/1/2024 10:33	0.04	0	0	0	0	0	0	0.04	0	0	0.04	0.04	0.04	0	0.04	0
7/1/2024 10:34	0.04	0	0	0	0	0.04	0.04	0.04	0	0.04	0	0	0	0	0	0
7/1/2024 10:35	0.04	0.04	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.01
7/1/2024 10:36	0	0	0	0	0	0	0.04	0.04	0	0.04	0	0.03	0	0	0	0
7/1/2024 10:37	0.04	0	0	0	0	0	0.04	0.08	0.12	0.04	0.04	0	0	0	0	0
7/1/2024 10:38	0.04	0.04	0	0	0.04	0	0.08	0.11	0.04	0.04	0	0	0.03	0	0	0
7/1/2024 10:39	0.04	0	0	0.04	0	0.03	0.04	0.08	0.08	0.04	0.04	0.04	0	0.08	0	0
7/1/2024 10:40	0	0	0.04	0	0.04	0	0.03	0.08	0.08	0	0.04	0	0	0	0	0
7/1/2024 10:41	0.04	0	0	0	0.04	0	0.04	0.04	0.07	0.04	0.04	0.04	0.04	0	0	0.01
7/1/2024 10:42	0.04	0.04	0	0	0.04	0	0.04	0.08	0.08	0.04	0	0.04	0.04	0	0	0
7/1/2024 10:43	0.04	0	0.04	0	0	0	0.08	0.04	0.08	0.04	0.04	0	0	0.04	0.04	0
7/1/2024 10:44	0	0	0	0.08	0.08	0.04	0.04	0.08	0.08	0.04	0	0	0.04	0	0	0
7/1/2024 10:45	0.08	0	0.03	0	0.04	0.08	0.08	0.08	0.04	0.04	0.04	0.04	0	0	0	0.01
7/1/2024 10:46	0	0.04	0	0.04	0.04	0.08	0.04	0.04	0.08	0.04	0.04	0.04	0.04	0	0	0
7/1/2024 10:47	0.04	0	0	0	0.08	0.04	0.08	0.08	0.04	0.04	0.04	0.04	0	0.08	0	0
7/1/2024 10:48	0	0.04	0	0	0.04	0.08	0.08	0.04	0.08	0.03	0.04	0	0.04	0	0.04	0
7/1/2024 10:49	0	0.08	0.08	0.03	0.07	0.04	0.04	0.03	0.04	0.08	0.04	0.04	0.04	0	0	0
7/1/2024 10:50	0.04	0.04	0	0	0.04	0.08	0.04	0.04	0.04	0.04	0.04	0.04	0	0	0.04	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 10:51	0	0.04	0.04	0	0.04	0.04	0.04	0.08	0.04	0.08	0.04	0.08	0	0	0	0
7/1/2024 10:52	0	0.04	0.04	0.04	0.04	0.08	0.04	0.08	0.07	0	0.04	0.04	0.04	0	0	0
7/1/2024 10:53	0	0.04	0	0.04	0.04	0.04	0.03	0.04	0.04	0.12	0.03	0	0.04	0	0.03	0.01
7/1/2024 10:54	0.03	0.04	0.04	0.04	0.08	0.07	0.04	0.08	0.08	0.04	0.08	0.04	0.04	0	0	0
7/1/2024 10:55	0	0.04	0.04	0.04	0.04	0.04	0.04	0.08	0.04	0.04	0.04	0.04	0.04	0	0.04	0
7/1/2024 10:56	0	0.07	0.04	0.04	0.08	0.04	0.04	0.04	0.08	0.04	0.04	0.04	0.04	0	0	0.01
7/1/2024 10:57	0	0.04	0	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0	0.04	0.04	0	0.04	0
7/1/2024 10:58	0	0.04	0.04	0.08	0.04	0.04	0.08	0.08	0.08	0.04	0.04	0.03	0.04	0	0	0
7/1/2024 10:59	0	0.04	0	0	0.04	0.04	0.04	0.11	0.04	0.04	0.04	0.04	0	0	0.04	0.01
7/1/2024 11:00	0	0.04	0	0.04	0.04	0	0.04	0.08	0.04	0.04	0.04	0.04	0.04	0	0.04	0
7/1/2024 11:01	0	0.04	0.08	0.04	0.04	0.04	0.04	0.12	0.12	0.03	0.04	0.04	0.04	0	0	0
7/1/2024 11:02	0	0	0	0	0.07	0.04	0.04	0.08	0.04	0.04	0.04	0.04	0	0	0	0
7/1/2024 11:03	0.08	0.04	0	0.04	0.08	0.04	0.04	0.04	0	0.04	0	0	0.04	0	0.04	0.01
7/1/2024 11:04	0	0	0.08	0.04	0.08	0.04	0.08	0.08	0.15	0.04	0.04	0.04	0.03	0	0	0
7/1/2024 11:05	0	0.04	0	0.04	0.12	0.08	0.08	0.08	0.08	0.04	0	0.04	0	0	0.04	0
7/1/2024 11:06	0.04	0	0.04	0	0.08	0.04	0.07	0.04	0.08	0	0.04	0.04	0.04	0	0	0.01
7/1/2024 11:07	0	0.08	0	0.04	0.04	0.08	0.04	0.08	0.04	0.08	0	0	0	0	0.04	0
7/1/2024 11:08	0	0.08	0.04	0	0.04	0.07	0.04	0.07	0.08	0.04	0	0.04	0.04	0	0	0
7/1/2024 11:09	0	0.04	0	0	0.08	0.04	0.04	0.08	0	0.04	0.04	0	0	0	0	0.01
7/1/2024 11:10	0	0.08	0	0.04	0.08	0.04	0.04	0.08	0	0.04	0.04	0.04	0.04	0	0	0
7/1/2024 11:11	0	0.03	0	0	0.07	0.04	0.08	0.04	0	0.08	0.04	0	0	0	0	0.01
7/1/2024 11:12	0	0.04	0.04	0.03	0.08	0.04	0.04	0.04	0.08	0.04	0.04	0	0	0	0	0
7/1/2024 11:13	0	0.04	0	0	0.08	0.08	0	0	0	0.04	0.04	0.04	0.04	0	0	0
7/1/2024 11:14	0	0.04	0	0	0.08	0.08	0.04	0.04	0	0.08	0.03	0	0	0	0	0.01
7/1/2024 11:15	0	0.04	0	0.04	0.04	0.08	0	0	0	0.04	0	0.04	0.04	0	0	0
7/1/2024 11:16	0	0.04	0.07	0	0.08	0.08	0	0	0	0.03	0.04	0	0	0	0	0.01
7/1/2024 11:17	0	0.04	0	0.04	0.08	0.08	0	0	0	0	0	0.04	0.04	0	0.08	0
7/1/2024 11:18	0	0.08	0	0	0.08	0.11	0.04	0	0	0.04	0.04	0.04	0	0	0	0.01
7/1/2024 11:19	0.04	0.04	0.04	0	0.04	0.08	0	0	0	0	0	0	0.04	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 11:20	0	0.04	0	0.04	0.03	0.08	0	0	0	0	0	0.04	0	0	0.04	0.01
7/1/2024 11:21	0	0.08	0.04	0	0	0.08	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 11:22	0	0.04	0	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:23	0	0.04	0	0.04	0	0.04	0	0	0	0	0.04	0	0.04	0	0.04	0
7/1/2024 11:24	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:25	0	0.04	0	0.04	0	0	0	0	0	0	0	0.04	0	0	0	0
7/1/2024 11:26	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0.01
7/1/2024 11:27	0	0	0.04	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0
7/1/2024 11:28	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/1/2024 11:30	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 11:32	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 11:34	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:35	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0
7/1/2024 11:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:38	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/1/2024 11:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:40	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.01
7/1/2024 11:41	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/1/2024 11:42	0	0	0	0	0	0.04	0	0.08	0.12	0	0	0	0	0	0	0.01
7/1/2024 11:43	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.01
7/1/2024 11:44	0.04	0	0	0	0	0	0	0.08	0.04	0	0	0	0	0	0	0.01
7/1/2024 11:45	0	0	0	0	0.04	0	0	0.03	0	0	0.04	0	0	0	0	0
7/1/2024 11:46	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.01
7/1/2024 11:47	0	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0.01
7/1/2024 11:48	0	0	0	0	0.04	0	0.04	0.04	0.11	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 11:49	0	0	0	0	0.04	0.04	0	0.04	0.04	0	0	0	0	0	0	0.01
7/1/2024 11:50	0.04	0	0	0	0.08	0.04	0.04	0.04	0	0.04	0	0.04	0	0	0	0.01
7/1/2024 11:51	0.08	0	0	0	0.04	0.04	0	0	0.04	0	0	0	0	0	0	0
7/1/2024 11:52	0.04	0	0	0	0.04	0.04	0.04	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:53	0.04	0.04	0	0	0.04	0.04	0.04	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:54	0.04	0.04	0	0	0.04	0.03	0.04	0	0	0	0	0	0	0	0	0.01
7/1/2024 11:55	0	0.04	0	0	0	0.04	0.03	0	0	0.04	0.04	0	0	0	0	0.01
7/1/2024 11:56	0.08	0	0	0	0	0.04	0.04	0.04	0	0.08	0.04	0	0	0	0	0
7/1/2024 11:57	0.04	0.04	0	0	0	0.04	0.04	0.04	0	0.04	0.04	0	0	0	0	0.01
7/1/2024 11:58	0.04	0.08	0	0	0	0	0.04	0.08	0	0.04	0.04	0.03	0.04	0	0	0.01
7/1/2024 11:59	0	0.04	0	0	0	0	0.04	0.08	0	0.04	0.04	0.04	0	0	0	0
7/1/2024 12:00	0.03	0.04	0.08	0	0	0.04	0.04	0.08	0	0.04	0.04	0.08	0.04	0	0	0.01
7/1/2024 12:01	0	0.04	0	0.04	0	0	0.08	0.04	0	0.04	0.04	0.04	0.04	0	0.04	0.01
7/1/2024 12:02	0	0	0.08	0.04	0	0	0.04	0.07	0.08	0.04	0.04	0.04	0.04	0	0	0.01
7/1/2024 12:03	0	0.04	0.04	0.04	0.04	0	0.08	0.08	0.04	0.04	0.04	0	0.04	0	0.04	0.01
7/1/2024 12:04	0	0.04	0.04	0.04	0.04	0	0.08	0.08	0.08	0.08	0.04	0.08	0	0.11	0.04	0.01
7/1/2024 12:05	0.04	0.04	0.04	0.04	0.15	0	0.12	0.08	0.04	0.04	0.08	0.04	0.03	0	0.03	0.01
7/1/2024 12:06	0	0.04	0.04	0.04	0.12	0	0.07	0.04	0.04	0.07	0.03	0.08	0.04	0	0	0
7/1/2024 12:07	0	0.08	0	0.04	0.08	0.08	0.04	0.04	0	0.04	0.04	0.04	0.08	0	0.08	0.01
7/1/2024 12:08	0	0.07	0.04	0	0.04	0.12	0	0	0	0.08	0.04	0.04	0.08	0	0.04	0.01
7/1/2024 12:09	0	0.08	0.04	0	0.08	0.08	0.04	0	0	0.04	0.04	0.04	0.08	0	0.04	0.01
7/1/2024 12:10	0	0.12	0	0.08	0.08	0.08	0	0.04	0	0.04	0.04	0	0.04	0	0.04	0.01
7/1/2024 12:11	0	0.04	0	0.03	0.04	0.08	0	0	0	0	0	0.04	0	0	0	0.01
7/1/2024 12:12	0	0.08	0	0.04	0.04	0.07	0	0	0	0	0	0	0	0.08	0.04	0
7/1/2024 12:13	0	0.08	0	0.08	0.04	0.04	0	0	0	0	0	0	0	0	0.04	0.01
7/1/2024 12:14	0	0.04	0.08	0.04	0	0.04	0	0	0	0	0	0	0.04	0	0.04	0.01
7/1/2024 12:15	0	0.04	0	0.04	0	0.04	0	0	0	0.04	0	0	0	0	0.04	0.01
7/1/2024 12:16	0	0.04	0	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:17	0	0.04	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 12:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.01
7/1/2024 12:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:24	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:36	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:38	0	0	0	0.04	0	0	0	0	0.04	0	0	0	0	0.04	0	0.01
7/1/2024 12:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 12:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 12:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 12:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 13:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 13:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 13:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 13:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 13:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 13:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 13:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 13:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03
7/1/2024 14:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 14:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03
7/1/2024 14:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 14:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 14:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 14:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04
7/1/2024 14:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 14:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 14:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 15:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/1/2024 15:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/1/2024 15:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/1/2024 15:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 15:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/1/2024 16:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M.2.6 JULY 2025 HISTORIC EVENT

Table M.2.6. July 2025 Historic Event Hyetograph Data.

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 19:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:06	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:15	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:24	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 19:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:33	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:42	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:54	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 19:55	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:56	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:57	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:58	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 19:59	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:02	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:03	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:04	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:05	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:06	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:08	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:09	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:11	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:13	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:15	0.03	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:16	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:17	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0
7/16/2025 20:18	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/16/2025 20:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:20	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/16/2025 20:21	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:22	0.04	0	0	0	0	0	0	0	0	0	0	0	0.04	0.08	0.04	0
7/16/2025 20:23	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 20:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0
7/16/2025 20:25	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:26	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/16/2025 20:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 20:28	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0.04	0.04	0.04	0
7/16/2025 20:29	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0
7/16/2025 20:30	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0.04	0	0
7/16/2025 20:31	0	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0.04	0	0.08	0
7/16/2025 20:32	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0.04	0.04	0
7/16/2025 20:33	0	0	0	0	0.04	0	0	0	0	0.04	0.04	0.04	0.04	0.04	0.04	0
7/16/2025 20:34	0	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0.04	0
7/16/2025 20:35	0	0	0	0.04	0.04	0	0	0	0	0	0	0.04	0.04	0	0.04	0
7/16/2025 20:36	0	0	0	0.04	0	0	0	0	0	0	0.04	0.04	0	0.12	0.04	0
7/16/2025 20:37	0	0	0.04	0	0.04	0	0	0	0	0	0	0.04	0.04	0.03	0.04	0
7/16/2025 20:38	0	0.04	0	0.03	0	0	0	0	0	0	0	0.04	0.03	0.04	0.04	0
7/16/2025 20:39	0	0	0.04	0	0.04	0	0	0	0	0	0	0.04	0.04	0	0.04	0
7/16/2025 20:40	0	0	0.04	0.04	0	0	0	0	0	0	0	0.04	0.04	0.04	0.03	0
7/16/2025 20:41	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0.04	0.04	0
7/16/2025 20:42	0	0	0.04	0.04	0.04	0	0	0	0	0	0	0.04	0.08	0.04	0	0
7/16/2025 20:43	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0.04	0
7/16/2025 20:44	0.04	0	0.04	0.04	0	0	0	0	0	0	0.04	0	0.04	0	0	0
7/16/2025 20:45	0	0	0.04	0	0	0	0	0	0	0	0	0.08	0	0	0	0
7/16/2025 20:46	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0.08	0.08	0
7/16/2025 20:47	0	0	0	0	0.04	0	0	0	0	0	0.04	0.03	0	0.04	0	0
7/16/2025 20:48	0.04	0	0.08	0.04	0	0	0	0	0	0.04	0	0	0.04	0	0	0
7/16/2025 20:49	0.04	0	0.04	0.04	0	0	0.04	0	0.04	0	0.04	0.04	0	0.04	0	0
7/16/2025 20:50	0	0	0	0.04	0	0	0.03	0	0	0.04	0.04	0	0	0	0	0
7/16/2025 20:51	0.03	0	0	0.04	0	0	0.04	0	0	0.04	0.04	0.04	0.04	0.04	0	0
7/16/2025 20:52	0	0	0.08	0.04	0	0	0.04	0	0.04	0	0	0.04	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 20:53	0	0	0	0	0	0	0.04	0	0.04	0.03	0.04	0.04	0.04	0	0.04	0
7/16/2025 20:54	0	0	0.03	0.04	0	0	0.04	0	0.04	0.04	0.04	0.08	0	0	0	0
7/16/2025 20:55	0	0.04	0	0.04	0.04	0	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0	0	0
7/16/2025 20:56	0	0.04	0	0.04	0	0	0.04	0.04	0	0.04	0.04	0	0.04	0	0	0
7/16/2025 20:57	0.04	0.04	0.07	0.04	0	0	0.04	0	0.04	0.04	0.04	0.04	0.04	0	0	0
7/16/2025 20:58	0	0	0.04	0.04	0	0	0	0.04	0	0.04	0.03	0	0.04	0	0	0
7/16/2025 20:59	0	0.04	0.04	0.04	0.03	0	0.04	0	0.04	0.04	0.04	0.04	0	0.2	0	0
7/16/2025 21:00	0	0	0.04	0.04	0	0	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.03	0.04	0
7/16/2025 21:01	0	0.04	0.04	0.03	0	0.04	0.04	0.04	0	0	0.04	0	0.03	0	0	0
7/16/2025 21:02	0	0	0	0.04	0.04	0	0	0	0.04	0.08	0.04	0.04	0	0	0	0
7/16/2025 21:03	0	0.03	0.08	0.04	0	0	0.04	0.04	0.03	0.04	0	0	0.04	0	0	0
7/16/2025 21:04	0	0	0	0.04	0	0.04	0	0.04	0	0.04	0.04	0	0	0	0.04	0
7/16/2025 21:05	0	0.04	0.04	0	0	0	0	0.04	0.04	0	0.04	0	0	0	0.04	0
7/16/2025 21:06	0	0.04	0.04	0.04	0	0	0.04	0.04	0	0.04	0	0	0	0	0	0
7/16/2025 21:07	0	0.04	0	0	0	0.04	0	0	0	0	0	0.04	0.04	0	0.04	0.01
7/16/2025 21:08	0	0	0.04	0.04	0	0	0.04	0.04	0.04	0.04	0.04	0	0	0	0	0
7/16/2025 21:09	0	0.04	0.04	0.04	0	0.04	0.04	0.04	0.04	0	0	0	0	0	0	0
7/16/2025 21:10	0	0.04	0	0.04	0	0	0.04	0	0.04	0	0	0	0	0	0.04	0.01
7/16/2025 21:11	0	0	0	0	0.04	0.04	0	0.04	0.04	0.04	0.04	0	0	0	0	0.01
7/16/2025 21:12	0	0	0.08	0	0	0	0.04	0.04	0.08	0	0	0	0	0	0	0
7/16/2025 21:13	0	0	0	0	0	0.04	0.03	0.04	0.04	0.04	0	0	0	0	0	0
7/16/2025 21:14	0	0.04	0	0.04	0	0	0	0.03	0.04	0	0	0.04	0	0	0	0
7/16/2025 21:15	0	0	0	0	0	0.03	0.04	0.04	0.04	0.03	0.04	0	0	0	0	0
7/16/2025 21:16	0	0	0.04	0	0.04	0	0.04	0	0.04	0	0	0	0.04	0	0	0
7/16/2025 21:17	0	0	0	0	0	0.04	0	0.04	0.04	0	0	0	0	0	0	0
7/16/2025 21:18	0	0.04	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0.01
7/16/2025 21:19	0	0	0	0	0	0.04	0.04	0	0.04	0	0	0	0	0	0	0
7/16/2025 21:20	0	0	0	0	0.04	0	0	0.04	0.04	0	0	0	0	0	0	0
7/16/2025 21:21	0	0	0	0	0	0.04	0.08	0.04	0.04	0.04	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 21:22	0	0.04	0	0	0	0	0.04	0.04	0.03	0	0	0.04	0	0.16	0	0
7/16/2025 21:23	0	0	0	0	0	0.04	0.04	0.04	0.04	0.04	0	0	0	0	0	0
7/16/2025 21:24	0	0	0	0	0	0	0.04	0.04	0.04	0	0.04	0	0	0	0	0.01
7/16/2025 21:25	0	0	0	0	0	0.04	0	0.04	0.04	0.04	0	0	0	0	0	0.01
7/16/2025 21:26	0	0	0	0	0	0	0.04	0.08	0.04	0	0	0	0	0	0	0.01
7/16/2025 21:27	0	0.04	0	0	0	0	0.04	0.04	0.04	0.04	0.04	0	0.04	0	0	0.01
7/16/2025 21:28	0	0	0.04	0	0	0.04	0.04	0.04	0.04	0	0	0	0	0	0	0.02
7/16/2025 21:29	0	0	0	0	0	0	0	0.04	0	0.04	0.04	0	0	0	0	0.01
7/16/2025 21:30	0	0	0	0	0	0	0.04	0.04	0.04	0	0	0.04	0	0	0	0.01
7/16/2025 21:31	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0.02
7/16/2025 21:32	0	0	0	0.04	0.04	0.04	0.04	0.04	0	0	0	0.04	0	0	0.04	0.01
7/16/2025 21:33	0.04	0	0	0	0	0	0	0	0.04	0	0	0	0.04	0.04	0	0.01
7/16/2025 21:34	0	0.04	0	0	0	0	0.04	0.03	0	0.04	0.04	0.03	0	0	0	0.01
7/16/2025 21:35	0	0	0.03	0	0	0.04	0	0.04	0	0	0	0	0.04	0	0	0.02
7/16/2025 21:36	0	0	0	0	0	0	0.04	0	0	0	0	0.04	0	0.04	0	0.02
7/16/2025 21:37	0.04	0.04	0	0	0	0.04	0	0.04	0	0.04	0	0	0.04	0	0	0.03
7/16/2025 21:38	0	0	0.04	0.04	0.04	0	0.04	0	0	0	0.04	0	0	0.04	0	0.03
7/16/2025 21:39	0	0.04	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0.02
7/16/2025 21:40	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0.04	0	0	0.03
7/16/2025 21:41	0	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0.03
7/16/2025 21:42	0	0	0.04	0	0.04	0	0	0	0	0	0	0.04	0	0	0	0.01
7/16/2025 21:43	0	0	0	0.04	0	0.04	0	0	0	0.04	0	0	0.04	0	0	0.02
7/16/2025 21:44	0	0.04	0	0	0.08	0	0	0	0	0	0	0	0	0	0	0.02
7/16/2025 21:45	0	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0.08	0	0.01
7/16/2025 21:46	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0	0.08	0.01
7/16/2025 21:47	0	0	0	0.04	0.08	0.04	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 21:48	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0.02
7/16/2025 21:49	0	0	0	0	0.07	0	0	0.04	0.04	0	0	0.04	0	0	0	0.01
7/16/2025 21:50	0.04	0	0	0	0.04	0	0	0	0.04	0	0	0	0.04	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 21:51	0	0	0	0	0.04	0.04	0.03	0	0	0	0	0	0	0	0	0.01
7/16/2025 21:52	0	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0	0	0.01
7/16/2025 21:53	0	0	0	0	0.04	0	0.04	0.04	0	0	0.03	0.04	0	0.12	0	0.01
7/16/2025 21:54	0	0	0.04	0	0.08	0.04	0	0	0.12	0	0	0.04	0.04	0	0.04	0.01
7/16/2025 21:55	0	0	0	0	0.04	0	0.08	0	0.04	0	0.04	0.04	0	0	0.04	0
7/16/2025 21:56	0	0	0	0	0.08	0.04	0.04	0.04	0.04	0	0.04	0.04	0.04	0	0	0
7/16/2025 21:57	0	0	0	0	0.04	0	0.04	0.04	0.07	0	0.04	0	0.04	0.08	0	0
7/16/2025 21:58	0	0.04	0	0	0.04	0.04	0.04	0.08	0.04	0	0	0.04	0	0	0	0.03
7/16/2025 21:59	0	0	0	0	0.04	0	0.04	0.04	0.08	0	0.12	0.04	0.04	0	0	0.01
7/16/2025 22:00	0	0	0	0	0.04	0	0.04	0.08	0.04	0.08	0.08	0	0	0	0	0.02
7/16/2025 22:01	0	0	0	0	0.04	0.03	0.04	0.04	0.04	0.08	0.04	0.04	0	0.04	0	0
7/16/2025 22:02	0	0	0	0	0.07	0	0.08	0.04	0.04	0.04	0	0	0.03	0	0	0.01
7/16/2025 22:03	0	0	0	0	0.08	0.04	0.04	0.04	0.04	0.04	0.04	0	0	0	0	0.01
7/16/2025 22:04	0	0	0	0	0.08	0	0.04	0.07	0.04	0	0	0	0	0	0	0.01
7/16/2025 22:05	0	0	0	0.04	0.12	0.04	0.04	0.04	0.08	0.04	0.04	0	0	0	0	0.01
7/16/2025 22:06	0	0	0	0	0.08	0	0.04	0.04	0	0	0.04	0.04	0	0	0	0.01
7/16/2025 22:07	0	0.04	0.04	0	0.04	0.04	0.04	0	0.04	0	0.04	0	0	0	0	0.03
7/16/2025 22:08	0	0	0	0	0.04	0	0	0.04	0.04	0.03	0	0	0.04	0.04	0.04	0.02
7/16/2025 22:09	0	0	0	0	0.08	0	0.03	0.04	0	0	0.04	0	0	0	0	0.02
7/16/2025 22:10	0	0	0	0.04	0.08	0.04	0.04	0.04	0.04	0.04	0	0	0	0	0	0.02
7/16/2025 22:11	0	0	0	0	0.07	0	0.08	0	0	0	0.04	0	0	0	0	0.02
7/16/2025 22:12	0	0.03	0	0	0.08	0.04	0.04	0	0.04	0.04	0	0	0	0	0	0.01
7/16/2025 22:13	0	0	0	0.04	0.08	0	0.04	0	0	0.04	0.03	0.04	0	0	0	0.02
7/16/2025 22:14	0	0	0	0	0.08	0.04	0.04	0	0	0	0	0	0	0	0	0.03
7/16/2025 22:15	0	0	0	0	0.04	0	0.04	0	0	0	0.04	0	0	0	0	0.01
7/16/2025 22:16	0	0	0	0	0.08	0.04	0	0	0	0	0	0.04	0.04	0.03	0	0.02
7/16/2025 22:17	0	0	0	0	0.08	0	0	0.08	0	0.04	0	0	0	0	0	0.02
7/16/2025 22:18	0	0.04	0	0	0.08	0.04	0	0	0	0	0	0.04	0	0	0	0.01
7/16/2025 22:19	0	0	0	0	0.04	0	0	0	0	0.04	0.04	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 22:20	0	0.04	0	0	0.03	0	0	0	0.03	0	0	0.04	0.04	0	0	0.02
7/16/2025 22:21	0	0	0	0.04	0	0	0	0	0	0	0	0.03	0	0	0	0.01
7/16/2025 22:22	0	0	0	0	0.04	0.04	0	0	0	0	0.04	0	0	0	0	0.02
7/16/2025 22:23	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0.03	0.02
7/16/2025 22:24	0	0.04	0.12	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/16/2025 22:25	0	0	0	0	0	0.04	0.04	0	0	0	0.04	0	0.04	0	0	0.02
7/16/2025 22:26	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0.03
7/16/2025 22:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0.08	0	0.03
7/16/2025 22:28	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0.03
7/16/2025 22:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.02
7/16/2025 22:30	0	0	0	0.03	0.04	0	0	0	0	0	0	0	0	0	0	0.02
7/16/2025 22:31	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:32	0	0	0.04	0	0	0	0	0.04	0	0	0	0	0	0	0	0.02
7/16/2025 22:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:34	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0.01
7/16/2025 22:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:36	0.12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:37	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0.01
7/16/2025 22:38	0	0	0	0	0	0.08	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:39	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 22:42	0.08	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:43	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:44	0.08	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:45	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 22:46	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:47	0.04	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0	0	0
7/16/2025 22:48	0.07	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 22:49	0.04	0	0	0	0.04	0	0	0	0	0	0	0.04	0	0	0	0.01
7/16/2025 22:50	0.04	0.04	0	0	0.08	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 22:51	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 22:52	0	0	0	0	0.04	0	0	0	0	0	0	0.04	0	0	0	0.01
7/16/2025 22:53	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0	0	0
7/16/2025 22:54	0	0	0	0	0.04	0	0	0	0.04	0	0.04	0	0	0	0	0
7/16/2025 22:55	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.01
7/16/2025 22:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 22:57	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0	0	0	0
7/16/2025 22:58	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 22:59	0	0	0	0	0	0.04	0	0	0	0	0.04	0.04	0	0	0	0.01
7/16/2025 23:00	0.04	0	0.04	0	0	0	0	0	0	0	0	0.04	0	0	0.08	0
7/16/2025 23:01	0.04	0	0	0	0	0	0	0	0	0	0.04	0	0.04	0	0	0
7/16/2025 23:02	0.04	0	0	0	0	0	0	0	0	0	0	0.04	0	0.04	0	0
7/16/2025 23:03	0	0	0	0	0.04	0	0.04	0	0	0	0	0.04	0.04	0	0	0
7/16/2025 23:04	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0.04	0
7/16/2025 23:05	0.04	0	0	0	0.15	0	0	0	0	0.04	0	0	0.04	0.04	0	0.01
7/16/2025 23:06	0	0	0.04	0	0.12	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 23:07	0	0	0	0	0.08	0	0	0	0	0	0	0	0	0.04	0	0.02
7/16/2025 23:08	0	0	0	0	0.04	0	0	0	0	0	0.04	0	0.04	0	0	0
7/16/2025 23:09	0	0	0	0.04	0.08	0	0	0	0	0	0	0	0	0	0.04	0.02
7/16/2025 23:10	0.04	0	0	0	0.08	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 23:11	0	0	0	0	0.04	0	0	0	0	0	0	0.04	0	0.08	0	0.01
7/16/2025 23:12	0	0	0.04	0	0.04	0	0.04	0	0	0	0	0	0	0	0.04	0
7/16/2025 23:13	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:14	0	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0.04	0.04	0.01
7/16/2025 23:15	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/16/2025 23:16	0	0	0	0	0.04	0	0	0	0	0.04	0	0	0	0	0	0
7/16/2025 23:17	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0.04	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 23:18	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/16/2025 23:19	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0.01
7/16/2025 23:20	0	0	0	0	0	0	0	0.04	0.04	0.04	0	0	0.04	0	0	0
7/16/2025 23:21	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0.01
7/16/2025 23:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:23	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0.16	0	0.01
7/16/2025 23:24	0	0.04	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 23:26	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0.04	0
7/16/2025 23:27	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:28	0	0	0	0	0	0	0	0	0.04	0	0	0	0.07	0	0	0.01
7/16/2025 23:29	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:30	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/16/2025 23:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 23:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:33	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/16/2025 23:34	0	0	0	0.04	0	0	0	0.08	0.04	0	0	0	0	0	0	0.01
7/16/2025 23:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 23:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:38	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/16/2025 23:39	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:40	0	0	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0.01
7/16/2025 23:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/16/2025 23:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:45	0	0	0	0	0	0	0.03	0.04	0.04	0	0	0	0	0	0	0
7/16/2025 23:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/16/2025 23:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:48	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/16/2025 23:49	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/16/2025 23:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:51	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0
7/16/2025 23:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:54	0	0	0	0	0	0	0.04	0.04	0.04	0	0	0	0	0	0	0
7/16/2025 23:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/16/2025 23:56	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/16/2025 23:57	0	0	0	0	0	0	0.04	0	0.04	0.04	0	0	0	0	0	0
7/16/2025 23:58	0	0	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0
7/16/2025 23:59	0	0	0	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0
7/17/2025 0:00	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0.07	0	0
7/17/2025 0:01	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0
7/17/2025 0:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 0:03	0	0	0	0	0	0	0	0.03	0	0.04	0	0	0	0	0	0
7/17/2025 0:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:05	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:08	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 0:09	0	0	0	0	0	0	0	0.08	0	0.04	0	0	0	0	0	0
7/17/2025 0:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:13	0	0	0	0	0	0	0	0.04	0	0	0.04	0	0	0	0	0
7/17/2025 0:14	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 0:15	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 0:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:19	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 0:20	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 0:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:22	0	0	0	0	0	0.03	0.04	0	0.11	0	0	0	0	0	0	0
7/17/2025 0:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:24	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 0:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:26	0	0	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0
7/17/2025 0:27	0	0.04	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/17/2025 0:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:30	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:31	0	0	0	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0
7/17/2025 0:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:37	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/17/2025 0:38	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/17/2025 0:39	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/17/2025 0:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:43	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/17/2025 0:44	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 0:45	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:47	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0
7/17/2025 0:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:52	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0
7/17/2025 0:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0
7/17/2025 0:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 0:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 1:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:27	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:34	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 1:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:47	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 1:48	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 1:49	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0	0
7/17/2025 1:50	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 1:51	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 1:52	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 1:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:55	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 1:56	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0.01
7/17/2025 1:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 1:58	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/17/2025 1:59	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0.01
7/17/2025 2:00	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:01	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:02	0	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:03	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0.01
7/17/2025 2:04	0	0.04	0	0	0	0.04	0	0	0	0	0	0	0.04	0.04	0	0
7/17/2025 2:05	0	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:06	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 2:07	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:08	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:09	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:10	0	0	0	0.04	0	0.04	0	0	0	0.04	0	0	0	0	0.04	0
7/17/2025 2:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 2:12	0	0	0	0.04	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:13	0.04	0.04	0	0	0	0	0.04	0	0	0	0	0.04	0	0	0	0
7/17/2025 2:14	0	0	0	0	0	0	0	0	0.04	0	0.04	0	0	0	0.04	0.01
7/17/2025 2:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:16	0	0.03	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:18	0	0	0	0	0	0.04	0	0	0	0	0	0	0.04	0	0	0
7/17/2025 2:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:20	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.04	0	0
7/17/2025 2:21	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:23	0	0	0.04	0	0	0	0	0.04	0	0.04	0	0	0.04	0	0.04	0
7/17/2025 2:24	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 2:25	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 2:27	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0
7/17/2025 2:28	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0.04	0
7/17/2025 2:29	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0.04	0	0
7/17/2025 2:30	0.07	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0
7/17/2025 2:31	0.04	0	0	0	0	0	0.03	0	0.04	0	0	0	0	0	0	0.01
7/17/2025 2:32	0.04	0	0	0	0	0	0.04	0	0	0	0.04	0.04	0	0	0	0.01
7/17/2025 2:33	0	0	0	0	0	0	0.04	0.04	0.04	0.04	0	0	0	0	0	0.01
7/17/2025 2:34	0.08	0	0	0	0	0	0.04	0	0	0	0.03	0.04	0	0	0	0
7/17/2025 2:35	0.08	0	0	0	0	0	0.04	0	0	0.04	0	0.04	0	0	0	0.01
7/17/2025 2:36	0.04	0	0	0	0	0	0	0	0	0	0.08	0.04	0	0	0	0
7/17/2025 2:37	0.08	0	0	0	0	0	0.04	0	0.04	0	0.04	0.04	0.04	0	0	0.01
7/17/2025 2:38	0.04	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/17/2025 2:39	0.08	0	0	0	0	0.04	0	0	0	0	0.04	0.04	0.04	0	0	0
7/17/2025 2:40	0.08	0	0.04	0	0	0	0.04	0	0	0.04	0.04	0.04	0.04	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 2:41	0.07	0	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0	0	0
7/17/2025 2:42	0.12	0	0	0	0	0	0	0	0	0	0	0.08	0.04	0.08	0.04	0
7/17/2025 2:43	0.08	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0.04	0	0
7/17/2025 2:44	0.12	0	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0.04	0.03	0
7/17/2025 2:45	0.08	0.04	0	0	0	0	0	0	0	0	0.04	0.04	0.04	0	0	0
7/17/2025 2:46	0.12	0	0.04	0	0	0	0	0.04	0	0	0.04	0.03	0.04	0.08	0.04	0
7/17/2025 2:47	0.11	0	0	0	0	0	0	0.03	0.04	0	0	0.04	0.04	0.04	0	0
7/17/2025 2:48	0.12	0	0	0.04	0	0	0	0.12	0	0	0.04	0.04	0.03	0.04	0.04	0
7/17/2025 2:49	0.2	0	0	0	0	0	0.04	0.12	0.08	0	0.04	0.04	0.08	0.04	0.04	0.01
7/17/2025 2:50	0.24	0	0	0	0	0	0	0.04	0.07	0.04	0.04	0.04	0.04	0.08	0	0
7/17/2025 2:51	0.15	0	0	0	0	0	0.04	0.12	0.04	0	0.04	0.08	0.04	0	0.04	0
7/17/2025 2:52	0.12	0	0.04	0	0	0	0.04	0.08	0.08	0.04	0.04	0.04	0.04	0.07	0.04	0
7/17/2025 2:53	0.04	0	0	0.04	0	0	0.04	0.08	0.08	0	0.04	0.04	0.04	0.04	0	0
7/17/2025 2:54	0.08	0	0	0	0	0	0.04	0.08	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0
7/17/2025 2:55	0.12	0	0	0	0	0	0.08	0	0.16	0.04	0	0.08	0.04	0.04	0	0
7/17/2025 2:56	0.08	0	0	0	0	0.04	0.08	0.19	0.08	0.04	0.07	0.04	0.08	0.08	0.04	0
7/17/2025 2:57	0.08	0	0.04	0	0	0	0.07	0.12	0.08	0.08	0.04	0.08	0.04	0.04	0.04	0
7/17/2025 2:58	0.07	0	0	0.03	0	0	0.08	0.08	0.07	0.04	0.08	0	0.08	0.04	0.04	0
7/17/2025 2:59	0.08	0	0.04	0	0	0	0.08	0.04	0.08	0.08	0.08	0.15	0.04	0.08	0.04	0
7/17/2025 3:00	0.08	0	0	0	0	0.04	0.08	0.08	0	0.07	0.04	0.08	0.04	0.04	0.04	0
7/17/2025 3:01	0.08	0.04	0.12	0.04	0	0	0.08	0.08	0	0.08	0.08	0	0.07	0.04	0.04	0
7/17/2025 3:02	0.12	0	0.04	0.04	0	0.04	0	0.08	0	0.12	0.08	0.16	0.04	0	0.04	0
7/17/2025 3:03	0.12	0.04	0.04	0.04	0	0	0.16	0.03	0	0.12	0.08	0.08	0.04	0.08	0.04	0.01
7/17/2025 3:04	0.12	0.04	0.04	0	0	0.04	0.08	0.08	0.08	0.08	0.04	0.08	0.04	0.04	0	0
7/17/2025 3:05	0.07	0.04	0.04	0.04	0	0.04	0.11	0.04	0.08	0.08	0.08	0.08	0.04	0.04	0.08	0.01
7/17/2025 3:06	0.04	0.04	0.07	0.08	0	0.04	0.12	0.08	0	0.08	0.04	0.04	0.08	0	0.03	0.01
7/17/2025 3:07	0.04	0.08	0.04	0.04	0	0	0.08	0.04	0.12	0.07	0.03	0.07	0	0	0.04	0.02
7/17/2025 3:08	0.08	0.04	0.04	0.08	0	0.04	0.08	0.08	0.04	0.08	0.04	0.04	0.04	0.11	0.08	0.05
7/17/2025 3:09	0.04	0.08	0.04	0.08	0	0.04	0.08	0.08	0.08	0.12	0.08	0.08	0.04	0.04	0.04	0.06

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 3:10	0.04	0.08	0.08	0	0	0.03	0.04	0.04	0.04	0.08	0.08	0.08	0.04	0.04	0.04	0.04
7/17/2025 3:11	0	0.08	0.04	0.08	0	0.04	0.08	0.08	0.08	0.04	0.04	0.08	0.04	0.08	0.08	0.06
7/17/2025 3:12	0.12	0.07	0.04	0.04	0	0.04	0	0	0	0.08	0.08	0.08	0.08	0.04	0.04	0.04
7/17/2025 3:13	0.04	0.08	0.08	0.04	0	0.08	0.12	0.12	0.04	0.08	0.08	0.12	0.04	0.04	0.04	0.04
7/17/2025 3:14	0.08	0.08	0	0.07	0	0.04	0.03	0.11	0.07	0.08	0.04	0.08	0.04	0.08	0.04	0.04
7/17/2025 3:15	0.08	0.04	0.12	0.04	0	0.04	0.12	0.04	0.04	0.04	0.08	0.08	0.08	0	0.04	0.03
7/17/2025 3:16	0	0.08	0	0.08	0	0	0.04	0.04	0.04	0.04	0.04	0.07	0.07	0	0.04	0.02
7/17/2025 3:17	0.07	0.08	0.08	0	0	0.04	0.08	0.04	0	0.07	0.07	0.04	0.08	0.2	0.04	0
7/17/2025 3:18	0.04	0.04	0	0	0	0.04	0.04	0.08	0.12	0.04	0.04	0.04	0.08	0.07	0.04	0.08
7/17/2025 3:19	0.04	0.04	0.11	0.2	0	0.04	0.04	0.04	0	0.08	0.04	0.08	0	0.04	0.04	0.08
7/17/2025 3:20	0.04	0.04	0.08	0.08	0	0.04	0.04	0.04	0.08	0.04	0.08	0.04	0.16	0.08	0.03	0.08
7/17/2025 3:21	0	0.04	0.08	0.04	0	0	0.04	0.04	0.04	0.04	0.04	0.08	0.08	0.04	0.04	0.06
7/17/2025 3:22	0	0.04	0.08	0.08	0	0.04	0.04	0.04	0.04	0.08	0.04	0.04	0.04	0.04	0.08	0.06
7/17/2025 3:23	0.04	0.03	0.04	0.04	0	0.04	0.04	0	0.04	0	0.04	0.04	0.08	0.08	0	0.05
7/17/2025 3:24	0	0.08	0.12	0.04	0	0.04	0.08	0.04	0	0.08	0.08	0.08	0.04	0.08	0.12	0.04
7/17/2025 3:25	0	0	0.04	0.07	0	0.04	0.04	0.08	0.04	0.08	0.04	0.04	0.03	0.04	0.04	0.03
7/17/2025 3:26	0	0.08	0	0.08	0	0	0.03	0.04	0	0.04	0	0.04	0.04	0	0.08	0.03
7/17/2025 3:27	0	0.04	0.04	0.04	0	0.07	0.04	0.03	0	0.04	0.08	0.04	0.04	0	0.04	0.03
7/17/2025 3:28	0	0	0	0.08	0	0.04	0	0	0	0.04	0.04	0	0.04	0.16	0	0.04
7/17/2025 3:29	0	0.08	0.12	0.04	0	0.04	0.04	0.04	0	0.03	0	0.04	0.04	0.07	0.12	0.02
7/17/2025 3:30	0.04	0	0.03	0	0	0.04	0	0.04	0	0.04	0.04	0.03	0.04	0.04	0.08	0.03
7/17/2025 3:31	0	0.04	0	0.08	0	0.04	0.04	0	0	0.04	0.04	0.04	0.08	0.04	0.04	0.03
7/17/2025 3:32	0	0.04	0.04	0	0	0	0	0.04	0.15	0	0.04	0.04	0	0.08	0.07	0.03
7/17/2025 3:33	0	0.04	0	0.04	0	0.04	0.04	0	0.04	0.04	0	0.04	0.08	0.04	0.04	0.03
7/17/2025 3:34	0	0	0	0	0	0	0	0	0	0	0.03	0	0.04	0.04	0.08	0.02
7/17/2025 3:35	0	0.04	0	0.04	0	0	0	0	0	0.04	0	0	0.04	0.04	0.04	0.02
7/17/2025 3:36	0	0	0.08	0	0	0.08	0	0.04	0	0	0.04	0.04	0.04	0	0.04	0.02
7/17/2025 3:37	0	0.04	0	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0.04
7/17/2025 3:38	0	0	0.04	0.04	0	0.04	0	0.04	0.12	0	0	0	0	0.04	0	0.04

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 3:39	0	0.04	0	0	0	0	0	0	0	0.04	0	0.04	0	0	0.04	0.04
7/17/2025 3:40	0	0	0	0.04	0	0.04	0	0.04	0.04	0	0.04	0	0.04	0	0.04	0.03
7/17/2025 3:41	0	0	0.04	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0.04
7/17/2025 3:42	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0.02
7/17/2025 3:43	0	0.04	0	0.04	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0.03
7/17/2025 3:44	0	0	0	0	0	0	0	0.04	0	0.04	0	0	0	0	0	0.02
7/17/2025 3:45	0	0.04	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0	0.01
7/17/2025 3:46	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0.02
7/17/2025 3:47	0	0.04	0	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0.01
7/17/2025 3:48	0	0	0.04	0	0	0	0	0	0	0.04	0	0	0	0	0	0.01
7/17/2025 3:49	0.04	0.03	0	0.04	0	0.04	0	0	0	0	0	0	0.08	0	0	0.01
7/17/2025 3:50	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 3:51	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 3:52	0	0	0	0.04	0	0	0.04	0	0	0	0	0.04	0	0.04	0	0.02
7/17/2025 3:53	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/17/2025 3:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 3:55	0	0.04	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 3:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 3:57	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0.01
7/17/2025 3:58	0.04	0	0.04	0.03	0	0	0	0	0	0	0.08	0	0	0	0	0
7/17/2025 3:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0
7/17/2025 4:00	0	0	0	0	0	0	0	0	0.08	0	0	0	0	0	0	0
7/17/2025 4:01	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.01
7/17/2025 4:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:03	0.04	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0
7/17/2025 4:04	0.04	0	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0	0.02
7/17/2025 4:05	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0.01
7/17/2025 4:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 4:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:09	0.08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:10	0	0	0	0	0	0	0	0.08	0	0	0	0	0	0	0	0.02
7/17/2025 4:11	0	0	0	0.04	0	0	0	0.04	0.08	0	0	0	0	0.04	0	0.01
7/17/2025 4:12	0	0.04	0	0	0	0.03	0.08	0.04	0	0	0	0	0	0	0	0.02
7/17/2025 4:13	0	0	0	0	0	0	0	0.04	0.04	0.04	0	0	0	0	0	0.01
7/17/2025 4:14	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0	0.01
7/17/2025 4:15	0	0	0	0	0	0	0.04	0	0	0	0.08	0	0.04	0	0	0.01
7/17/2025 4:16	0.04	0	0	0	0	0.04	0	0.04	0	0.04	0	0.04	0	0	0	0.01
7/17/2025 4:17	0	0.04	0	0	0	0	0.04	0	0	0	0.04	0	0	0	0	0.01
7/17/2025 4:18	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0	0.01
7/17/2025 4:19	0.04	0	0	0	0	0	0.04	0	0	0.04	0.04	0	0	0	0	0.01
7/17/2025 4:20	0	0	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:21	0	0	0	0	0	0	0.04	0.03	0	0	0	0	0	0	0.04	0.01
7/17/2025 4:22	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0	0	0	0.01
7/17/2025 4:23	0.04	0.04	0	0	0	0	0	0	0.12	0	0.04	0.04	0	0.04	0	0.01
7/17/2025 4:24	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0
7/17/2025 4:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.01
7/17/2025 4:26	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:27	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/17/2025 4:28	0.04	0	0	0	0	0	0	0.04	0	0	0.04	0	0.08	0	0	0.01
7/17/2025 4:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.01
7/17/2025 4:30	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0.02
7/17/2025 4:31	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0.01
7/17/2025 4:32	0	0.04	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 4:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.01
7/17/2025 4:34	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:35	0	0	0	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0
7/17/2025 4:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 4:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:38	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0
7/17/2025 4:39	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.01
7/17/2025 4:40	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 4:41	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:43	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 4:44	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/17/2025 4:45	0.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:46	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0.04	0	0
7/17/2025 4:47	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0
7/17/2025 4:48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:51	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 4:52	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:56	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 4:57	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/17/2025 4:58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 4:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:02	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 5:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:04	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0	0	0.01
7/17/2025 5:05	0.04	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 5:06	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:08	0	0	0	0	0	0	0.04	0	0	0.03	0	0	0	0	0	0
7/17/2025 5:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:11	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/17/2025 5:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:16	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:19	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:22	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:23	0	0.04	0	0	0	0	0	0	0.04	0	0	0	0	0	0.03	0.01
7/17/2025 5:24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:25	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.01
7/17/2025 5:26	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/17/2025 5:27	0	0	0	0	0	0	0.04	0	0	0	0	0	0.04	0	0	0
7/17/2025 5:28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:29	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0
7/17/2025 5:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:31	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:32	0	0.04	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0.01
7/17/2025 5:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:34	0	0	0	0	0	0	0	0.04	0.04	0	0	0	0	0	0	0.01

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 5:35	0	0	0.04	0	0	0	0	0	0	0	0	0.04	0.04	0	0	0
7/17/2025 5:36	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:37	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 5:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0
7/17/2025 5:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:41	0	0.04	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0
7/17/2025 5:42	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:44	0.04	0	0	0	0	0	0	0.04	0.04	0	0	0	0.04	0	0	0
7/17/2025 5:45	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/17/2025 5:46	0	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0.01
7/17/2025 5:47	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:48	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:50	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 5:51	0	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0.08	0
7/17/2025 5:52	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:53	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:55	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0.01
7/17/2025 5:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 5:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 5:58	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 5:59	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/17/2025 6:00	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 6:01	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0.01
7/17/2025 6:02	0.04	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 6:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 6:05	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:06	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0
7/17/2025 6:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:08	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 6:09	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:10	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0.04	0
7/17/2025 6:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 6:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:16	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.01
7/17/2025 6:19	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:20	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:21	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:22	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 6:23	0	0	0	0	0	0	0	0	0	0	0.03	0	0	0	0	0
7/17/2025 6:24	0	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0
7/17/2025 6:25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:27	0.04	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0
7/17/2025 6:28	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:31	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 6:33	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:34	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0
7/17/2025 6:35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:36	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:37	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:41	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.02
7/17/2025 6:42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:47	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:48	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0	0
7/17/2025 6:49	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:50	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:51	0	0	0	0	0	0	0	0	0	0	0	0.04	0	0	0	0.01
7/17/2025 6:52	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:53	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:54	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:56	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:57	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:58	0	0	0.04	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 6:59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Date	PS8	BR01	BR06	BR07	IN11	IN12	ML08	ML09	ML10	TU03	TU04	TU06	TU07	TU08	TU09	IN08
7/17/2025 7:02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:06	0	0	0	0	0	0	0	0.04	0	0	0	0	0	0	0	0
7/17/2025 7:07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:08	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:09	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/17/2025 7:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

APPENDIX M.3 – HEC RAS PIPE NETWORK INFORMATION

EXISTING CONDITIONS HEC RAS NODE ATTRIBUTE TABLE

EXISTING CONDITIONS HEC RAS CONDUIT ATTRIBUTE TABLE

PROPOSED CONDITIONS HEC RAS NODE ATTRIBUTE TABLE

PROPOSED CONDITIONS HEC RAS CONDUIT ATTRIBUTE TABLE

M INTERIOR DRAINAGE

M.3 HEC RAS PIPE NETWORK INFORMATION

The HEC RAS node and conduit attribute tables are included below for both existing conditions and proposed conditions.

Node coordinates are in the North Kansas State Plane projection. Node elevations (Elev) are in feet NAVD 88.

Existing nodes have no HEC-RAS pipe network side inlet attributes.

All conduits have zero upstream (US) and downstream (DS) offset. US Entrance, DS Exit, US Backflow, and DS Backflow coefficients are all set to 0.5. If DS Flap Gate equals Flap, then that conduit has a no negative flow flap gate, preventing water from Turkey Creek entering the leveed area through the pipe.

M INTERIOR DRAINAGE

M.3.1 EXISTING CONDITIONS HEC RAS NODE ATTRIBUTE TABLE

Table M.3.1. Existing Conditions Node Attribute Table

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
0	Node 65	ST-04	Junction	Junction with drop inlet	1:01	926.92	20	932.51	5.59	Top Inlet 1	932.51	932.51	5	3	2.5	0.67	2249371.75	265356.69
1	Node 5	ST-04	Start	US Junction, with drop inlet	0:01	930	20	935.09	5.09	Top Inlet 1	935.09	935.09	5	3	2.5	0.67	2249372.18	265323.07
2	Node 91	ST-04	Junction	Junction with drop inlet	1:01	923.05	20	932.03	8.98	Top Inlet 1	932.03	932.03	5	3	2.5	0.67	2249406.83	265360.06
3	Node 68	ST-04	Culvert Opening	DS Culvert opening	1:00	919.03	0	918.25	-0.78								2249519.87	265369.71
4	Node 702	ST-07	Start	US Junction, with drop inlet	0:01	929.96	20	932.41	2.45	Top Inlet 1	932.41	932.41	5	3	2.5	0.67	2249702.99	265343.08
5	Node 707 (1)	ST-07	Junction	Junction w/o drop inlet	2:01	929.87	20	932.61	2.74	Top Inlet 2	932.61	932.61	0	3	0	0.67	2249706.31	265352.49
6	Node 826	ST-07	Start	US Junction, with drop inlet	0:01	930.4	20	932.14	1.74	Top Inlet 1	932.14	932.14	5	3	2.5	0.67	2249756.98	265344.94
7	Node 87	ST-02	Culvert Opening	DS Culvert opening	1:00	923.17	0	922.67	-0.50								2248973.50	265539.63
8	Node 854	ST-05	Culvert Opening	DS Culvert opening	1:00	918.07	0	917.55	-0.52								2249690.28	265521.96
9	Node 334	ST-07	Culvert Opening	DS Culvert opening	1:00	927.72	0	927.57	-0.15					3		0.67	2249699.31	265449.40
10	Node 72	ST-03	Start	US Junction, with drop inlet	0:01	931.26	6	935.71	4.45	Top Inlet 3	935.71	935.71	6.28	3	3.14	0.67	2249090.21	265673.20

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
11	Node 1	ST-02	Junction	Junction with drop inlet	1:01	926.34	20	934.31	7.97	Top Inlet 1	934.31	934.31	5	3	2.5	0.67	2248969.49	265612.34
12	Node 64	ST-03	Junction	Junction with drop inlet	1:01	930.73	20	934.64	3.91	Top Inlet 1	934.64	934.64	5	3	2.5	0.67	2249202.45	265668.79
13	Node 60	ST-03	Junction	Junction with drop inlet	1:01	927.94	20	931.66	3.72	Top Inlet 1	931.66	931.66	5	3	2.5	0.67	2249261.70	265636.55
14	Node 62	ST-03	Junction	Junction with drop inlet	2:01	929.99	20	933.7	3.71	Top Inlet 1	933.7	933.7	5	3	2.5	0.67	2249256.68	265672.45
15	Node 88	ST-03	Culvert Opening	DS Culvert opening	1:00	924.1	0	924.85	0.75								2249265.77	265587.09
16	Node 685	ST-05	Start	US Junction, with drop inlet	0:01	927.74	32	930.19	2.45	Top Inlet 4	930.19	930.19	8	3	4	0.67	2249473.18	265664.41
17	Node 799	ST-05	Junction	Junction with drop inlet	1:01	924.91	44	928.01	3.10	Top Inlet 5	928.01	928.01	11	3	5.5	0.67	2249654.58	265627.86
18	Node 1075	ST-05	Junction	Junction with drop inlet	1:01	926.31	32	929.32	3.01	Top Inlet 4	929.32	929.32	8	3	4	0.67	2249597.52	265652.11
19	Node 390	ST-05	Junction	Junction with drop inlet	1:01	925.52	44	928.17	2.65	Top Inlet 5	928.17	928.17	11	3	5.5	0.67	2249648.81	265656.80
20	Node 52	ST-02	Junction	Junction with drop inlet	2:01	932.66	44	935.66	3.00	Top Inlet 5	935.66	935.66	11	3	5.5	0.67	2248950.80	265732.45
21	Node 53	ST-02	Start	US Junction, with drop inlet	0:01	933.2	44	936.2	3.00	Top Inlet 5	936.2	936.2	11	3	5.5	0.67	2248913.97	265747.89
22	Node 541	ST-10	Start	US Junction, with drop inlet	0:01	920.35	25	923.34	2.99	Top Inlet 4	923.34	923.34	8	3	4	0.67	2250174.63	265749.82
23	Node 50	ST-10	Culvert Opening	DS Culvert opening	1:00	918.21	0	917.75	-0.46					3		0.67	2250162.29	265781.86

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
24	Node 70	ST-20	Start	US Junction, with drop inlet	0:01	922.4	3.14	924.79	2.39	Top Inlet 3	924.79	924.79	6.28	3	3.14	0.67	2250797.91	265738.01
25	Node 972	ST-20	Junction	Junction with drop inlet	1:01	922.3	20	925.3	3.00	Top Inlet 1	925.3	925.3	5	3	2.5	0.67	2250819.48	265763.01
26	Node 54	ST-02	Junction	Junction with drop inlet	1:01	936.17	20	939.17	3.00	Top Inlet 1	939.17	939.17	5	3	2.5	0.67	2248912.78	265843.52
27	Node 55	ST-02	Junction	Junction with drop inlet	1:01	935.22	20	938.22	3.00	Top Inlet 1	938.22	938.22	5	3	2.5	0.67	2248951.39	265836.99
28	Node 63	ST-03	Start	US Junction, with drop inlet	0:01	932.17	6	935.88	3.71	Top Inlet 6	935.88	935.88	10	3	6	0.67	2249281.62	265860.68
29	Node 415	ST-09	Culvert Opening	DS Culvert opening	1:00	914.76	0	913.27	-1.49								2250199.49	265922.29
30	Node 540	ST-20	Junction	Junction with drop inlet	1:01	922.11	44	925.11	3.00	Top Inlet 5	925.11	925.11	11	3	5.5	0.67	2250882.95	265840.57
31	Node 9	ST-09	Start	US Junction, with drop inlet	0:01	936.46	20	939.46	3.00	Top Inlet 1	939.46	939.46	5	3	2.5	0.67	2249360.68	266051.67
32	Node 761	ST-09	Junction	Junction with drop inlet	1:01	932.11	20	935.11	3.00	Top Inlet 1	935.11	935.11	5	3	2.5	0.67	2249563.03	266061.51
33	Node 389	ST-09	Junction	Junction with drop inlet	1:01	932.56	20	935.56	3.00	Top Inlet 1	935.56	935.56	5	3	2.5	0.67	2249531.99	266057.98
34	Node 893	ST-09	Junction	Junction with drop inlet	1:01	923.38	20	928.51	5.13	Top Inlet 1	928.51	928.51	5	3	2.5	0.67	2249889.38	265985.79
35	Node 1035	ST-20	Start	US Junction, with drop inlet	0:01	923.6	16	926.6	3.00	Top Inlet 4	926.6	926.6	8	3	4	0.67	2250754.78	265953.07
36	Node 75	ST-20	Junction	Junction with drop	2:01	920.64	20	923.64	3.00	Top Inlet 1	923.64	923.64	5	3	2.5	0.67	2250913.49	265944.03

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
37	Node 237	ST-09	Junction	Junction with drop inlet	1:01	927.72	20	930.72	3.00	Top Inlet 1	930.72	930.72	5	3	2.5	0.67	2249795.24	266082.00
38	Node 2	ST-10	Start	US Junction, with drop inlet	0:01	919.34	25	923.03	3.69	Top Inlet 4	923.03	923.03	8	3	4	0.67	2250453.22	266083.69
39	Node 4	ST-10	Culvert Opening	DS Culvert opening	1:00	913.82	0	910.51	-3.31								2250450.38	266149.17
40	Node 411	ST-20	Start	US Junction, with drop inlet	0:01	919.91	3.14	922.91	3.00	Top Inlet 3	922.91	922.91	6.28	3	3.14	0.67	2251129.95	266132.01
41	Node 371	ST-20	Junction	Junction with drop inlet	1:01	919.45	20	922.45	3.00	Top Inlet 1	922.45	922.45	5	3	2.5	0.67	2251115.42	266140.03
42	Node 1052	ST-20	Junction	Junction with drop inlet	2:01	919.26	20	922.26	3.00	Top Inlet 1	922.26	922.26	5	3	2.5	0.67	2251082.22	266163.38
43	Node 56	ST-02	Junction	Junction with drop inlet	1:01	951.56	20	954.56	3.00	Top Inlet 1	954.56	954.56	5	3	2.5	0.67	2248898.53	266214.18
44	Node 908	ST-11	Junction	Junction with drop inlet	1:01	916.15	25	922.02	5.87	Top Inlet 4	922.02	922.02	8	3	4	0.67	2250282.65	266229.70
45	Node 648	ST-11	Junction	Junction with drop inlet	1:01	917.55	25	920.88	3.33	Top Inlet 4	920.88	920.88	8	3	4	0.67	2250266.89	266315.63
46	Node 276	ST-11	Culvert Opening	DS Culvert opening	1:00	914.39	0	909.59	-4.80								2250452.03	266221.81
47	Node 658	ST-16	Start	US Junction, with drop inlet	0:01	919.92	10	922.92	3.00	Top Inlet 6	922.92	922.92	10	3	6	0.67	2250933.09	266212.71
48	Node 247	ST-16	Junction	Junction with drop inlet	1:01	919.74	10	922.74	3.00	Top Inlet 6	922.74	922.74	10	3	6	0.67	2250907.74	266211.67
49	Node	ST-16	Start	US Junction,	0:01	918.24	10	921.24	3.00	Top Inlet	921.24	921.24	10	3	6	0.67	2250931	266283.

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
	963			with drop inlet						6							.72	67
50	Node 534	ST-16	Junction	Junction with drop inlet	2:01	918.14	10	920.9	2.76	Top Inlet 6	920.9	920.9	10	3	6	0.67	2250906.54	266282.37
51	Node 769	ST-11	Start	US Junction, with drop inlet	0:01	918.81	20	920.81	2.00	Top Inlet 1	920.81	920.81	5	3	2.5	0.67	2250231.13	266368.28
52	Node 361	ST-11	Junction	Junction with drop inlet	1:01	918.07	20	920.87	2.80	Top Inlet 1	920.87	920.87	5	3	2.5	0.67	2250263.11	266369.66
53	Node 640	ST-20	Junction	Junction with drop inlet	1:01	918.42	20	921.42	3.00	Top Inlet 1	921.42	921.42	5	3	2.5	0.67	2251177.23	266337.08
54	Node 58	ST-02	Junction	Junction with drop inlet	1:01	972.15	20	975.15	3.00	Top Inlet 1	975.15	975.15	5	3	2.5	0.67	2248881.12	266539.49
55	Node 1081	ST-16	Start	US Junction, with drop inlet	0:01	913.63	10	916.63	3.00	Top Inlet 6	916.63	916.63	10	3	6	0.67	2250922.17	266547.13
56	Node 129	ST-16	Junction	Junction with drop inlet	2:01	913.46	10	916.46	3.00	Top Inlet 6	916.46	916.46	10	3	6	0.67	2250896.13	266545.39
57	Node 354	ST-20	Start	US Junction, with drop inlet	0:01	916.48	20	919.48	3.00	Top Inlet 1	919.48	919.48	5	3	2.5	0.67	2251272.52	266458.32
58	Node 766	ST-20	Junction	Junction with drop inlet	2:01	916.41	20	919.41	3.00	Top Inlet 1	919.41	919.41	5	3	2.5	0.67	2251233.51	266474.06
59	Node 73	ST-02	Junction	Junction with drop inlet	2:01	978.6	20	982.02	3.42	Top Inlet 1	982.02	982.02	5	3	2.5	0.67	2248878.80	266635.41
60	Node 74	ST-02	Start	US Junction, with drop inlet	0:01	978.7	20	981.7	3.00	Top Inlet 1	981.7	981.7	5	3	2.5	0.67	2248917.74	266636.70
61	Node 857	ST-15	Start	US Junction, with drop	0:01	968.26	3.14	971.26	3.00	Top Inlet 3	971.26	971.26	6.28	3	3.14	0.67	2249455.14	266594.01

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
62	Node 680	ST-15	Junction	Junction with drop inlet	1:01	968.17	3.14	971.37	3.20	Top Inlet 3	971.37	971.37	6.28	3	3.14	0.67	2249483.39	266628.78
63	Node 248	ST-15	Junction	Junction with drop inlet	1:01	964.29	3.14	967.29	3.00	Top Inlet 3	967.29	967.29	6.28	3	3.14	0.67	2249631.59	266623.85
64	Node 393	ST-15	Junction	Junction with drop inlet	1:01	952.21	3.14	955.21	3.00	Top Inlet 3	955.21	955.21	6.28	3	3.14	0.67	2249758.84	266621.92
65	Node 795	ST-15	Junction	Junction with drop inlet	2:01	932.2	3.14	935.2	3.00	Top Inlet 3	935.2	935.2	6.28	3	3.14	0.67	2249939.45	266630.57
66	Node 448	ST-15	Start	US Junction, with drop inlet	0:01	932.3	3.14	934.99	2.69	Top Inlet 3	934.99	934.99	6.28	3	3.14	0.67	2249941.33	266597.30
67	Node 101	ST-15	Junction	Junction with drop inlet	1:01	923.34	3.14	926.34	3.00	Top Inlet 3	926.34	926.34	6.28	3	3.14	0.67	2250124.85	266638.83
68	Node 830	ST-15	Junction	Junction with drop inlet	1:01	921.18	3.14	924.18	3.00	Top Inlet 3	924.18	924.18	6.28	3	3.14	0.67	2250216.28	266652.23
69	Node 561	ST-15	Junction	Junction with drop inlet	3:01	920.61	3.14	923.61	3.00	Top Inlet 3	923.61	923.61	6.28	3	3.14	0.67	2250270.34	266648.98
70	Node 430	ST-15	Start	US Junction, with drop inlet	0:01	920.94	3.14	923.94	3.00	Top Inlet 3	923.94	923.94	6.28	3	3.14	0.67	2250223.72	266595.15
71	Node 690	ST-15	Start	US Junction, with drop inlet	0:01	913.94	3.14	917.74	3.80	Top Inlet 3	917.74	917.74	6.28	3	3.14	0.67	2250476.03	266622.61
72	Node 303	ST-15	Junction	Junction with drop inlet	2:01	911.31	20	917.68	6.37	Top Inlet 1	917.68	917.68	5	3	2.5	0.67	2250473.61	266656.58
73	Node 984	ST-15	Junction	Junction with drop inlet	1:01	910.38	25	916.88	6.50	Top Inlet 4	916.88	916.88	8	3	4	0.67	2250551.46	266658.68
74	Node	ST-15	Culvert	DS Culvert	1:00	908.68	0	908.14	-0.54				6.28	3	3.14	0.67	2250661	266680.

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
	1022		Opening	opening													.95	90
75	Node 359	ST-16	Junction	Junction with drop inlet	1:01	909.66	20	915.25	5.59	Top Inlet 1	915.25	915.25	5	3	2.5	0.67	2250925 .64	266630. 46
76	Node 222	ST-16	Junction	Junction with drop inlet	2:01	909.31	20	915.1	5.79	Top Inlet 1	915.1	915.1	5	3	2.5	0.67	2250903 .35	266646. 63
77	Node 620	ST-16	Start	US Junction, with drop inlet	0:01	910.77	10	915.63	4.86	Top Inlet 6	915.63	915.63	10	3	6	0.67	2250950 .29	266678. 38
78	Node 484	ST-16	Junction	Junction with drop inlet	2:01	909.94	10	915.19	5.25	Top Inlet 6	915.19	915.19	10	3	6	0.67	2250955 .15	266652. 42
79	Node 760	ST-16	Junction	Junction with drop inlet	2:01	911.68	10	914.68	3.00	Top Inlet 6	914.68	914.68	10	3	6	0.67	2251104 .98	266659. 44
80	Node 742	ST-16	Junction	Junction with drop inlet	1:01	911.84	10	914.84	3.00	Top Inlet 6	914.84	914.84	10	3	6	0.67	2251148 .38	266661. 19
81	Node 100	ST-16	Junction	Junction with drop inlet	1:01	911.67	10	914.67	3.00	Top Inlet 6	914.67	914.67	10	3	6	0.67	2251096 .48	266658. 94
82	Node 1045	ST-16	Start	US Junction, with drop inlet	0:01	912.22	10	915.22	3.00	Top Inlet 6	915.22	915.22	10	3	6	0.67	2251145 .13	266686. 68
83	Node 358	ST-16	Start	US Junction, with drop inlet	0:01	912.16	10	915.16	3.00	Top Inlet 6	915.16	915.16	10	3	6	0.67	2251101 .34	266684. 88
84	Node 641	ST-20	Junction	Junction with drop inlet	2:01	911.42	20	914.22	2.80	Top Inlet 1	914.22	914.22	5	3	2.5	0.67	2251274 .37	266658. 52
85	Node 206	ST-20	Start	US Junction, with drop inlet	0:01	912.98	20	915.98	3.00	Top Inlet 1	915.98	915.98	5	3	2.5	0.67	2251331 .03	266640. 98
86	Node 535	ST-15	Start	US Junction, with drop inlet	0:01	922.24	3.14	927.24	5.00	Top Inlet 3	927.24	927.24	6.28	3	3.14	0.67	2250122 .32	266742. 59

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
87	Node 545	ST-15	Junction	Junction with drop inlet	1:01	922.42	3.14	925.42	3.00	Top Inlet 3	925.42	925.42	6.28	3	3.14	0.67	2250219.12	266698.37
88	Node 123	ST-15	Junction	Junction with drop inlet	1:01	922.09	3.14	925.09	3.00	Top Inlet 3	925.09	925.09	6.28	3	3.14	0.67	2250254.10	266689.28
89	Node 332	ST-16	Culvert Opening	DS Culvert opening	1:00	908.43	0	908.3	-0.13								2250792.96	266721.83
90	Node 509	ST-20	Junction	Junction with drop inlet	1:01	911.32	20	914.32	3.00	Top Inlet 1	914.32	914.32	5	3	2.5	0.67	2251271.61	266690.49
91	Node 78	ST-02	Junction	Junction with drop inlet	1:01	991.81	20	994.81	3.00	Top Inlet 1	994.81	994.81	5	3	2.5	0.67	2248872.09	266818.50
92	Node 79	ST-02	Junction	Junction with drop inlet	1:01	1002.93	20	1005.93	3.00	Top Inlet 1	1005.93	1005.93	5	3	2.5	0.67	2248864.26	267021.73
93	Node 596	ST-15B	Culvert Opening	DS Culvert opening	1:00	975.58	0	975.57	-0.01				6.28	3	3.14	0.67	2249530.37	266979.84
94	Node 903	ST-20	Junction	Junction with drop inlet	1:01	907.57	20	910.57	3.00	Top Inlet 1	910.57	910.57	5	3	2.5	0.67	2251338.96	266941.54
95	Node 11	ST-20	Start	US Junction, with drop inlet	0:01	907.99	3.14	910.64	2.65	Top Inlet 3	910.64	910.64	6.28	3	3.14	0.67	2251411.77	267046.34
96	Node 80	ST-02	Start	US Junction, with drop inlet	0:01	1008.02	20	1011.02	3.00	Top Inlet 1	1011.02	1011.02	5	3	2.5	0.67	2248843.61	267105.58
97	Node 754	ST-15B	Junction	Junction with drop inlet	1:01	987.15	3.14	990.15	3.00	Top Inlet 3	990.15	990.15	6.28	3	3.14	0.67	2249423.33	267060.55
98	Node 887	ST-15B	Start	US Junction, with drop inlet	0:01	987.22	3.14	990.12	2.90	Top Inlet 3	990.12	990.12	6.28	3	3.14	0.67	2249399.82	267085.83
99	Node 767	ST-20	Junction	Junction with drop inlet	2:01	907.07	20	910.07	3.00	Top Inlet 1	910.07	910.07	5	3	2.5	0.67	2251347.53	267099.46

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
100	Node 333	ST-20	Junction	Junction with drop inlet	1:01	907.89	20	910.89	3.00	Top Inlet 1	910.89	910.89	5	3	2.5	0.67	2251390.30	267098.98
101	Node 12	ST-19	Start	US Junction, with drop inlet	0:01	990.44	20	993.44	3.00	Top Inlet 1	993.44	993.44	5	3	2.5	0.67	2249384.02	267274.42
102	Node 13	ST-19	Junction	Junction with drop inlet	1:01	988.17	20	991.17	3.00	Top Inlet 1	991.17	991.17	5	3	2.5	0.67	2249432.14	267298.67
103	Node 15	ST-19	Start	US Junction, with drop inlet	0:01	971.84	20	974.84	3.00	Top Inlet 1	974.84	974.84	5	3	2.5	0.67	2249776.74	267279.56
104	Node 16	ST-19	Junction	Junction with drop inlet	2:01	971.68	20	974.68	3.00	Top Inlet 1	974.68	974.68	5	3	2.5	0.67	2249774.91	267304.18
105	Node 21	ST-19	Start	US Junction, with drop inlet	0:01	963.91	20	966.91	3.00	Top Inlet 1	966.91	966.91	5	3	2.5	0.67	2250114.73	267289.85
106	Node 1025 (1)	ST-20	Junction	Junction with drop inlet	2:01	902.93	44	909.37	6.44	Top Inlet 5	909.37	909.37	11	3	5.5	0.67	2251338.04	267300.65
107	Node 22	ST-19	Junction	Junction with drop inlet	2:01	963.28	20	966.28	3.00	Top Inlet 1	966.28	966.28	5	3	2.5	0.67	2250113.26	267317.03
108	Node 23	ST-19	Junction	Junction with drop inlet	1:01	959.69	20	962.69	3.00	Top Inlet 1	962.69	962.69	5	3	2.5	0.67	2250187.11	267320.34
109	Node 24	ST-19	Junction	Junction with drop inlet	1:01	954.23	20	958.23	4.00	Top Inlet 1	958.23	958.23	5	3	2.5	0.67	2250223.85	267351.94
110	Node 47	ST-19	Junction	Junction with drop inlet	1:01	911.15	25	914.92	3.77	Top Inlet 4	914.92	914.92	8	3	4	0.67	2251031.71	267363.69
111	Node 48	ST-19	Junction	Junction with drop inlet	1:01	911.65	20	916.17	4.52	Top Inlet 1	916.17	916.17	5	3	2.5	0.67	2251003.06	267358.18
112	Node 49	ST-19	Start	US Junction, with drop	0:01	914.7	20	918.87	4.17	Top Inlet 1	918.87	918.87	5	3	2.5	0.67	2250978.07	267331.00

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
113	Node 614 (1)	ST-20	Start	US Junction, with drop inlet	0:01	905.64	44	910.14	4.50	Top Inlet 5	910.14	910.14	11	3	5.5	0.67	2251379.30	267319.90
114	Node 25	ST-19	Start	US Junction, with drop inlet	0:01	943.56	20	946.56	3.00	Top Inlet 1	946.56	946.56	5	3	2.5	0.67	2250037.58	267498.15
115	Node 26	ST-19	Start	US Junction, with drop inlet	0:01	943.47	20	946.47	3.00	Top Inlet 1	946.47	946.47	5	3	2.5	0.67	2250181.23	267513.95
116	Node 27	ST-19	Junction	Junction with drop inlet	2:01	935.99	20	947.48	11.49	Top Inlet 1	947.48	947.48	5	3	2.5	0.67	2250179.02	267533.42
117	Node 28	ST-19	Junction	Junction with drop inlet	1:01	942.6	20	946.6	4.00	Top Inlet 1	946.6	946.6	5	3	2.5	0.67	2250207.31	267516.89
118	Node 29	ST-19	Junction	Junction with drop inlet	2:01	934.81	20	947.47	12.66	Top Inlet 1	947.47	947.47	5	3	2.5	0.67	2250212.82	267532.32
119	Node 30	ST-19	Junction	Junction with drop inlet	1:01	928.26	20	935.9	7.64	Top Inlet 1	935.9	935.9	5	3	2.5	0.67	2250399.09	267535.26
120	Node 37	ST-19	Junction	Junction with drop inlet	1:01	920.52	20	927.99	7.47	Top Inlet 1	927.99	927.99	5	3	2.5	0.67	2250618.78	267517.62
121	Node 38	ST-19	Junction	Junction with drop inlet	2:01	916.65	20	921.05	4.40	Top Inlet 1	921.05	921.05	5	3	2.5	0.67	2250836.53	267526.71
122	Node 43	ST-19	Junction	Junction with drop inlet	2:01	908.63	25	913.46	4.83	Top Inlet 4	913.46	913.46	8	3	4	0.67	2251039.85	267539.54
123	Node 31	ST-19	Start	US Junction, with drop inlet	0:01	947.15	20	950.15	3.00	Top Inlet 1	950.15	950.15	5	3	2.5	0.67	2250328.55	267647.68
124	Node 32	ST-19	Junction	Junction with drop inlet	1:01	941.56	20	944.56	3.00	Top Inlet 1	944.56	944.56	5	3	2.5	0.67	2250420.39	267652.45

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
125	Node 33	ST-19	Junction	Junction with drop inlet	1:01	940.89	20	943.89	3.00	Top Inlet 1	943.89	943.89	5	3	2.5	0.67	2250438.03	267673.03
126	Node 36	ST-19	Junction	Junction with drop inlet	1:01	931.44	20	934.44	3.00	Top Inlet 1	934.44	934.44	5	3	2.5	0.67	2250709.89	267674.86
127	Node 39	ST-19	Junction	Junction with drop inlet	1:01	922.54	20	925.54	3.00	Top Inlet 1	925.54	925.54	5	3	2.5	0.67	2250829.29	267627.84
128	Node 46	ST-19	Culvert Opening	DS Culvert opening	1:00	906.46	0	895.94	-10.52								2251136.58	267557.25
129	Node 35	ST-19	Junction	Junction with drop inlet	1:01	934.96	20	937.96	3.00	Top Inlet 1	937.96	937.96	5	3	2.5	0.67	2250634.58	267681.84
130	Node 44	ST-20	Culvert Opening	DS Culvert opening	1:00	900.25	0	898.45	-1.80								2251238.41	267689.75
131	Node 447	ST-22	Culvert Opening	DS Culvert opening	1:00	898.68	0	897.9	-0.78								2251259.76	267799.91
132	Node 192	ST-22	Start	US Junction, with drop inlet	0:01	908.6	32	910.76	2.16	Top Inlet 4	910.76	910.76	8	3	4	0.67	2251361.44	267802.91
133	Node 170	A-12	Start	US Junction, with drop inlet	0:01	990.35	20	993.35	3.00	Top Inlet 1	993.35	993.35	5	3	2.5	0.67	2249318.46	267906.03
134	Node 862	A-12	Start	US Junction, with drop inlet	0:01	972.06	32	975.06	3.00	Top Inlet 4	975.06	975.06	8	3	4	0.67	2249617.87	267920.79
135	Node 730	ST-23	Start	US Junction, with drop inlet	0:01	912.06	20	914.36	2.30	Top Inlet 1	914.36	914.36	5	3	2.5	0.67	2251087.39	267858.41
136	Node 883	ST-23	Culvert Opening	DS Culvert opening	1:00	907.13	0	899.33	-7.80								2251171.39	267862.62
137	Node 41	ST-23	Junction	Junction with drop	1:01	910.39	20	913.64	3.25	Top Inlet 1	913.64	913.64	5	3	2.5	0.67	2251119.82	267858.34

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
138	Node 249	A-12	Junction	Junction with drop inlet	1:01	990.06	20	993.06	3.00	Top Inlet 1	993.06	993.06	5	3	2.5	0.67	2249315.80	267960.47
139	Node 403	A-12	Junction	Junction with drop inlet	2:01	969.9	32	972.9	3.00	Top Inlet 4	972.9	972.9	8	3	4	0.67	2249647.84	267976.48
140	Node 451	A-12	Start	US Junction, with drop inlet	0:01	962.19	32	965.19	3.00	Top Inlet 4	965.19	965.19	8	3	4	0.67	2249882.14	267932.24
141	Node 1084	A-12	Junction	Junction with drop inlet	2:01	962.04	32	965.04	3.00	Top Inlet 4	965.04	965.04	8	3	4	0.67	2249879.06	267986.89
142	Node 318	A-12	Culvert Opening	DS Culvert opening	1:00	951.61	0	951.6	-0.01								2249876.55	268026.17
143	Node 122	ST-27	Junction	Junction with drop inlet	1:01	949.96	20	952.96	3.00	Top Inlet 1	952.96	952.96	5	3	2.5	0.67	2250473.91	268014.56
144	Node 40	ST-27	Start	US Junction, with drop inlet	0:01	949.99	20	952.89	2.90	Top Inlet 1	952.89	952.89	5	3	2.5	0.67	2250476.16	267960.96
145	Node 792	ST-27	Start	US Junction, with drop inlet	0:01	935.64	20	938.04	2.40	Top Inlet 1	938.04	938.04	5	3	2.5	0.67	2250799.22	267975.59
146	Node 555	ST-27	Junction	Junction with drop inlet	2:01	935.54	20	938.54	3.00	Top Inlet 1	938.54	938.54	5	3	2.5	0.67	2250793.39	268029.47
147	Node 978	ST-27	Junction	Junction with drop inlet	1:01	915.12	20	919.97	4.85	Top Inlet 1	919.97	919.97	5	3	2.5	0.67	2251099.64	268043.70
148	Node 104	ST-26	Start	US Junction, with drop inlet	0:01	907.01	32	910.01	3.00	Top Inlet 4	910.01	910.01	8	3	4	0.67	2251443.41	267994.70
149	Node 1021	ST-27	Culvert Opening	DS Culvert opening	1:00	895.5	0	895.7	0.20								2251149.04	268102.07

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
150	Node 152	ST-26	Culvert Opening	DS Culvert opening	1:00	899.11	0	898.58	-0.53								2251231.43	268065.50
151	Node 85	ST-26	Junction	Junction with drop inlet	1:01	900	20	912.34	12.34	Top Inlet 1	912.34	912.34	5	3	2.5	0.67	2251285.39	268065.70
152	Node 3	ST-26	Junction	Junction with drop inlet	1:01	906.65	32	909.77	3.12	Top Inlet 4	909.77	909.77	8	3	4	0.67	2251443.88	268056.74
153	Node 590	A-3	Start	US Junction, with drop inlet	0:01	961.58	3.14	964.58	3.00	Top Inlet 3	964.58	964.58	6.28	3	3.14	0.67	2249336.49	268225.06
154	Node 10	A-3	Start	US Junction, with drop inlet	0:01	956.84	3.14	959.84	3.00	Top Inlet 3	959.84	959.84	6.28	3	3.14	0.67	2249316.49	268295.74
155	Node 944	A-3	Junction	Junction with drop inlet	3:01	957.94	3.14	960.94	3.00	Top Inlet 3	960.94	960.94	6.28	3	3.14	0.67	2249377.20	268301.17
156	Node 997	A-3	Junction	Junction with drop inlet	1:01	959.2	3.14	962.2	3.00	Top Inlet 3	962.2	962.2	6.28	3	3.14	0.67	2249365.50	268259.87
157	Node 1020	A-2	Culvert Opening	DS Culvert opening	1:00	933.28	0	933.27	-0.01				6.28	3	3.14	0.67	2250213.39	268281.16
158	Node 495	ST-37	Start	US Junction, with drop inlet	0:01	905.75	20	908.58	2.83	Top Inlet 1	908.58	908.58	5	3	2.5	0.67	2251324.85	268299.07
159	Node 370	ST-37	Start	US Junction, with drop inlet	0:01	908.94	20	911.94	3.00	Top Inlet 1	911.94	911.94	5	3	2.5	0.67	2251657.30	268184.55
160	Node 301	A-3	Start	US Junction, with drop inlet	0:01	958.06	3.14	960.86	2.80	Top Inlet 3	960.86	960.86	6.28	3	3.14	0.67	2249332.57	268312.07
161	Node 700	A-3	Junction	Junction with drop inlet	2:01	957.99	3.14	960.89	2.90	Top Inlet 3	960.89	960.89	6.28	3	3.14	0.67	2249363.66	268312.99
162	Node 17	A-3	Junction	Junction with drop	1:01	961.35	3.14	964.35	3.00	Top Inlet	964.35	964.35	6.28	3	3.14	0.67	2249362	268392.

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet						3							.79	17
163	Node 329	A-3	Junction	Junction with drop inlet	1:01	953.42	3.14	956.42	3.00	Top Inlet 3	956.42	956.42	6.28	3	3.14	0.67	2249483 .92	268326. 49
164	Node 811	A-3	Culvert Opening	DS Culvert opening	1:00	953.13	0	953.12	-0.01				6.28	3	3.14	0.67	2249580 .01	268322. 41
165	Node 57	A-2	Start	US Junction, with drop inlet	0:01	934.92	3.14	937.92	3.00	Top Inlet 3	937.92	937.92	6.28	3	3.14	0.67	2250158 .75	268303. 80
166	Node 1033	A-2	Junction	Junction with drop inlet	2:01	934.99	3.14	937.99	3.00	Top Inlet 3	937.99	937.99	6.28	3	3.14	0.67	2250190 .98	268319. 20
167	Node 1038	ST-37	Junction	Junction with drop inlet	1:01	905.6	20	908.6	3.00	Top Inlet 1	908.6	908.6	5	3	2.5	0.67	2251278 .50	268371. 93
168	Node 464	ST-37	Junction	Junction w/o drop inlet	2:01	905.65	20	908.28	2.63	Top Inlet 2	908.28	908.28	0	3	0	0.67	2251321 .06	268364. 49
169	Node 884	ST-37	Junction	Junction w/o drop inlet	1:01	905.7	20	908.6	2.90	Top Inlet 2	908.6	908.6	0	3	0	0.67	2251339 .25	268331. 60
170	Node 615	ST-37	Start	US Junction, with drop inlet	0:01	905.75	20	908.3	2.55	Top Inlet 1	908.3	908.3	5	3	2.5	0.67	2251353 .36	268356. 87
171	Node 902	ST-37	Junction	Junction with drop inlet	2:01	908.28	20	911.28	3.00	Top Inlet 1	911.28	911.28	5	3	2.5	0.67	2251644 .94	268303. 68
172	Node 777	ST-37	Start	US Junction, with drop inlet	0:01	908.37	20	911.17	2.80	Top Inlet 1	911.17	911.17	5	3	2.5	0.67	2251602 .56	268302. 42
173	Node 20	A-3	Start	US Junction, with drop inlet	0:01	967.7	3.14	970.2	2.50	Top Inlet 3	970.2	970.2	6.28	3	3.14	0.67	2249439 .74	268491. 71
174	Node 808	A-3	Junction	Junction with drop inlet	2:01	967.52	3.14	972.12	4.60	Top Inlet 3	972.12	972.12	6.28	3	3.14	0.67	2249375 .07	268514. 90

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
175	Node 429	A-3	Junction	Junction with drop inlet	1:01	967.61	3.14	970.11	2.50	Top Inlet 3	970.11	970.11	6.28	3	3.14	0.67	2249439.96	268523.59
176	Node 868	A-2	Junction	Junction with drop inlet	1:01	943.68	3.14	946.68	3.00	Top Inlet 3	946.68	946.68	6.28	3	3.14	0.67	2250114.43	268520.34
177	Node 127	A-2	Junction	Junction with drop inlet	1:01	940.11	3.14	943.11	3.00	Top Inlet 3	943.11	943.11	6.28	3	3.14	0.67	2250179.62	268525.19
178	Node 704	A-4	Start	US Junction, with drop inlet	0:01	925.61	3.14	928.61	3.00	Top Inlet 3	928.61	928.61	6.28	3	3.14	0.67	2250570.59	268548.56
179	Node 899	ST-37	Start	US Junction, with drop inlet	0:01	904.55	20	907.25	2.70	Top Inlet 1	907.25	907.25	5	3	2.5	0.67	2251311.88	268528.29
180	Node 492	ST-37	Junction	Junction with drop inlet	1:01	906.75	20	909.75	3.00	Top Inlet 1	909.75	909.75	5	3	2.5	0.67	2251628.77	268471.80
181	Node 556	A-3	Junction	Junction with drop inlet	1:01	972.03	3.14	975.03	3.00	Top Inlet 3	975.03	975.03	6.28	3	3.14	0.67	2249337.28	268550.99
182	Node 1073	A-3	Start	US Junction, with drop inlet	0:01	977.42	3.14	980.42	3.00	Top Inlet 3	980.42	980.42	6.28	3	3.14	0.67	2249294.90	268670.66
183	Node 979	A-3	Junction	Junction with drop inlet	2:01	975.53	3.14	978.53	3.00	Top Inlet 3	978.53	978.53	6.28	3	3.14	0.67	2249331.16	268637.48
184	Node 674	A-3	Start	US Junction, with drop inlet	0:01	977.14	3.14	980.14	3.00	Top Inlet 3	980.14	980.14	6.28	3	3.14	0.67	2249290.99	268643.36
185	Node 290	A-3	Junction	Junction with drop inlet	1:01	976.16	3.14	979.16	3.00	Top Inlet 3	979.16	979.16	6.28	3	3.14	0.67	2249329.01	268671.73
186	Node 454	A-2	Start	US Junction, with drop inlet	0:01	943.78	3.14	946.78	3.00	Top Inlet 3	946.78	946.78	6.28	3	3.14	0.67	2250105.63	268551.72
187	Node	A-4	Culvert	DS Culvert	1:00	921.47	0	921.46	-0.01				6.28	3	3.14	0.67	2250633	268571.

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
	916		Opening	opening													.13	73
188	Node 812	ST-37	Junction	Junction with drop inlet	1:01	904.5	20	906.9	2.40	Top Inlet 1	906.9	906.9	5	3	2.5	0.67	2251317.36	268627.23
189	Node 350	ST-37	Junction	Junction with drop inlet	1:01	904.44	18	907.34	2.90	Top Inlet 4	907.34	907.34	8	3	4	0.67	2251398.19	268669.86
190	Node 128	ST-37	Start	US Junction, with drop inlet	0:01	904.49	20	907.19	2.70	Top Inlet 1	907.19	907.19	5	3	2.5	0.67	2251353.35	268664.78
191	Node 618	ST-37	Junction	Junction with drop inlet	1:01	906.11	20	909.11	3.00	Top Inlet 1	909.11	909.11	5	3	2.5	0.67	2251624.03	268578.26
192	Node 758	ST-37	Start	US Junction, with drop inlet	0:01	905.87	20	908.57	2.70	Top Inlet 1	908.57	908.57	5	3	2.5	0.67	2251590.00	268632.87
193	Node 83	ST-37	Junction	Junction with drop inlet	2:01	905.83	20	908.83	3.00	Top Inlet 1	908.83	908.83	5	3	2.5	0.67	2251626.55	268631.21
194	Node 168	A-6	Start	US Junction, with drop inlet	0:01	945.33	3.14	948.33	3.00	Top Inlet 3	948.33	948.33	6.28	3	3.14	0.67	2249771.37	268787.56
195	Node 69	ST-37	Culvert Opening	DS Culvert opening	1:00	894.75	0	894.49	-0.26					3		0.67	2250911.55	268790.09
196	Node 864	ST-37	Start	US Junction, with drop inlet	0:01	903.83	20	906.83	3.00	Top Inlet 1	906.83	906.83	5	3	2.5	0.67	2251039.04	268760.94
197	Node 177	ST-37	Junction	Junction with drop inlet	2:01	902.53	20	905.53	3.00	Top Inlet 1	905.53	905.53	5	3	2.5	0.67	2251040.60	268777.65
198	Node 1051	ST-37	Start	US Junction, with drop inlet	0:01	904.43	20	907.33	2.90	Top Inlet 1	907.33	907.33	5	3	2.5	0.67	2251171.66	268687.15
199	Node 634	ST-37	Junction	Junction with drop inlet	1:01	904.38	20	907.38	3.00	Top Inlet 1	907.38	907.38	5	3	2.5	0.67	2251174.68	268723.23

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
200	Node 385	ST-37	Junction	Junction with drop inlet	2:01	904.33	20	907.33	3.00	Top Inlet 1	907.33	907.33	5	3	2.5	0.67	2251278.13	268688.06
201	Node 905	ST-37	Junction	Junction w/o drop inlet	3:01	904.4	20	907.3	2.90	Top Inlet 2	907.3	907.3	0	3	0	0.67	2251335.84	268685.45
202	Node 989	ST-37	Start	US Junction, with drop inlet	0:01	904.82	6	907.52	2.70	Top Inlet 7	907.52	907.52	6	3	2	0.67	2251495.13	268694.07
203	Node 275	ST-37	Junction	Junction with drop inlet	1:01	904.71	6	907.71	3.00	Top Inlet 7	907.71	907.71	6	3	2	0.67	2251553.14	268688.50
204	Node 188	ST-37	Junction	Junction w/o drop inlet	2:01	904.65	20	908.45	3.80	Top Inlet 2	908.45	908.45	0	3	0	0.67	2251600.00	268683.91
205	Node 915	ST-37	Junction	Junction with drop inlet	1:01	905.65	20	908.65	3.00	Top Inlet 1	908.65	908.65	5	3	2.5	0.67	2251609.26	268717.93
206	Node 491	ST-37	Junction	Junction with drop inlet	2:01	905.68	20	908.68	3.00	Top Inlet 1	908.68	908.68	5	3	2.5	0.67	2251636.33	268708.33
207	Node 265	A-9	Start	US Junction, with drop inlet	0:01	969.72	3.14	972.72	3.00	Top Inlet 3	972.72	972.72	6.28	3	3.14	0.67	2249342.59	268867.88
208	Node 587	A-6	Junction	Junction with drop inlet	2:01	944.58	3.14	947.58	3.00	Top Inlet 3	947.58	947.58	6.28	3	3.14	0.67	2249784.78	268815.73
209	Node 67	A-6	Start	US Junction, with drop inlet	0:01	946.09	3.14	949.09	3.00	Top Inlet 3	949.09	949.09	6.28	3	3.14	0.67	2249724.33	268824.15
210	Node 61	A-5	Culvert Opening	DS Culvert opening	1:00	918.55	0	918.21	-0.34				6.28	3	3.14	0.67	2250223.07	268890.33
211	Node 828	ST-37	Junction	Junction with drop inlet	2:01	903.47	20	906.29	2.82	Top Inlet 1	906.29	906.29	5	3	2.5	0.67	2251301.94	268909.15
212	Node 1079	ST-37	Junction	Junction with drop inlet	1:01	903.5	32	906.4	2.90	Top Inlet 4	906.4	906.4	8	3	4	0.67	2251357.66	268918.23

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
213	Node 233	ST-37	Junction	Junction with drop inlet	1:01	907.11	20	910.11	3.00	Top Inlet 1	910.11	910.11	5	3	2.5	0.67	2251652.09	268800.11
214	Node 1054	ST-37	Start	US Junction, with drop inlet	0:01	908.54	20	911.54	3.00	Top Inlet 1	911.54	911.54	5	3	2.5	0.67	2251641.51	268904.51
215	Node 369	ST-37	Junction	Junction with drop inlet	1:01	908.47	20	911.47	3.00	Top Inlet 1	911.47	911.47	5	3	2.5	0.67	2251668.39	268890.49
216	Node 425	A-9	Junction	Junction with drop inlet	1:01	958.44	3.14	961.44	3.00	Top Inlet 3	961.44	961.44	6.28	3	3.14	0.67	2249334.99	269044.70
217	Node 743	A-6	Culvert Opening	DS Culvert opening	1:00	932.32	0	932.31	-0.01				6.28	3	3.14	0.67	2249857.18	268948.13
218	Node 566	A-5	Junction	Junction with drop inlet	1:01	920.09	3.14	922.57	2.48	Top Inlet 3	922.57	922.57	6.28	3	3.14	0.67	2250177.58	268926.31
219	Node 6	A-5	Junction	Junction with drop inlet	1:01	923.67	3.14	928.16	4.49	Top Inlet 3	928.16	928.16	6.28	3	3.14	0.67	2250175.07	269028.75
220	Node 721	ST-37	Junction	Junction with drop inlet	1:01	897.75	20	905.11	7.36	Top Inlet 1	905.11	905.11	5	3	2.5	0.67	2251056.05	269037.37
221	Node 42	ST-37	Start	US Junction, with drop inlet	0:01	898.92	20	907.95	9.03	Top Inlet 1	907.95	907.95	5	3	2.5	0.67	2251182.56	269025.99
222	Node 783	ST-37	Junction	Junction with drop inlet	1:01	903.43	20	906.43	3.00	Top Inlet 1	906.43	906.43	5	3	2.5	0.67	2251306.20	268923.68
223	Node 1050	ST-37	Start	US Junction, with drop inlet	0:01	909.09	20	912.09	3.00	Top Inlet 1	912.09	912.09	5	3	2.5	0.67	2251569.57	268947.30
224	Node 442	A-9	Junction	Junction with drop inlet	2:01	953.95	3.14	956.95	3.00	Top Inlet 3	956.95	956.95	6.28	3	3.14	0.67	2249332.28	269084.36
225	Node 406	A-9	Start	US Junction, with drop	0:01	961.92	3.14	964.92	3.00	Top Inlet 3	964.92	964.92	6.28	3	3.14	0.67	2249256.31	269089.20

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
226	Node 831	A-9	Junction	Junction with drop inlet	2:01	947.29	3.14	950.29	3.00	Top Inlet 3	950.29	950.29	6.28	3	3.14	0.67	2249388.08	269094.47
227	Node 153	A-9	Start	US Junction, with drop inlet	0:01	947.35	3.14	950.35	3.00	Top Inlet 3	950.35	950.35	6.28	3	3.14	0.67	2249382.83	269124.90
228	Node 579	A-9	Start	US Junction, with drop inlet	0:01	938.09	3.14	941.09	3.00	Top Inlet 3	941.09	941.09	6.28	3	3.14	0.67	2249475.58	269155.11
229	Node 550	A-9	Junction	Junction with drop inlet	1:01	937.83	3.14	940.83	3.00	Top Inlet 3	940.83	940.83	6.28	3	3.14	0.67	2249483.99	269140.58
230	Node 14	A-9	Junction	Junction with drop inlet	4:01	932.98	3.14	933.28	0.30	Top Inlet 3	933.28	933.28	6.28	3	3.14	0.67	2249492.01	269159.63
231	Node 260	A-9	Culvert Opening	DS Culvert opening	1:00	932.86	0	932.85	-0.01				6.28	3	3.14	0.67	2249508.22	269135.44
232	Node 312	A-8	Culvert Opening	DS Culvert opening	1:00	929.46	0	929.45	-0.01				6.28	3	3.14	0.67	2249618.67	269091.56
233	Node 145	A-7	Culvert Opening	DS Culvert opening	1:00	930.7	0	930.68	-0.02				6.28	3	3.14	0.67	2249793.14	269096.95
234	Node 71	ST-37	Culvert Opening	DS Culvert opening	1:00	895	0	892.63	-2.37					3		0.67	2250899.78	269094.23
235	Node 176	ST-37	Junction	Junction with drop inlet	2:01	896.55	20	904.57	8.02	Top Inlet 1	904.57	904.57	5	3	2.5	0.67	2251077.61	269100.88
236	Node 669	ST-37	Junction	Junction with drop inlet	1:01	901.12	32	905.8	4.68	Top Inlet 4	905.8	905.8	8	3	4	0.67	2251377.59	269090.00
237	Node 77	ST-37	Junction	Junction with drop inlet	2:01	898.21	32	905.72	7.51	Top Inlet 4	905.72	905.72	8	3	4	0.67	2251328.03	269096.05
238	Node 947	A-9	Junction	Junction with drop	1:02	936.75	3.14	940.05	3.30	Top Inlet 3	940.05	940.05	6.28	3	3.14	0.67	2249457.39	269204.95

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
239	Node 19	A-9	Junction	Junction with drop inlet	4:00	936.82	3.14	939.82	3.00	Top Inlet 3	939.82	939.82	6.28	3	3.14	0.67	2249455.96	269258.41
240	Node 310	A-9	Start	US Junction, with drop inlet	0:01	936.86	3.14	939.66	2.80	Top Inlet 3	939.66	939.66	6.28	3	3.14	0.67	2249479.96	269259.38
241	Node 527	A-9	Start	US Junction, with drop inlet	0:01	934.08	3.14	937.08	3.00	Top Inlet 3	937.08	937.08	6.28	3	3.14	0.67	2249508.55	269193.30
242	Node 861	A-9	Start	US Junction, with drop inlet	0:01	936.83	3.14	939.83	3.00	Top Inlet 3	939.83	939.83	6.28	3	3.14	0.67	2249486.13	269281.62
243	Node 657	A-9	Start	US Junction, with drop inlet	0:01	936.87	3.14	939.38	2.51	Top Inlet 3	939.38	939.38	6.28	3	3.14	0.67	2249494.94	269229.05
244	Node 827	A-7	Junction	Junction with drop inlet	1:01	946.76	3.14	949.76	3.00	Top Inlet 3	949.76	949.76	6.28	3	3.14	0.67	2249833.62	269261.56
245	Node 568	A-5	Start	US Junction, with drop inlet	0:01	945.26	3.14	947.56	2.30	Top Inlet 3	947.56	947.56	6.28	3	3.14	0.67	2250185.65	269286.64
246	Node 712	A-5	Junction	Junction with drop inlet	2:01	937.44	3.14	940.44	3.00	Top Inlet 3	940.44	940.44	6.28	3	3.14	0.67	2250159.88	269188.69
247	Node 280	A-5	Start	US Junction, with drop inlet	0:01	937.78	3.14	940.78	3.00	Top Inlet 3	940.78	940.78	6.28	3	3.14	0.67	2250124.96	269192.37
248	Node 751	A-9	Junction	Junction with drop inlet	1:01	937.46	3.14	939.46	2.00	Top Inlet 3	939.46	939.46	6.28	3	3.14	0.67	2249339.82	269406.30
249	Node 335	A-9	Junction	Junction with drop inlet	2:01	937.15	3.14	940.15	3.00	Top Inlet 3	940.15	940.15	6.28	3	3.14	0.67	2249438.19	269305.07
250	Node 729	A-9	Junction	Junction with drop	1:01	937.25	3.14	940.25	3.00	Top Inlet 3	940.25	940.25	6.28	3	3.14	0.67	2249453.71	269312.90

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
251	Node 1077	A-8	Junction	Junction with drop inlet	1:01	944.83	3.14	947.83	3.00	Top Inlet 3	947.83	947.83	6.28	3	3.14	0.67	2249611.15	269410.83
252	Node 549	A-7	Start	US Junction, with drop inlet	0:01	947.91	3.14	950.51	2.60	Top Inlet 3	950.51	950.51	6.28	3	3.14	0.67	2249834.36	269313.50
253	Node 139	A-5	Junction	Junction with drop inlet	3:01	945.2	3.14	948.2	3.00	Top Inlet 3	948.2	948.2	6.28	3	3.14	0.67	2250157.00	269295.53
254	Node 1078	A-5	Start	US Junction, with drop inlet	0:01	948.06	3.14	951.06	3.00	Top Inlet 3	951.06	951.06	6.28	3	3.14	0.67	2250166.09	269342.88
255	Node 977	A-5	Junction	Junction with drop inlet	1:01	951.81	3.14	954.81	3.00	Top Inlet 3	954.81	954.81	6.28	3	3.14	0.67	2250151.76	269365.42
256	Node 565	A-5	Start	US Junction, with drop inlet	0:01	951.92	3.14	954.92	3.00	Top Inlet 3	954.92	954.92	6.28	3	3.14	0.67	2250120.39	269363.42
257	Node 707	A-10	Start	US Junction, with drop inlet	0:01	952.53	3.14	955.43	2.90	Top Inlet 3	955.43	955.43	6.28	3	3.14	0.67	2250579.43	269401.45
258	Node 34	A-10	Junction	Junction with drop inlet	1:01	952.42	3.14	955.42	3.00	Top Inlet 3	955.42	955.42	6.28	3	3.14	0.67	2250597.73	269388.38
259	Node 939	A-10	Culvert Opening	DS Culvert opening	1:00	943.73	0	943.72	-0.01				6.28	3	3.14	0.67	2250721.94	269299.23
260	Node 45	ST-39	Start	US Junction, with drop inlet	0:01	905.81	16	909.34	3.53	Top Inlet 4	909.34	909.34	8	3	4	0.67	2251196.31	269377.25
261	Node 246	ST-37	Junction	Junction with drop inlet	2:01	904.63	32	908.38	3.75	Top Inlet 4	908.38	908.38	8	3	4	0.67	2251408.32	269336.42
262	Node 1014	ST-37	Junction	Junction with drop inlet	1:01	905.19	20	909.53	4.34	Top Inlet 1	909.53	909.53	5	3	2.5	0.67	2251444.55	269399.64

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
263	Node 86	ST-37	Start	US Junction, with drop inlet	0:01	907.43	20	912.15	4.72	Top Inlet 1	912.15	912.15	5	3	2.5	0.67	2251450.93	269348.15
264	Node 595	ST-37	Junction	Junction with drop inlet	1:01	910.97	20	914.14	3.17	Top Inlet 1	914.14	914.14	5	3	2.5	0.67	2251729.84	269404.20
265	Node 181	ST-37	Junction	Junction with drop inlet	1:01	911.01	20	914.41	3.40	Top Inlet 1	914.41	914.41	5	3	2.5	0.67	2251777.68	269390.92
266	Node 59	ST-39	Culvert Opening	DS Culvert opening	1:00	905	0	893.23	-11.77								2251070.95	269426.36
267	Node 66	ST-37	Start	US Junction, with drop inlet	0:01	911.1	20	913.25	2.15	Top Inlet 1	913.25	913.25	5	3	2.5	0.67	2251778.95	269465.01
268	Node 802	A-9	Start	US Junction, with drop inlet	0:01	938.87	3.14	941.87	3.00	Top Inlet 3	941.87	941.87	6.28	3	3.14	0.67	2249302.02	269608.30
269	Node 401	A-9	Junction	Junction with drop inlet	1:01	938.55	3.14	941.55	3.00	Top Inlet 3	941.55	941.55	6.28	3	3.14	0.67	2249307.36	269572.32
270	Node 191	A-9	Junction	Junction with drop inlet	1:01	953.15	3.14	956.15	3.00	Top Inlet 3	956.15	956.15	6.28	3	3.14	0.67	2249440.79	269639.31
271	Node 502	A-8	Junction	Junction with drop inlet	1:01	955.44	3.14	958.44	3.00	Top Inlet 3	958.44	958.44	6.28	3	3.14	0.67	2249711.11	269627.58
272	Node 973	A-8	Junction	Junction with drop inlet	1:01	960.56	3.14	963.56	3.00	Top Inlet 3	963.56	963.56	6.28	3	3.14	0.67	2249713.52	269772.53
273	Node 512	A-11	Culvert Opening	DS Culvert opening	1:00	968.33	0	968.32	-0.01				6.28	3	3.14	0.67	2250771.19	269772.21
274	Node 494	ST-43	Junction	Junction with drop inlet	1:01	903.84	20	906.91	3.07	Top Inlet 1	906.91	906.91	5	3	2.5	0.67	2251444.97	269688.92
275	Node 914	ST-43	Start	US Junction, with drop	0:01	904.97	20	907.6	2.63	Top Inlet 1	907.6	907.6	5	3	2.5	0.67	2251493.77	269675.06

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				inlet														
276	Node 781	ST-45A	Junction	Junction with drop inlet	1:01	904.6	6	907.46	2.86	Top Inlet 4	907.46	907.46	8	3	4	0.67	2251512.48	269678.43
277	Node 18	ST-45A	Culvert Opening	DS Culvert opening	1:00	902.35	0	900.58	-1.77					3		0.67	2251547.95	269781.79
278	Node 7	ST-45B	Start	US Junction, with drop inlet	0:01	905.47	25	907.57	2.10	Top Inlet 4	907.57	907.57	8	3	4	0.67	2251634.69	269695.10
279	Node 8	ST-45B	Culvert Opening	DS Culvert opening	1:00	902.64	0	896.6	-6.04								2251645.49	269780.54
280	Node 76	ST-45C	Start	US Junction, with drop inlet	0:01	905.21	25	907.73	2.52	Top Inlet 4	907.73	907.73	8	3	4	0.67	2251719.87	269758.07
281	Node 89	ST-45A	Start	US Junction, w/o drop inlet	0:01	904.9	0	907.33	2.43					3		0.67	2251743.98	269673.31
282	Node 610	A-9	Junction	Junction with drop inlet	2:01	964.14	3.14	967.14	3.00	Top Inlet 3	967.14	967.14	6.28	3	3.14	0.67	2249432.42	269846.72
283	Node 946	A-9	Start	US Junction, with drop inlet	0:01	964.27	3.14	967.27	3.00	Top Inlet 3	967.27	967.27	6.28	3	3.14	0.67	2249463.12	269848.43
284	Node 536	A-8	Start	US Junction, with drop inlet	0:01	960.62	3.14	963.62	3.00	Top Inlet 3	963.62	963.62	6.28	3	3.14	0.67	2249711.15	269801.40
285	Node 148	A-11	Start	US Junction, with drop inlet	0:01	968.4	3.14	972.01	3.61	Top Inlet 3	972.01	972.01	6.28	3	3.14	0.67	2250685.16	269810.89
286	Node 780	ST-43	Culvert Opening	DS Culvert opening	1:00	889.96	0	889.88	-0.08								2251454.76	269812.42
287	Node 90	ST-45C	Culvert Opening	DS Culvert	1:00	895.03	0	889.27	-5.76					3		0.67	2251729.49	269803.68

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elev.	Base Area	Terrain Elev	Depth	Top Inlet Type	Top Inlet Elev	Drop Inlet Elev	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef	X Coord	Y Coord
				opening														
288	Node 922	A-9	Junction	Junction with drop inlet	2:01	972.97	3.14	975.97	3.00	Top Inlet 3	975.97	975.97	6.28	3	3.14	0.67	2249420.71	270147.46
289	Node 860	A-9	Junction	Junction with drop inlet	1:01	973.08	3.14	975.88	2.80	Top Inlet 3	975.88	975.88	6.28	3	3.14	0.67	2249451.45	270147.85
290	Node 308	A-9	Start	US Junction, with drop inlet	0:01	973.25	3.14	976.25	3.00	Top Inlet 3	976.25	976.25	6.28	3	3.14	0.67	2249478.53	270092.84
291	Node 728	A-9	Junction	Junction with drop inlet	1:01	973.1	3.14	976.1	3.00	Top Inlet 3	976.1	976.1	6.28	3	3.14	0.67	2249476.49	270125.60
292	Node 654	A-9	Junction	Junction with drop inlet	2:01	974.36	3.14	977.36	3.00	Top Inlet 3	977.36	977.36	6.28	3	3.14	0.67	2249415.17	270295.40
293	Node 240	A-9	Start	US Junction, with drop inlet	0:01	974.68	3.14	977.68	3.00	Top Inlet 3	977.68	977.68	6.28	3	3.14	0.67	2249445.06	270316.20
294	Node 949	A-9	Start	US Junction, with drop inlet	0:01	986	3.14	989	3.00	Top Inlet 3	989	989	6.28	3	3.14	0.67	2249406.46	270534.77

M.3.2 EXISTING CONDITIONS HEC RAS CONDUIT ATTRIBUTE TABLE

Table M.3.2. Existing Conditions Conduit Attribute Table

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
0	ST-05d	ST-05	Node 685	Node 1075	Hydraulic	124.94	10	Circular	1.5	1.5	0.024	927.74	926.31	0.01	None
1	ST-05a	ST-05	Node 799	Node 854	Hydraulic	111.76	10	Circular	2	2	0.024	924.91	918.07	0.06	None
2	ST-05c	ST-05	Node 1075	Node 390	Hydraulic	51.51	10	Circular	1.5	1.5	0.024	926.31	925.52	0.02	None
3	ST-05b	ST-05	Node 390	Node 799	Hydraulic	29.50	10	Circular	2	2	0.024	925.52	924.91	0.03	None
4	ST-09c	ST-09	Node 761	Node 237	Hydraulic	233.11	10	Circular	1.5	1.5	0.024	932.11	927.72	0.02	None
5	ST-09e	ST-09	Node 9	Node 389	Hydraulic	171.42	10	Circular	1.5	1.5	0.024	936.46	932.56	0.02	None
6	ST-09b	ST-09	Node 237	Node 893	Hydraulic	134.60	10	Circular	2	2	0.024	927.72	923.38	0.03	None
7	ST-09d	ST-09	Node 389	Node 761	Hydraulic	31.24	10	Circular	1.5	1.5	0.024	932.56	932.11	0.02	None
8	ST-09a	ST-09	Node 893	Node 415	Hydraulic	316.54	10	Circular	3	3	0.024	923.38	914.76	0.03	None
9	ST-11a	ST-11	Node 769	Node 361	Hydraulic	32.01	10	Circular	1.5	1.5	0.015	918.81	918.07	0.03	None
10	ST-11d	ST-11	Node 908	Node 276	Hydraulic	169.57	10	Circular	3	3	0.024	916.15	914.39	0.01	None
11	ST-11b	ST-11	Node 361	Node 648	Hydraulic	54.16	10	Circular	2	2	0.015	918.07	917.55	0.01	None
12	ST-11c	ST-11	Node 648	Node 908	Hydraulic	87.36	10	Circular	3	3	0.024	917.55	916.15	0.02	None
13	ST-16i	ST-16	Node 484	Node 359	Hydraulic	36.79	10	Circular	2	2	0.009	909.94	909.66	0.01	None
14	ST-16d	ST-16	Node 534	Node 129	Hydraulic	262.85	10	Circular	2	2	0.015	918.14	913.46	0.02	None
15	ST-16j	ST-16	Node 620	Node 484	Hydraulic	26.41	10	Circular	1.5	1.5	0.009	910.77	909.94	0.04	None
16	ST-16k	ST-16	Node 100	Node 484	Hydraulic	141.48	10	Circular	2	2	0.009	911.67	909.94	0.01	None
17	ST-16m	ST-16	Node 358	Node 760	Hydraulic	25.70	10	Circular	1.5	1.5	0.009	912.16	911.68	0.02	None
18	ST-16h	ST-16	Node 359	Node 222	Hydraulic	27.54	10	Circular	2	2	0.009	909.66	909.31	0.02	None
19	ST-16n	ST-16	Node 742	Node 760	Hydraulic	43.44	10	Circular	2	2	0.009	911.84	911.68	0.00	None
20	ST-16l	ST-16	Node 760	Node 100	Hydraulic	8.51	10	Circular	2	2	0.009	911.68	911.67	0.00	None
21	ST-16c	ST-16	Node 1081	Node 129	Hydraulic	26.10	10	Circular	1.5	1.5	0.015	913.63	913.46	0.01	None
22	ST-16o	ST-16	Node 1045	Node 742	Hydraulic	25.69	10	Circular	1.5	1.5	0.009	912.22	911.84	0.02	None
23	ST-16b	ST-16	Node 129	Node 222	Hydraulic	101.50	10	Circular	2	2	0.015	913.46	909.31	0.04	None
24	ST-16e	ST-16	Node 963	Node 534	Hydraulic	25.21	10	Circular	1.5	1.5	0.015	918.24	918.14	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
25	ST-16g	ST-16	Node 658	Node 247	Hydraulic	25.37	10	Circular	1.5	1.5	0.015	919.92	919.74	0.01	None
26	ST-16f	ST-16	Node 247	Node 534	Hydraulic	70.71	10	Circular	2	2	0.015	919.74	918.14	0.02	None
27	ST-15Ba	ST-15B	Node 887	Node 754	Hydraulic	34.52	10	Circular	1.25	1.25	0.015	987.22	987.15	0.00	None
28	ST-15Bb	ST-15B	Node 754	Node 596	Hydraulic	134.06	10	Circular	1.25	1.25	0.015	987.15	975.58	0.09	None
29	ST-15c	ST-15	Node 690	Node 303	Hydraulic	34.06	10	Circular	1.25	1.25	0.024	913.94	911.31	0.09	None
30	ST-15o	ST-15	Node 857	Node 680	Hydraulic	44.81	10	Circular	1.25	1.25	0.024	968.26	968.17	0.00	None
31	ST-15g	ST-15	Node 535	Node 545	Hydraulic	106.43	10	Circular	3	3	0.024	922.24	922.42	0.00	None
32	ST-15i	ST-15	Node 101	Node 830	Hydraulic	92.41	10	Circular	2	2	0.024	923.34	921.18	0.02	None
33	ST-15f	ST-15	Node 123	Node 561	Hydraulic	43.45	10	Circular	3	3	0.024	922.09	920.61	0.04	None
34	ST-15l	ST-15	Node 393	Node 795	Hydraulic	180.81	10	Circular	1.5	1.5	0.024	952.21	932.2	0.11	None
35	ST-15k	ST-15	Node 448	Node 795	Hydraulic	33.32	10	Circular	1.25	1.25	0.024	932.3	932.2	0.00	None
36	ST-15f 1	ST-15	Node 545	Node 123	Hydraulic	36.14	10	Circular	3	3	0.024	922.42	922.09	0.01	None
37	ST-15n	ST-15	Node 680	Node 248	Hydraulic	148.28	10	Circular	1.5	1.5	0.024	968.17	964.29	0.03	None
38	ST-15e	ST-15	Node 430	Node 561	Hydraulic	71.21	10	Circular	1.25	1.25	0.024	920.94	920.61	0.00	None
39	ST-15m	ST-15	Node 248	Node 393	Hydraulic	127.27	10	Circular	1.5	1.5	0.024	964.29	952.21	0.10	None
40	ST-15b	ST-15	Node 303	Node 984	Hydraulic	77.88	10	Circular	3.5	3.5	0.024	911.31	910.38	0.01	None
41	ST-15a	ST-15	Node 984	Node 1022	Hydraulic	112.70	10	Circular	3.5	3.5	0.024	910.38	908.68	0.02	None
42	ST-15h	ST-15	Node 830	Node 561	Hydraulic	54.16	10	Circular	2	2	0.024	921.18	920.61	0.01	None
43	ST-15d	ST-15	Node 561	Node 303	Hydraulic	203.41	10	Circular	3.5	3.5	0.024	920.61	911.31	0.05	None
44	ST-15j	ST-15	Node 795	Node 101	Hydraulic	185.58	10	Circular	2	2	0.024	932.2	923.34	0.05	None
45	ST-20s	ST-20	Node 540	Node 75	Hydraulic	107.88	10	Circular	2.5	2.5	0.024	922.11	920.64	0.01	None
46	ST-20q	ST-20	Node 75	Node 1052	Hydraulic	276.73	10	Circular	2.5	2.5	0.024	920.64	919.26	0.01	None
47	ST-20u	ST-20	Node 70	Node 972	Hydraulic	33.02	10	Circular	2	2	0.024	922.4	922.3	0.00	None
48	ST-20t	ST-20	Node 972	Node 540	Hydraulic	100.22	10	Circular	2	2	0.024	922.3	922.11	0.00	None
49	ST-20r	ST-20	Node 1035	Node 75	Hydraulic	158.98	10	Circular	1.25	1.25	0.024	923.6	920.64	0.02	None
50	ST-20m	ST-20	Node 1052	Node 640	Hydraulic	197.99	10	Circular	2.833333	2.833333	0.024	919.26	918.42	0.00	None
51	ST-20o	ST-20	Node 411	Node 371	Hydraulic	16.60	10	Circular	1.25	1.25	0.024	919.91	919.45	0.04	None
52	ST-20l	ST-20	Node 640	Node 766	Hydraulic	148.09	10	Circular	2.833333	2.833333	0.024	918.42	916.41	0.01	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
53	ST-20n	ST-20	Node 371	Node 1052	Hydraulic	40.59	10	Circular	1.5	1.5	0.024	919.45	919.26	0.01	None
54	ST-20g	ST-20	Node 509	Node 903	Hydraulic	259.93	10	Circular	3	3	0.024	911.32	907.57	0.01	None
55	ST-20i	ST-20	Node 206	Node 641	Hydraulic	59.32	10	Circular	1.25	1.25	0.024	912.98	911.42	0.03	None
56	ST-20k	ST-20	Node 354	Node 766	Hydraulic	42.06	10	Circular	1.25	1.25	0.024	916.48	916.41	0.00	None
57	ST-20j	ST-20	Node 766	Node 641	Hydraulic	188.93	10	Circular	3	3	0.024	916.41	911.42	0.03	None
58	ST-20h	ST-20	Node 641	Node 509	Hydraulic	32.09	10	Circular	3	3	0.024	911.42	911.32	0.00	None
59	ST-20e	ST-20	Node 11	Node 333	Hydraulic	56.85	10	Circular	2	2	0.024	907.99	907.89	0.00	None
60	ST-20c	ST-20	Node 767	Node 1025 (1)	Hydraulic	201.41	10	Circular	3.333333	3.333333	0.024	907.07	902.93	0.02	None
61	ST-20b	ST-20	Node 614 (1)	Node 1025 (1)	Hydraulic	45.53	10	Circular	1.5	1.5	0.024	905.64	902.93	0.07	None
62	ST-20f	ST-20	Node 903	Node 767	Hydraulic	158.16	10	Circular	3	3	0.024	907.57	907.07	0.00	None
63	ST-20d	ST-20	Node 333	Node 767	Hydraulic	42.77	10	Circular	1.5	1.5	0.024	907.89	907.07	0.02	None
64	ST-20a	ST-20	Node 1025 (1)	Node 44	Hydraulic	421.40	10	Circular	4	4	0.024	902.93	900.25	0.01	None
65	ST-22 1	ST-22	Node 192	Node 447	Hydraulic	101.73	10	Circular	1.25	1.25	0.024	908.6	898.68	0.10	None
66	ST-23	ST-23	Node 730	Node 41	Hydraulic	32.43	10	Circular	1.25	1.25	0.024	912.06	910.39	0.06	None
67	ST-23 1	ST-23	Node 41	Node 883	Hydraulic	51.74	10	Circular	1.5	1.5	0.024	910.39	907.13	0.07	None
68	ST-26a	ST-26	Node 3	Node 85	Hydraulic	158.74	10	Circular	1.5	1.5	0.024	906.65	900	0.04	None
69	ST-26b	ST-26	Node 104	Node 3	Hydraulic	62.03	10	Circular	1.25	1.25	0.015	907.01	906.65	0.01	None
70	ST-43	ST-43	Node 914	Node 494	Hydraulic	50.73	10	Circular	1.25	1.25	0.024	904.97	903.84	0.02	None
71	A-10b	A-10	Node 707	Node 34	Hydraulic	22.49	10	Circular	1.5	1.5	0.024	952.53	952.42	0.01	None
72	A-10a	A-10	Node 34	Node 939	Hydraulic	152.90	10	Circular	1.5	1.5	0.024	952.42	943.73	0.06	None
73	A-6c	A-6	Node 67	Node 587	Hydraulic	61.04	10	Circular	1.5	1.5	0.024	946.09	944.58	0.03	None
74	A-7b	A-7	Node 549	Node 827	Hydraulic	51.95	10	Circular	1.25	1.25	0.024	947.91	946.76	0.02	None
75	A-6b	A-6	Node 168	Node 587	Hydraulic	31.20	10	Circular	1.5	1.5	0.024	945.33	944.58	0.03	None
76	A-6a	A-6	Node 587	Node 743	Hydraulic	150.91	10	Circular	1.5	1.5	0.024	944.58	932.32	0.08	None
77	A-7a	A-7	Node 827	Node 145	Hydraulic	169.52	10	Circular	1.25	1.25	0.024	946.76	930.7	0.10	None
78	A-2b	A-2	Node 57	Node 1033	Hydraulic	35.72	10	Circular	1.25	1.25	0.024	934.92	934.99	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
79	A-2e	A-2	Node 454	Node 868	Hydraulic	32.58	10	Circular	1.25	1.25	0.024	943.78	943.68	0.00	None
80	A-2d	A-2	Node 868	Node 127	Hydraulic	65.37	10	Circular	1.25	1.25	0.024	943.68	940.11	0.06	None
81	A-2a	A-2	Node 1033	Node 1020	Hydraulic	44.14	10	Circular	1.25	1.25	0.024	934.99	933.28	0.04	None
82	A-2c	A-2	Node 127	Node 1033	Hydraulic	206.30	10	Circular	1.25	1.25	0.024	940.11	934.99	0.03	None
83	A-4a	A-4	Node 704	Node 916	Hydraulic	66.69	10	Circular	1.25	1.25	0.024	925.61	921.47	0.06	None
84	ST-37	ST-37	Node 128	Node 905	Hydraulic	27.09	10	Circular	1.25	1.25	0.015	904.49	904.4	0.00	None
85	ST-37 1	ST-37	Node 1050	Node 1079	Hydraulic	213.90	10	Circular	1.25	1.25	0.015	909.09	903.5	0.03	None
86	ST-37 2	ST-37	Node 884	Node 464	Hydraulic	37.59	10	Circular	1.5	1.5	0.024	905.7	905.65	0.00	None
87	ST-37 3	ST-37	Node 492	Node 618	Hydraulic	106.57	10	Circular	2	2	0.015	906.75	906.11	0.01	None
88	ST-37 4	ST-37	Node 905	Node 385	Hydraulic	57.76	10	Circular	3	3	0.015	904.4	904.33	0.00	None
89	ST-37 5	ST-37	Node 370	Node 902	Hydraulic	119.77	10	Circular	1.25	1.25	0.015	908.94	908.28	0.01	None
90	ST-37 6	ST-37	Node 83	Node 491	Hydraulic	77.74	10	Circular	2	2	0.015	905.83	905.68	0.00	None
91	ST-37 7	ST-37	Node 275	Node 188	Hydraulic	47.08	10	Circular	1.25	1.25	0.015	904.71	904.65	0.00	None
92	ST-37 8	ST-37	Node 1038	Node 385	Hydraulic	316.18	10	Circular	2	2	0.024	905.6	904.33	0.00	None
93	ST-37 9	ST-37	Node 777	Node 902	Hydraulic	42.40	10	Circular	1	1	0.015	908.37	908.28	0.00	None
94	ST-37 10	ST-37	Node 350	Node 905	Hydraulic	64.63	10	Circular	3	3	0.015	904.44	904.4	0.00	None
95	ST-37 11	ST-37	Node 615	Node 464	Hydraulic	33.18	10	Circular	1.25	1.25	0.024	905.75	905.65	0.00	None
96	ST-37 12	ST-37	Node 1054	Node 369	Hydraulic	30.32	10	Circular	1.25	1.25	0.015	908.54	908.47	0.00	None
97	ST-37 14	ST-37	Node 618	Node 83	Hydraulic	53.00	10	Circular	2	2	0.015	906.11	905.83	0.01	None
98	ST-37 15	ST-37	Node 385	Node 828	Hydraulic	222.37	10	Circular	3.5	3.5	0.024	904.33	903.47	0.00	None
99	ST-37 16	ST-37	Node 369	Node 233	Hydraulic	91.83	10	Circular	1.25	1.25	0.015	908.47	907.11	0.02	None
100	ST-37 17	ST-37	Node 812	Node 905	Hydraulic	61.08	10	Circular	2.5	2.5	0.024	904.5	904.4	0.00	None
101	ST-37 18	ST-37	Node 491	Node 915	Hydraulic	28.72	10	Circular	3	3	0.015	905.68	905.65	0.00	None
102	ST-37 19	ST-37	Node 246	Node 669	Hydraulic	248.33	10	Circular	1.5	1.5	0.015	904.63	901.12	0.01	None
103	ST-37 20	ST-37	Node 233	Node 491	Hydraulic	93.13	10	Circular	1.25	1.25	0.015	907.11	905.68	0.02	None
104	ST-37 21	ST-37	Node 495	Node 884	Hydraulic	35.57	10	Circular	1.5	1.5	0.024	905.75	905.7	0.00	None
105	ST-37 22	ST-37	Node 989	Node 275	Hydraulic	58.27	10	Circular	1.25	1.25	0.015	904.82	904.71	0.00	None
106	ST-37 23	ST-37	Node 464	Node 1038	Hydraulic	43.21	10	Circular	1.75	1.75	0.024	905.65	905.6	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
107	ST-37 24	ST-37	Node 758	Node 83	Hydraulic	36.58	10	Circular	1.25	1.25	0.015	905.87	905.83	0.00	None
108	ST-37 25	ST-37	Node 915	Node 188	Hydraulic	35.26	10	Circular	3	3	0.015	905.65	904.65	0.03	None
109	ST-37 26	ST-37	Node 188	Node 350	Hydraulic	202.30	10	Circular	3	3	0.015	904.65	904.44	0.00	None
110	ST-37 27	ST-37	Node 828	Node 783	Hydraulic	15.15	10	Circular	3.5	3.5	0.024	903.47	903.43	0.00	None
111	ST-37 28	ST-37	Node 899	Node 812	Hydraulic	99.09	10	Circular	1.25	1.25	0.024	904.55	904.5	0.00	None
112	ST-37 29	ST-37	Node 1079	Node 828	Hydraulic	56.45	10	Circular	1.5	1.5	0.024	903.5	903.47	0.00	None
113	ST-37 30	ST-37	Node 902	Node 492	Hydraulic	168.90	10	Circular	1.25	1.25	0.015	908.28	906.75	0.01	None
114	ST-37 31	ST-37	Node 669	Node 77	Hydraulic	49.92	10	Circular	2	2	0.024	901.12	898.21	0.07	None
115	ST-37 32	ST-37	Node 783	Node 77	Hydraulic	173.74	10	Circular	3.5	3.5	0.024	903.43	898.21	0.03	None
116	ST-37 35	ST-37	Node 864	Node 177	Hydraulic	16.79	10	Circular	1.25	1.25	0.015	903.83	902.53	0.11	None
117	ST-37 37	ST-37	Node 42	Node 721	Hydraulic	127.02	10	Circular	1.5	1.5	0.015	898.92	897.75	0.01	None
118	ST-37 38	ST-37	Node 721	Node 176	Hydraulic	67.07	10	Circular	1.5	1.5	0.015	897.75	896.55	0.02	None
119	ST-37 39	ST-37	Node 634	Node 177	Hydraulic	144.70	10	Circular	2	2	0.015	904.38	902.53	0.01	None
120	ST-37 40	ST-37	Node 1051	Node 634	Hydraulic	36.21	10	Circular	1.25	1.25	0.015	904.43	904.38	0.00	None
121	ST-37 41	ST-37	Node 86	Node 246	Hydraulic	44.20	10	Circular	1.25	1.25	0.015	907.43	904.63	0.07	None
122	ST-37 42	ST-37	Node 1014	Node 246	Hydraulic	72.87	10	Circular	1.5	1.5	0.015	905.19	904.63	0.01	None
123	ST-37 43	ST-37	Node 66	Node 181	Hydraulic	74.10	10	Circular	1.25	1.25	0.015	911.1	911.01	0.00	None
124	ST-37 45	ST-37	Node 181	Node 595	Hydraulic	49.64	10	Circular	1.5	1.5	0.015	911.01	910.97	0.00	None
125	ST-37 46	ST-37	Node 595	Node 1014	Hydraulic	285.33	10	Circular	1.5	1.5	0.015	910.97	905.19	0.02	None
126	A-1i	A-12	Node 403	Node 1084	Hydraulic	231.45	10	Circular	1.5	1.5	0.024	969.9	962.04	0.03	None
127	A-1l	A-12	Node 170	Node 249	Hydraulic	54.51	10	Circular	1.25	1.25	0.024	990.35	990.06	0.01	None
128	A-1j	A-12	Node 862	Node 403	Hydraulic	63.24	10	Circular	1.25	1.25	0.024	972.06	969.9	0.04	None
129	ST-27	ST-27	Node 792	Node 555	Hydraulic	54.20	10	Circular	1.25	1.25	0.024	935.64	935.54	0.00	None
130	ST-27 1	ST-27	Node 122	Node 555	Hydraulic	319.83	10	Circular	1.5	1.5	0.024	949.96	935.54	0.05	None
131	A-1g	A-12	Node 1084	Node 318	Hydraulic	39.36	10	Circular	1.25	1.25	0.024	962.04	951.61	0.29	None
132	ST-27 2	ST-27	Node 40	Node 122	Hydraulic	53.64	10	Circular	1.25	1.25	0.024	949.99	949.96	0.00	None
133	A-1k	A-12	Node 249	Node 403	Hydraulic	332.43	10	Circular	1.5	1.5	0.024	990.06	969.9	0.06	None
134	ST-27 3	ST-27	Node 555	Node 978	Hydraulic	306.58	10	Circular	1.5	1.5	0.024	935.54	915.12	0.07	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
135	A-1h	A-12	Node 451	Node 1084	Hydraulic	54.74	10	Circular	1.25	1.25	0.024	962.19	962.04	0.00	None
136	ST-27 4	ST-27	Node 978	Node 1021	Hydraulic	76.47	10	Circular	1.5	1.5	0.024	915.12	895.5	0.27	None
137	A-8c	A-8	Node 973	Node 502	Hydraulic	144.97	10	Circular	1.5	1.5	0.024	960.56	955.44	0.04	None
138	A-8b	A-8	Node 502	Node 1077	Hydraulic	238.69	10	Circular	2	2	0.024	955.44	944.83	0.04	None
139	A-8a	A-8	Node 1077	Node 312	Hydraulic	319.35	10	Circular	2	2	0.024	944.83	929.46	0.05	None
140	A-8d	A-8	Node 536	Node 973	Hydraulic	28.96	10	Circular	1.5	1.5	0.024	960.62	960.56	0.00	None
141	A-11a	A-11	Node 148	Node 512	Hydraulic	94.32	10	Circular	1.25	1.25	0.024	968.4	968.33	0.00	None
142	A-3h	A-3	Node 17	Node 700	Hydraulic	79.18	10	Circular	1.25	1.25	0.024	961.35	957.99	0.04	None
143	A-3p	A-3	Node 1073	Node 290	Hydraulic	34.13	10	Circular	1.25	1.25	0.024	977.42	976.16	0.04	None
144	A-3d	A-3	Node 590	Node 997	Hydraulic	45.31	10	Circular	1.25	1.25	0.024	961.58	959.2	0.05	None
145	A-3c	A-3	Node 997	Node 944	Hydraulic	42.93	10	Circular	1.25	1.25	0.024	959.2	957.94	0.03	None
146	A-3m	A-3	Node 979	Node 556	Hydraulic	86.70	10	Circular	1.25	1.25	0.024	975.53	972.03	0.04	None
147	A-3j	A-3	Node 429	Node 808	Hydraulic	65.46	10	Circular	1.25	1.25	0.024	967.61	967.52	0.00	None
148	A-3f	A-3	Node 700	Node 944	Hydraulic	17.98	10	Circular	1.25	1.25	0.024	957.99	957.94	0.00	None
149	A-3i	A-3	Node 808	Node 17	Hydraulic	123.34	10	Circular	1.25	1.25	0.024	967.52	961.35	0.05	None
150	A-3n	A-3	Node 674	Node 979	Hydraulic	40.59	10	Circular	1.25	1.25	0.024	977.14	975.53	0.04	None
151	A-3b	A-3	Node 944	Node 329	Hydraulic	109.67	10	Circular	3	3	0.024	957.94	953.42	0.04	None
152	A-3l	A-3	Node 556	Node 808	Hydraulic	52.26	10	Circular	1.25	1.25	0.024	972.03	967.52	0.09	None
153	A-3g	A-3	Node 301	Node 700	Hydraulic	31.10	10	Circular	1.25	1.25	0.024	958.06	957.99	0.00	None
154	A-3k	A-3	Node 20	Node 429	Hydraulic	31.87	10	Circular	1.25	1.25	0.024	967.7	967.61	0.00	None
155	A-3o	A-3	Node 290	Node 979	Hydraulic	34.32	10	Circular	1.25	1.25	0.024	976.16	975.53	0.02	None
156	A-3a	A-3	Node 329	Node 811	Hydraulic	96.18	10	Circular	3	3	0.024	953.42	953.13	0.00	None
157	A-9	A-9	Node 335	Node 19	Hydraulic	49.93	10	Circular	2	2	0.024	937.15	936.82	0.01	None
158	A-9 1	A-9	Node 401	Node 751	Hydraulic	169.16	10	Circular	2	2	0.024	938.55	937.46	0.01	None
159	A-9 2	A-9	Node 802	Node 401	Hydraulic	36.38	10	Circular	2	2	0.024	938.87	938.55	0.01	None
160	A-9 3	A-9	Node 310	Node 19	Hydraulic	24.02	10	Circular	2	2	0.024	936.86	936.82	0.00	None
161	A-9 4	A-9	Node 527	Node 14	Hydraulic	37.51	10	Circular	2	2	0.024	934.08	932.98	0.03	None
162	A-9 5	A-9	Node 657	Node 947	Hydraulic	44.62	10	Circular	2	2	0.024	936.87	936.75	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
163	A-9 6	A-9	Node 861	Node 19	Hydraulic	38.06	10	Circular	2	2	0.024	936.83	936.82	0.00	None
164	A-9 7	A-9	Node 579	Node 14	Hydraulic	17.04	10	Circular	2	2	0.024	938.09	932.98	0.34	None
165	A-9 8	A-9	Node 550	Node 14	Hydraulic	20.67	10	Circular	2	2	0.024	937.83	932.98	0.26	None
166	A-9 9	A-9	Node 14	Node 260	Hydraulic	29.12	10	Circular	2	2	0.024	932.98	932.86	0.00	None
167	A-9 10	A-9	Node 947	Node 14	Hydraulic	57.03	10	Circular	2	2	0.024	936.75	932.98	0.07	None
168	A-9 11	A-9	Node 610	Node 191	Hydraulic	207.58	10	Circular	1.5	1.5	0.024	964.14	953.15	0.05	None
169	A-9 12	A-9	Node 654	Node 922	Hydraulic	148.04	10	Circular	1.25	1.25	0.024	974.36	972.97	0.01	None
170	A-9 13	A-9	Node 425	Node 442	Hydraulic	39.75	10	Circular	1.25	1.25	0.024	958.44	953.95	0.12	None
171	A-9 14	A-9	Node 831	Node 550	Hydraulic	106.42	10	Circular	1.5	1.5	0.024	947.29	937.83	0.09	None
172	A-9 15	A-9	Node 860	Node 922	Hydraulic	30.74	10	Circular	1.5	1.5	0.024	973.08	972.97	0.00	None
173	A-9 16	A-9	Node 751	Node 335	Hydraulic	141.15	10	Circular	2	2	0.024	937.46	937.15	0.00	None
174	A-9 17	A-9	Node 406	Node 442	Hydraulic	76.12	10	Circular	1.25	1.25	0.024	961.92	953.95	0.11	None
175	A-9 18	A-9	Node 922	Node 610	Hydraulic	300.97	10	Circular	1.5	1.5	0.024	972.97	964.14	0.03	None
176	A-9 19	A-9	Node 949	Node 654	Hydraulic	239.53	10	Circular	1.25	1.25	0.024	986	974.36	0.05	None
177	A-9 20	A-9	Node 442	Node 831	Hydraulic	56.71	10	Circular	1.25	1.25	0.024	953.95	947.29	0.12	None
178	A-9 21	A-9	Node 308	Node 728	Hydraulic	32.82	10	Circular	1.25	1.25	0.024	973.25	973.1	0.00	None
179	A-9 22	A-9	Node 729	Node 335	Hydraulic	17.39	10	Circular	1.5	1.5	0.024	937.25	937.15	0.01	None
180	A-9 23	A-9	Node 191	Node 729	Hydraulic	326.66	10	Circular	1.5	1.5	0.024	953.15	937.25	0.05	None
181	A-9 24	A-9	Node 240	Node 654	Hydraulic	36.41	10	Circular	1.25	1.25	0.024	974.68	974.36	0.01	None
182	A-9 25	A-9	Node 947	Node 19	Hydraulic	53.48	10	Circular	1.25	1.25	0.024	936.75	936.82	0.00	None
183	A-9 26	A-9	Node 265	Node 425	Hydraulic	176.99	10	Circular	1.25	1.25	0.024	969.72	958.44	0.06	None
184	A-9 27	A-9	Node 728	Node 860	Hydraulic	33.50	10	Circular	1.5	1.5	0.024	973.1	973.08	0.00	None
185	A-9 28	A-9	Node 946	Node 610	Hydraulic	30.75	10	Circular	1.25	1.25	0.024	964.27	964.14	0.00	None
186	A-9 29	A-9	Node 153	Node 831	Hydraulic	30.88	10	Circular	1.25	1.25	0.024	947.35	947.29	0.00	None
187	A-5 1	A-5	Node 712	Node 6	Hydraulic	160.66	10	Circular	3	3	0.024	937.44	923.67	0.09	None
188	A-5 2	A-5	Node 566	Node 61	Hydraulic	58.00	10	Circular	3	3	0.024	920.09	918.55	0.03	None
189	A-5 3	A-5	Node 139	Node 712	Hydraulic	106.88	10	Circular	3	3	0.024	945.2	937.44	0.07	None
190	A-5 5	A-5	Node 565	Node 977	Hydraulic	31.44	10	Circular	1.25	1.25	0.024	951.92	951.81	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
191	A-5 6	A-5	Node 6	Node 566	Hydraulic	102.47	10	Circular	3	3	0.024	923.67	920.09	0.04	None
192	A-5 7	A-5	Node 977	Node 139	Hydraulic	70.09	10	Circular	1.25	1.25	0.024	951.81	945.2	0.10	None
193	A-5 9	A-5	Node 1078	Node 139	Hydraulic	48.22	10	Circular	2	2	0.024	948.06	945.2	0.06	None
194	A-5 11	A-5	Node 280	Node 712	Hydraulic	35.12	10	Circular	1.25	1.25	0.024	937.78	937.44	0.01	None
195	A-5 12	A-5	Node 568	Node 139	Hydraulic	30.00	10	Circular	1.25	1.25	0.024	945.26	945.2	0.00	None
196	A-5 13	A-3	Node 10	Node 944	Hydraulic	60.95	10	Circular	1.25	1.25	0.024	956.84	957.94	-0.02	None
197	ST-19	ST-19	Node 49	Node 48	Hydraulic	36.92	10	Circular	1.25	1.25	0.015	914.7	911.65	0.10	None
198	ST-19 1	ST-19	Node 48	Node 47	Hydraulic	29.18	10	Circular	1.25	1.25	0.015	911.65	911.15	0.02	None
199	ST-19 2	ST-19	Node 47	Node 43	Hydraulic	176.04	10	Circular	1.25	1.25	0.015	911.15	908.63	0.01	None
200	ST-19 3	ST-19	Node 43	Node 46	Hydraulic	98.33	10	Circular	3.16	3.16	0.024	908.63	906.46	0.02	None
201	ST-19 4	ST-19	Node 38	Node 43	Hydraulic	203.72	10	Circular	3.16	3.16	0.024	916.65	908.63	0.04	None
202	ST-19 5	ST-19	Node 12	Node 13	Hydraulic	53.89	10	Circular	2	2	0.024	990.44	988.17	0.05	None
203	ST-19 6	ST-19	Node 13	Node 16	Hydraulic	342.81	10	Circular	2	2	0.024	988.17	971.68	0.05	None
204	ST-19 7	ST-19	Node 15	Node 16	Hydraulic	24.68	10	Circular	2	2	0.024	971.84	971.68	0.01	None
205	ST-19 8	ST-19	Node 16	Node 22	Hydraulic	338.60	10	Circular	2	2	0.024	971.68	963.28	0.03	None
206	ST-19 9	ST-19	Node 21	Node 22	Hydraulic	27.23	10	Circular	2	2	0.024	963.91	963.28	0.03	None
207	ST-19 10	ST-19	Node 22	Node 23	Hydraulic	73.92	10	Circular	2	2	0.024	963.28	959.69	0.05	None
208	ST-19 11	ST-19	Node 23	Node 24	Hydraulic	48.46	10	Circular	2	2	0.024	959.69	954.23	0.13	None
209	ST-19 12	ST-19	Node 24	Node 28	Hydraulic	165.78	10	Circular	2	2	0.024	954.23	942.6	0.07	None
210	ST-19 13	ST-19	Node 25	Node 27	Hydraulic	145.77	10	Circular	2	2	0.024	943.56	935.99	0.05	None
211	ST-19 14	ST-19	Node 26	Node 27	Hydraulic	19.60	10	Circular	2	2	0.024	943.47	935.99	0.51	None
212	ST-19 15	ST-19	Node 27	Node 29	Hydraulic	33.82	10	Circular	2	2	0.024	935.99	934.81	0.04	None
213	ST-19 16	ST-19	Node 28	Node 29	Hydraulic	16.38	10	Circular	2	2	0.024	942.6	934.81	0.69	None
214	ST-19 17	ST-19	Node 29	Node 30	Hydraulic	186.28	10	Circular	3.16	3.16	0.024	934.81	928.26	0.04	None
215	ST-19 18	ST-19	Node 31	Node 32	Hydraulic	91.97	10	Circular	2	2	0.015	947.15	941.56	0.06	None
216	ST-19 19	ST-19	Node 32	Node 33	Hydraulic	27.10	10	Circular	2	2	0.015	941.56	940.89	0.03	None
217	ST-19 20	ST-19	Node 33	Node 35	Hydraulic	196.75	10	Circular	2	2	0.015	940.89	934.96	0.03	None
218	ST-19 21	ST-19	Node 35	Node 36	Hydraulic	75.64	10	Circular	2	2	0.015	934.96	931.44	0.05	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
219	ST-19 22	ST-19	Node 36	Node 39	Hydraulic	128.32	10	Circular	2	2	0.015	931.44	922.54	0.07	None
220	ST-19 23	ST-19	Node 30	Node 37	Hydraulic	220.40	10	Circular	3.16	3.16	0.024	928.26	920.52	0.04	None
221	ST-19 24	ST-19	Node 37	Node 38	Hydraulic	217.95	10	Circular	3.16	3.16	0.024	920.52	916.65	0.02	None
222	ST-19 25	ST-19	Node 39	Node 38	Hydraulic	101.39	10	Circular	2	2	0.015	922.54	916.65	0.06	None
223	ST-02	ST-02	Node 52	Node 1	Hydraulic	121.55	10	Circular	2	2	0.015	932.66	926.34	0.05	None
224	ST-02 1	ST-02	Node 53	Node 52	Hydraulic	39.93	10	Circular	2	2	0.015	933.2	932.66	0.02	None
225	ST-02 2	ST-02	Node 55	Node 52	Hydraulic	104.54	10	Circular	2	2	0.015	935.22	932.66	0.03	None
226	ST-02 3	ST-02	Node 54	Node 55	Hydraulic	39.16	10	Circular	2	2	0.015	936.17	935.22	0.03	None
227	ST-02 4	ST-02	Node 56	Node 54	Hydraulic	370.93	10	Circular	2	2	0.015	951.56	936.17	0.04	None
228	ST-03	ST-03	Node 63	Node 62	Hydraulic	189.88	10	Circular	1.25	1.25	0.015	932.17	929.99	0.01	None
229	ST-03 1	ST-03	Node 62	Node 60	Hydraulic	36.25	10	Circular	1.5	1.5	0.024	929.99	927.94	0.07	None
230	ST-04 1	ST-04	Node 91	Node 68	Hydraulic	113.45	10	Circular	2	2	0.015	923.05	919.03	0.04	None
231	ST-03 3	ST-03	Node 64	Node 62	Hydraulic	54.36	10	Circular	1.25	1.25	0.024	930.73	929.99	0.02	None
232	ST-03 4	ST-03	Node 72	Node 64	Hydraulic	112.32	10	Circular	1.25	1.25	0.015	931.26	930.73	0.00	None
233	ST-10	ST-10	Node 2	Node 4	Hydraulic	65.54	10	Circular	1.5	1.5	0.024	919.34	913.82	0.09	None
234	ST-02 5	ST-02	Node 80	Node 79	Hydraulic	86.35	10	Circular	2	2	0.015	1008.02	1002.93	0.06	None
235	ST-02 6	ST-02	Node 79	Node 78	Hydraulic	203.38	10	Circular	2	2	0.015	1002.93	991.81	0.06	None
236	ST-02 7	ST-02	Node 78	Node 73	Hydraulic	183.21	10	Circular	2	2	0.015	991.81	978.6	0.07	None
237	ST-02 8	ST-02	Node 74	Node 73	Hydraulic	38.96	10	Circular	2	2	0.015	978.7	978.6	0.00	None
238	ST-02 9	ST-02	Node 73	Node 58	Hydraulic	95.96	10	Circular	2	2	0.015	978.6	972.15	0.07	None
239	ST-02 10	ST-02	Node 58	Node 56	Hydraulic	325.78	10	Circular	2	2	0.015	972.15	951.56	0.06	None
240	ST-26	ST-26	Node 85	Node 152	Hydraulic	53.95	10	Circular	2	2	0.015	900	899.11	0.02	None
241	ST-16	ST-16	Node 222	Node 332	Hydraulic	133.57	10	Circular	2	2	0.015	909.31	908.43	0.01	None
242	ST-43 1	ST-43	Node 494	Node 780	Hydraulic	123.89	10	Circular	2	2	0.024	903.84	889.96	0.11	None
243	ST-02 11	ST-02	Node 1	Node 87	Hydraulic	72.82	10	Circular	2	2	0.015	926.34	923.17	0.05	None
244	ST-03 5	ST-03	Node 60	Node 88	Hydraulic	49.62	10	Circular	1.5	1.5	0.024	927.94	924.1	0.08	None
245	ST-39	ST-39	Node 45	Node 59	Hydraulic	134.64	10	Circular	2	2	0.024	905.81	905	0.01	None
246	Conduit 1	ST-04	Node 65	Node 91	Hydraulic	35.24	10	Circular	2	2	0.015	926.92	923.05	0.13	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
247	Conduit 2	ST-04	Node 5	Node 65	Hydraulic	33.61	10	Circular	2	2	0.015	930	926.92	0.11	None
248	Conduit 271	ST-07	Node 707 (1)	Node 334	Hydraulic	97.16	10	Circular	1.5	1.5	0.015	929.87	927.72	0.02	None
249	Conduit 47	ST-07	Node 826	Node 707 (1)	Hydraulic	51.24	10	Circular	1.5	1.5	0.015	930.4	929.87	0.01	None
250	Conduit 818	ST-07	Node 702	Node 707 (1)	Hydraulic	9.98	10	Circular	1.5	1.5	0.015	929.96	929.87	0.02	None
251	ST-37 13	ST-37	Node 177	Node 69	Hydraulic	129.65	10	Circular	2	2	0.015	902.53	894.75	0.06	None
252	ST-37 33	ST-37	Node 77	Node 176	Hydraulic	250.47	10	Circular	4	4	0.024	898.21	896.55	0.01	None
253	ST-37 34	ST-37	Node 176	Node 71	Hydraulic	177.96	10	Circular	4	4	0.024	896.55	895	0.01	None
254	ST-45A	ST-45A	Node 89	Node 781	Hydraulic	231.56	10	Circular	1.5	1.5	0.015	904.9	904.6	0.00	None
255	ST-45A 1	ST-45A	Node 781	Node 18	Hydraulic	109.27	10	Circular	1.5	1.5	0.015	904.6	902.35	0.02	None
256	ST-45B	ST-45B	Node 7	Node 8	Hydraulic	86.11	10	Circular	1.5	1.5	0.015	905.47	902.64	0.03	None
257	ST-45C	ST-45C	Node 76	Node 90	Hydraulic	46.61	10	Circular	1.25	1.25	0.024	905.21	895.03	0.23	None
258	Conduit 10	ST-10	Node 541	Node 50	Hydraulic	34.33	10	Circular	1.5	1.5	0.024	920.35	918.21	0.07	None

M.3.3 PROPOSED CONDITIONS HEC RAS NODE ATTRIBUTE TABLE

Table M.3.3. Proposed Conditions Node Attribute Table

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
0	Node 702	ST-07	Start	US Junction -with top inlet	0:01	929.96	20	932.41	2.45	Top Inlet 1	932.41			932.54	5	3	2.50	0.67
1	Node 707 (1)	ST-07	Junction	Junction -with top inlet	2:01	929.87	20	932.6	2.73	Top Inlet 2	932.6			932.65	0	3	0.00	0.67
2	Node 334	ST-07	Culvert Opening	DS Culvert opening	1:00	927.72	0	927.78	0.06							3		0.67
3	Node 826	ST-07	Start	US Junction -with top inlet	0:01	930.4	20	932.12	1.72	Top Inlet 1	932.12			932.42	5	3	2.50	0.67
4	Node 87	ST-02	Culvert Opening	DS Culvert opening	1:00	924.3	0	923.39	-0.91									
5	Node 88	ST-03	Culvert Opening	DS Culvert opening	1:00	922.01	0	922.54	0.53									
6	Node 854	ST-05	Culvert Opening	DS Culvert opening	1:00	918.98	0	917.47	-1.51									
7	Node 51	ST-05	Junction	Junction -with top inlet	1:01	919.3	25	927.47	8.17	Top Inlet 3	927.47			927.88	8	3	4.00	0.67
8	Node 72	ST-03	Start	US Junction -with top	0:01	931.26	6	935.7	4.44	Top Inlet 4	935.7			935.71	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
9	Node 1	ST-02	Junction	Junction -with top inlet	1:01	929.52	20	934.25	4.73	Top Inlet 1	934.25			934.31	5	3	2.50	0.67
10	Node 64	ST-03	Junction	Junction -with top inlet	1:01	930.73	20	934.63	3.90	Top Inlet 1	934.63			934.64	5	3	2.50	0.67
11	Node 60	ST-03	Junction	Junction -with top inlet	1:01	924.42	20	931.72	7.30	Top Inlet 1	931.72			931.66	5	3	2.50	0.67
12	Node 62	ST-03	Junction	Junction -with top inlet	2:01	929.99	20	933.71	3.72	Top Inlet 1	933.71			933.7	5	3	2.50	0.67
13	Node 685	ST-05	Start	US Junction -with top inlet	0:01	927.74	32	930.19	2.45	Top Inlet 3	930.19			930.19	8	3	4.00	0.67
14	Node 799	ST-05	Junction	Junction -with top inlet	1:01	924.91	44	927.99	3.08	Top Inlet 5	927.99			928.01	11	3	5.50	0.67
15	Node 1075	ST-05	Junction	Junction -with top inlet	1:01	926.31	32	929.34	3.03	Top Inlet 3	929.34			929.32	8	3	4.00	0.67
16	Node 390	ST-05	Junction	Junction -with top inlet	1:01	925.52	44	928.19	2.67	Top Inlet 5	928.19			928.17	11	3	5.50	0.67
17	Node 52	ST-02	Junction	Junction -with top inlet	2:01	932.66	44	935.63	2.97	Top Inlet 5	935.63			935.66	11	3	5.50	0.67
18	Node 53	ST-02	Start	US Junction -with top	0:01	933.2	44	936.2	3.00	Top Inlet 5	936.2			936.2	11	3	5.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
19	Node 55	ST-02	Junction	Junction -with top inlet	1:01	935.22	20	938.25	3.03	Top Inlet 1	938.25			938.22	5	3	2.50	0.67
20	Node 541	ST-10	Start	US Junction -with top inlet	0:01	919.41	25	923.34	3.93	Top Inlet 3	923.34			923.34	8	3	4.00	0.67
21	Node 70	ST-20	Start	US Junction -with top inlet	0:01	922.4	3.14	924.81	2.41	Top Inlet 4	924.81			924.79	6.28	3	3.14	0.67
22	Node 972	ST-20	Junction	Junction -with top inlet	1:01	922.3	20	925.28	2.98	Top Inlet 1	925.28			925.3	5	3	2.50	0.67
23	Node 54	ST-02	Junction	Junction -with top inlet	1:01	936.17	20	939.12	2.95	Top Inlet 1	939.12			939.17	5	3	2.50	0.67
24	Node 63	ST-03	Start	US Junction -with top inlet	0:01	932.17	6	935.89	3.72	Top Inlet 6	935.89			935.88	10	3	5.00	0.67
25	Node 415	ST-09	Culvert Opening	DS Culvert opening	1:00	914.76	0	913.3	-1.46									
26	Node 81	ST-09	Junction	Junction -with top inlet	1:01	915.64	25	924.56	8.92	Top Inlet 7	924.56			924.78	24	3	20.00	0.67
27	Node 1035	ST-20	Start	US Junction -with top inlet	0:01	923.6	16	926.59	2.99	Top Inlet 3	926.59			926.6	8	3	4.00	0.67
28	Node	ST-20	Junctio	Junction -with	1:01	922.11	44	925.16	3.05	Top	925.16			925.11	11	3	5.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
	540		n	top inlet						Inlet 5								
29	Node 75	ST-20	Junction	Junction -with top inlet	2:01	920.64	20	923.56	2.92	Top Inlet 1	923.56			923.64	5	3	2.50	0.67
30	Node 9	ST-09	Start	US Junction -with top inlet	0:01	936.46	20	939.37	2.91	Top Inlet 1	939.37			939.46	5	3	2.50	0.67
31	Node 761	ST-09	Junction	Junction -with top inlet	1:01	932.11	20	935.06	2.95	Top Inlet 1	935.06			935.11	5	3	2.50	0.67
32	Node 389	ST-09	Junction	Junction -with top inlet	1:01	932.56	20	935.56	3.00	Top Inlet 1	935.56			935.56	5	3	2.50	0.67
33	Node 237	ST-09	Junction	Junction -with top inlet	1:01	927.72	20	930.74	3.02	Top Inlet 1	930.74			930.72	5	3	2.50	0.67
34	Node 893	ST-09	Junction	Junction -with top inlet	1:01	923.38	20	928.55	5.17	Top Inlet 1	928.55			928.51	5	3	2.50	0.67
35	Node 2	ST-10	Junction	Junction -with top inlet	1:01	912.59	25	921.87	9.28	Top Inlet 3	921.87			923.03	8	3	4.00	0.67
36	Node 4	ST-10	Culvert Opening	DS Culvert opening	1:00	912.03	0	911.12	-0.91									
37	Node 411	ST-20	Start	US Junction -with top inlet	0:01	919.91	3.14	922.91	3.00	Top Inlet 4	922.91			922.91	6.28	3	3.14	0.67
38	Node 371	ST-20	Junction	Junction -with top	1:01	919.45	20	922.45	3.00	Top Inlet 1	922.45			922.45	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
39	Node 1052	ST-20	Junction	Junction -with top inlet	2:01	919.26	20	922.27	3.01	Top Inlet 1	922.27			922.26	5	3	2.50	0.67
40	Node 56	ST-02	Junction	Junction -with top inlet	1:01	951.56	20	954.57	3.01	Top Inlet 1	954.57			954.56	5	3	2.50	0.67
41	Node 89	ST-12	Culvert Opening	DS Culvert opening	1:00	910.33	0	909.22	-1.11									0.67
42	Node 5	ST-12	Start	US Junction -with top inlet	0:01	914.37	10	920.78	6.41	Top Inlet 8	920.78			920.78	18	3	18.00	0.67
43	Node 658	ST-16	Start	US Junction -with top inlet	0:01	919.92	10	922.94	3.02	Top Inlet 6	922.94			922.92	10	3	5.00	0.67
44	Node 247	ST-16	Junction	Junction -with top inlet	1:01	919.74	10	922.77	3.03	Top Inlet 6	922.77			922.74	10	3	5.00	0.67
45	Node 963	ST-16	Start	US Junction -with top inlet	0:01	918.24	10	921.26	3.02	Top Inlet 6	921.26			921.24	10	3	5.00	0.67
46	Node 534	ST-16	Junction	Junction -with top inlet	2:01	918.14	10	920.89	2.75	Top Inlet 6	920.89			920.9	10	3	5.00	0.67
47	Node 640	ST-20	Junction	Junction -with top inlet	1:01	918.42	20	921.38	2.96	Top Inlet 1	921.38			921.42	5	3	2.50	0.67
48	Node 58	ST-02	Junction	Junction -with top	1:01	972.15	20	975.12	2.97	Top Inlet 1	975.12			975.15	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
49	Node 1081	ST-16	Start	US Junction -with top inlet	0:01	913.63	10	916.62	2.99	Top Inlet 6	916.62			916.63	10	3	5.00	0.67
50	Node 129	ST-16	Junction	Junction -with top inlet	2:01	913.46	10	916.45	2.99	Top Inlet 6	916.45			916.46	10	3	5.00	0.67
51	Node 354	ST-20	Start	US Junction -with top inlet	0:01	916.48	20	919.41	2.93	Top Inlet 1	919.41			919.48	5	3	2.50	0.67
52	Node 766	ST-20	Junction	Junction -with top inlet	2:01	916.41	20	919.38	2.97	Top Inlet 1	919.38			919.41	5	3	2.50	0.67
53	Node 73	ST-02	Junction	Junction -with top inlet	2:01	978.6	20	981.97	3.37	Top Inlet 1	981.97			982.02	5	3	2.50	0.67
54	Node 74	ST-02	Start	US Junction -with top inlet	0:01	978.7	20	981.78	3.08	Top Inlet 1	981.78			981.7	5	3	2.50	0.67
55	Node 359	ST-16	Junction	Junction -with top inlet	1:01	909.66	20	915.23	5.57	Top Inlet 1	915.23			915.25	5	3	2.50	0.67
56	Node 222	ST-16	Junction	Junction -with top inlet	2:01	911.13	20	915.03	3.90	Top Inlet 1	915.03			915.1	5	3	2.50	0.67
57	Node 484	ST-16	Junction	Junction -with top inlet	2:01	909.94	10	915.19	5.25	Top Inlet 6	915.19			915.19	10	3	5.00	0.67
58	Node 620	ST-16	Start	US Junction -with	0:01	910.77	10	915.68	4.91	Top Inlet 6	915.68			915.63	10	3	5.00	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				top inlet														
59	Node 760	ST-16	Junction	Junction -with top inlet	2:01	911.68	10	914.62	2.94	Top Inlet 6	914.62			914.68	10	3	5.00	0.67
60	Node 742	ST-16	Junction	Junction -with top inlet	1:01	911.84	10	914.84	3.00	Top Inlet 6	914.84			914.84	10	3	5.00	0.67
61	Node 100	ST-16	Junction	Junction -with top inlet	1:01	911.67	10	914.66	2.99	Top Inlet 6	914.66			914.67	10	3	5.00	0.67
62	Node 1045	ST-16	Start	US Junction -with top inlet	0:01	912.22	10	915.22	3.00	Top Inlet 6	915.22			915.22	10	3	5.00	0.67
63	Node 358	ST-16	Start	US Junction -with top inlet	0:01	912.16	10	915.16	3.00	Top Inlet 6	915.16			915.16	10	3	5.00	0.67
64	Node 641	ST-20	Junction	Junction -with top inlet	2:01	911.42	20	913.99	2.57	Top Inlet 1	913.99			914.22	5	3	2.50	0.67
65	Node 509	ST-20	Junction	Junction -with top inlet	1:01	911.32	20	914.28	2.96	Top Inlet 1	914.28			914.32	5	3	2.50	0.67
66	Node 206	ST-20	Start	US Junction -with top inlet	0:01	912.98	20	915.97	2.99	Top Inlet 1	915.97			915.98	5	3	2.50	0.67
67	Node 78	ST-02	Junction	Junction -with top inlet	1:01	991.81	20	994.84	3.03	Top Inlet 1	994.84			994.81	5	3	2.50	0.67
68	Node	ST-20	Junctio	Junction -with	1:01	907.57	20	910.59	3.02	Top	910.59			910.57	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
	903		n	top inlet						Inlet 1								
69	Node 79	ST-02	Junction	Junction -with top inlet	1:01	1002.93	20	1005.9	2.97	Top Inlet 1	1005.9			1005.93	5	3	2.50	0.67
70	Node 11	ST-20	Start	US Junction -with top inlet	0:01	907.99	3.14	910.85	2.86	Top Inlet 4	910.85			910.64	6.28	3	3.14	0.67
71	Node 80	ST-02	Start	US Junction -with top inlet	0:01	1008.02	20	1011.03	3.01	Top Inlet 1	1011.03			1011.02	5	3	2.50	0.67
72	Node 767	ST-20	Junction	Junction -with top inlet	2:01	907.07	20	909.99	2.92	Top Inlet 1	909.99			910.07	5	3	2.50	0.67
73	Node 333	ST-20	Junction	Junction -with top inlet	1:01	907.89	20	910.85	2.96	Top Inlet 1	910.85			910.89	5	3	2.50	0.67
74	Node 1025 (1)	ST-20	Junction	Junction -with top inlet	2:01	902.93	44	909.36	6.43	Top Inlet 5	909.36			909.37	11	3	5.50	0.67
75	Node 614 (1)	ST-20	Start	US Junction -with top inlet	0:01	905.64	44	910.01	4.37	Top Inlet 5	910.01			910.14	11	3	5.50	0.67
76	Node 44	ST-20	Culvert Opening	DS Culvert opening	1:00	898.52	0	898.31	-0.21									
77	Node 84	ST-20	Junction	Junction -with top inlet	1:01	901	25	911.3	10.30	Top Inlet 3	911.3			912.16	8	3	4.00	0.67
78	Node	ST-22	Culvert Openin	DS Culvert	1:00	897.64	0	897.53	-0.11									

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
	447		g	opening														
79	Node 863	ST-22	Junction	Junction -with top inlet	1:01	900	25	910.7	10.70	Top Inlet 3	910.7			910.85	8	3	4.00	0.67
80	Node 192	ST-22	Start	US Junction -with top inlet	0:01	908.6	32	910.72	2.12	Top Inlet 3	910.72			910.76	8	3	4.00	0.67
81	Node 170	A-12	Start	US Junction -with top inlet	0:01	990.35	20	993.33	2.98	Top Inlet 1	993.33			993.35	5	3	2.50	0.67
82	Node 862	A-12	Start	US Junction -with top inlet	0:01	972.06	32	975.05	2.99	Top Inlet 3	975.05			975.06	8	3	4.00	0.67
83	Node 451	A-12	Start	US Junction -with top inlet	0:01	962.19	32	965.18	2.99	Top Inlet 3	965.18			965.19	8	3	4.00	0.67
84	Node 249	A-12	Junction	Junction -with top inlet	1:01	990.06	20	993.1	3.04	Top Inlet 1	993.1			993.06	5	3	2.50	0.67
85	Node 403	A-12	Junction	Junction -with top inlet	2:01	969.9	32	972.91	3.01	Top Inlet 3	972.91			972.9	8	3	4.00	0.67
86	Node 1084	A-12	Junction	Junction -with top inlet	2:01	962.04	32	965.06	3.02	Top Inlet 3	965.06			965.04	8	3	4.00	0.67
87	Node 318	A-12	Culvert Opening	DS Culvert opening	1:00	951.61	0	951.57	-0.04									

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
88	Node 104	ST-26	Start	US Junction -with top inlet	0:01	907.01	32	910	2.99	Top Inlet 3	910			910.01	8	3	4.00	0.67
89	Node 3	ST-26	Junction	Junction -with top inlet	1:01	906.65	32	909.78	3.13	Top Inlet 3	909.78			909.77	8	3	4.00	0.67
90	Node 152	ST-26	Culvert Opening	DS Culvert opening	1:00	897.6	0	897.38	-0.22									
91	Node 85	ST-26	Junction	Junction -with top inlet	1:01	900	20	912.42	12.42	Top Inlet 1	912.42			912.34	5	3	2.50	0.67
92	Node 370	ST-37	Start	US Junction -with top inlet	0:01	908.94	20	911.94	3.00	Top Inlet 1	911.94			911.94	5	3	2.50	0.67
93	Node 590	A-3	Start	US Junction -with top inlet	0:01	961.58	3.14	964.59	3.01	Top Inlet 4	964.59			964.58	6.28	3	3.14	0.67
94	Node 10	A-3	Start	US Junction -with top inlet	0:01	956.84	3.14	960.23	3.39	Top Inlet 4	960.23			959.84	6.28	3	3.14	0.67
95	Node 944	A-3	Junction	Junction -with top inlet	3:01	957.94	3.14	960.97	3.03	Top Inlet 4	960.97			960.94	6.28	3	3.14	0.67
96	Node 997	A-3	Junction	Junction -with top inlet	1:01	959.2	3.14	962.2	3.00	Top Inlet 4	962.2			962.2	6.28	3	3.14	0.67
97	Node 57	A-2	Start	US Junction -with	0:01	934.92	3.14	937.93	3.01	Top Inlet 4	937.93			937.92	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				top inlet														
98	Node 1020	A-2	Culvert Opening	DS Culvert opening	1:00	933.28	0	933.41	0.13						6.28	3	3.14	0.67
99	Node 495	ST-37	Start	US Junction -with top inlet	0:01	905.75	20	908.56	2.81	Top Inlet 1	908.56			908.58	5	3	2.50	0.67
100	Node 902	ST-37	Junction	Junction -with top inlet	2:01	908.28	20	911.29	3.01	Top Inlet 1	911.29			911.28	5	3	2.50	0.67
101	Node 777	ST-37	Start	US Junction -with top inlet	0:01	908.37	20	911.18	2.81	Top Inlet 1	911.18			911.17	5	3	2.50	0.67
102	Node 301	A-3	Start	US Junction -with top inlet	0:01	958.06	3.14	960.86	2.80	Top Inlet 4	960.86			960.86	6.28	3	3.14	0.67
103	Node 700	A-3	Junction	Junction -with top inlet	2:01	957.99	3.14	960.88	2.89	Top Inlet 4	960.88			960.89	6.28	3	3.14	0.67
104	Node 17	A-3	Junction	Junction -with top inlet	1:01	961.35	3.14	964.4	3.05	Top Inlet 4	964.4			964.35	6.28	3	3.14	0.67
105	Node 329	A-3	Junction	Junction -with top inlet	1:01	953.42	3.14	956.43	3.01	Top Inlet 4	956.43			956.42	6.28	3	3.14	0.67
106	Node 811	A-3	Culvert Opening	DS Culvert opening	1:00	953.13	0	952.88	-0.25						6.28	3	3.14	0.67
107	Node	A-2	Junctio	Junction -with	2:01	934.99	3.14	938	3.01	Top	938			937.99	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
	1033		n	top inlet						Inlet 4								
108	Node 1038	ST-37	Junction	Junction -with top inlet	1:01	905.6	20	908.56	2.96	Top Inlet 1	908.56			908.6	5	3	2.50	0.67
109	Node 884	ST-37	Junction	Junction -with top inlet	1:01	905.7	20	908.59	2.89	Top Inlet 2	908.59			908.6	0	3	0.00	0.67
110	Node 615	ST-37	Start	US Junction -with top inlet	0:01	905.75	20	908.28	2.53	Top Inlet 1	908.28			908.3	5	3	2.50	0.67
111	Node 464	ST-37	Junction	Junction -with top inlet	2:01	905.65	20	908.25	2.60	Top Inlet 2	908.25			908.28	0	3	0.00	0.67
112	Node 556	A-3	Junction	Junction -with top inlet	1:01	972.03	3.14	975.06	3.03	Top Inlet 4	975.06			975.03	6.28	3	3.14	0.67
113	Node 20	A-3	Start	US Junction -with top inlet	0:01	967.7	3.14	970.22	2.52	Top Inlet 4	970.22			970.2	6.28	3	3.14	0.67
114	Node 808	A-3	Junction	Junction -with top inlet	2:01	967.52	3.14	972.12	4.60	Top Inlet 4	972.12			972.12	6.28	3	3.14	0.67
115	Node 429	A-3	Junction	Junction -with top inlet	1:01	967.61	3.14	970.06	2.45	Top Inlet 4	970.06			970.11	6.28	3	3.14	0.67
116	Node 868	A-2	Junction	Junction -with top inlet	1:01	943.68	3.14	946.62	2.94	Top Inlet 4	946.62			946.68	6.28	3	3.14	0.67
117	Node 454	A-2	Start	US Junction -with	0:01	943.78	3.14	946.81	3.03	Top Inlet 4	946.81			946.78	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				top inlet														
118	Node 127	A-2	Junction	Junction -with top inlet	1:01	940.11	3.14	943.1	2.99	Top Inlet 4	943.1			943.11	6.28	3	3.14	0.67
119	Node 704	A-4	Start	US Junction -with top inlet	0:01	925.61	3.14	928.62	3.01	Top Inlet 4	928.62			928.61	6.28	3	3.14	0.67
120	Node 899	ST-37	Start	US Junction -with top inlet	0:01	904.55	20	907.25	2.70	Top Inlet 1	907.25			907.25	5	3	2.50	0.67
121	Node 492	ST-37	Junction	Junction -with top inlet	1:01	906.75	20	909.82	3.07	Top Inlet 1	909.82			909.75	5	3	2.50	0.67
122	Node 1073	A-3	Start	US Junction -with top inlet	0:01	977.42	3.14	980.47	3.05	Top Inlet 4	980.47			980.42	6.28	3	3.14	0.67
123	Node 979	A-3	Junction	Junction -with top inlet	2:01	975.53	3.14	978.5	2.97	Top Inlet 4	978.5			978.53	6.28	3	3.14	0.67
124	Node 674	A-3	Start	US Junction -with top inlet	0:01	977.14	3.14	980.16	3.02	Top Inlet 4	980.16			980.14	6.28	3	3.14	0.67
125	Node 290	A-3	Junction	Junction -with top inlet	1:01	976.16	3.14	979.16	3.00	Top Inlet 4	979.16			979.16	6.28	3	3.14	0.67
126	Node 916	A-4	Culvert Opening	DS Culvert opening	1:00	921.47	0	921.6	0.13						6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
127	Node 350	ST-37	Junction	Junction -with top inlet	1:01	904.44	18	907.31	2.87	Top Inlet 3	907.31			907.34	8	3	4.00	0.67
128	Node 128	ST-37	Start	US Junction -with top inlet	0:01	904.49	20	907.16	2.67	Top Inlet 1	907.16			907.19	5	3	2.50	0.67
129	Node 812	ST-37	Junction	Junction -with top inlet	1:01	904.5	20	906.84	2.34	Top Inlet 1	906.84			906.9	5	3	2.50	0.67
130	Node 618	ST-37	Junction	Junction -with top inlet	1:01	906.11	20	909.14	3.03	Top Inlet 1	909.14			909.11	5	3	2.50	0.67
131	Node 758	ST-37	Start	US Junction -with top inlet	0:01	905.87	20	908.57	2.70	Top Inlet 1	908.57			908.57	5	3	2.50	0.67
132	Node 83	ST-37	Junction	Junction -with top inlet	2:01	905.83	20	909.03	3.20	Top Inlet 1	909.03			908.83	5	3	2.50	0.67
133	Node 168	A-6	Start	US Junction -with top inlet	0:01	945.33	3.14	948.43	3.10	Top Inlet 4	948.43			948.33	6.28	3	3.14	0.67
134	Node 864	ST-37	Start	US Junction -with top inlet	0:01	903.83	20	906.84	3.01	Top Inlet 1	906.84			906.83	5	3	2.50	0.67
135	Node 177	ST-37	Junction	Junction -with top inlet	2:01	898.05	20	905.51	7.46	Top Inlet 1	905.51			905.53	5	3	2.50	0.67
136	Node 1051	ST-37	Start	US Junction -with	0:01	904.43	20	907.34	2.91	Top Inlet 1	907.34			907.33	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				top inlet														
137	Node 634	ST-37	Junction	Junction -with top inlet	1:01	904.38	20	907.41	3.03	Top Inlet 1	907.41			907.38	5	3	2.50	0.67
138	Node 385	ST-37	Junction	Junction -with top inlet	2:01	904.33	20	907.32	2.99	Top Inlet 1	907.32			907.33	5	3	2.50	0.67
139	Node 905	ST-37	Junction	Junction -with top inlet	3:01	904.4	20	907.29	2.89	Top Inlet 2	907.29			907.3	0	3	0.00	0.67
140	Node 989	ST-37	Start	US Junction -with top inlet	0:01	904.82	6	907.51	2.69	Top Inlet 9	907.51			907.52	6	3	2.00	0.67
141	Node 275	ST-37	Junction	Junction -with top inlet	1:01	904.71	6	907.75	3.04	Top Inlet 9	907.75			907.71	6	3	2.00	0.67
142	Node 188	ST-37	Junction	Junction -with top inlet	2:01	904.65	20	908.45	3.80	Top Inlet 2	908.45			908.45	0	3	0.00	0.67
143	Node 915	ST-37	Junction	Junction -with top inlet	1:01	905.65	20	908.62	2.97	Top Inlet 1	908.62			908.65	5	3	2.50	0.67
144	Node 491	ST-37	Junction	Junction -with top inlet	2:01	905.68	20	908.69	3.01	Top Inlet 1	908.69			908.68	5	3	2.50	0.67
145	Node 233	ST-37	Junction	Junction -with top inlet	1:01	907.11	20	910.14	3.03	Top Inlet 1	910.14			910.11	5	3	2.50	0.67
146	Node 265	A-9	Start	US Junction -with top	0:01	969.72	3.14	972.72	3.00	Top Inlet 4	972.72			972.72	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
147	Node 587	A-6	Junction	Junction -with top inlet	2:01	944.58	3.14	947.61	3.03	Top Inlet 4	947.61			947.58	6.28	3	3.14	0.67
148	Node 67	A-6	Start	US Junction -with top inlet	0:01	946.09	3.14	949.08	2.99	Top Inlet 4	949.08			949.09	6.28	3	3.14	0.67
149	Node 566	A-5	Junction	Junction -with top inlet	1:01	920.09	3.14	922.57	2.48	Top Inlet 4	922.57			922.57	6.28	3	3.14	0.67
150	Node 61	A-5	Culvert Opening	DS Culvert opening	1:00	918.55	0	918.28	-0.27						6.28	3	3.14	0.67
151	Node 828	ST-37	Junction	Junction -with top inlet	2:01	903.47	20	906.28	2.81	Top Inlet 1	906.28			906.29	5	3	2.50	0.67
152	Node 783	ST-37	Junction	Junction -with top inlet	1:01	903.43	20	906.43	3.00	Top Inlet 1	906.43			906.43	5	3	2.50	0.67
153	Node 1079	ST-37	Junction	Junction -with top inlet	1:01	903.5	32	906.4	2.90	Top Inlet 3	906.4			906.4	8	3	4.00	0.67
154	Node 1054	ST-37	Start	US Junction -with top inlet	0:01	908.54	20	911.64	3.10	Top Inlet 1	911.64			911.54	5	3	2.50	0.67
155	Node 369	ST-37	Junction	Junction -with top inlet	1:01	908.47	20	911.56	3.09	Top Inlet 1	911.56			911.47	5	3	2.50	0.67
156	Node 425	A-9	Junction	Junction -with top	1:01	958.44	3.14	961.41	2.97	Top Inlet 4	961.41			961.44	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
157	Node 743	A-6	Culvert Opening	DS Culvert opening	1:00	932.32	0	932.35	0.03						6.28	3	3.14	0.67
158	Node 6	A-5	Junction	Junction -with top inlet	1:01	923.67	3.14	928.19	4.52	Top Inlet 4	928.19			928.16	6.28	3	3.14	0.67
159	Node 721	ST-37	Junction	Junction -with top inlet	2:01	896.97	20	905.12	8.15	Top Inlet 1	905.12			905.11	5	3	2.50	0.67
160	Node 42	ST-37	Start	US Junction -with top inlet	0:01	898.92	20	907.95	9.03	Top Inlet 1	907.95			907.95	5	3	2.50	0.67
161	Node 1050	ST-37	Start	US Junction -with top inlet	0:01	909.09	20	912.19	3.10	Top Inlet 1	912.19			912.09	5	3	2.50	0.67
162	Node 442	A-9	Junction	Junction -with top inlet	2:01	953.95	3.14	956.9	2.95	Top Inlet 4	956.9			956.95	6.28	3	3.14	0.67
163	Node 406	A-9	Start	US Junction -with top inlet	0:01	961.92	3.14	964.92	3.00	Top Inlet 4	964.92			964.92	6.28	3	3.14	0.67
164	Node 831	A-9	Junction	Junction -with top inlet	2:01	947.29	3.14	950.26	2.97	Top Inlet 4	950.26			950.29	6.28	3	3.14	0.67
165	Node 153	A-9	Start	US Junction -with top inlet	0:01	947.35	3.14	950.38	3.03	Top Inlet 4	950.38			950.35	6.28	3	3.14	0.67
166	Node	A-9	Start	US Junctio	0:01	938.09	3.14	941.11	3.02	Top	941.11			941.09	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
	579			n -with top inlet						Inlet 4								
167	Node 550	A-9	Junction	Junction -with top inlet	1:01	937.83	3.14	940.82	2.99	Top Inlet 4	940.82			940.83	6.28	3	3.14	0.67
168	Node 14	A-9	Junction	Junction -with top inlet	4:01	932.98	3.14	933.28	0.30	Top Inlet 4	933.28			933.28	6.28	3	3.14	0.67
169	Node 260	A-9	Culvert Opening	DS Culvert opening	1:00	932.86	0	932.86	0.00						6.28	3	3.14	0.67
170	Node 312	A-8	Culvert Opening	DS Culvert opening	1:00	929.46	0	928.81	-0.65						6.28	3	3.14	0.67
171	Node 145	A-7	Culvert Opening	DS Culvert opening	1:00	930.7	0	930.31	-0.39						6.28	3	3.14	0.67
172	Node 18	ST-37	Culvert Opening	DS Culvert opening	1:00	895.33	0	895.03	-0.30									
173	Node 669	ST-37	Junction	Junction -with top inlet	1:01	901.12	32	905.78	4.66	Top Inlet 3	905.78			905.8	8	3	4.00	0.67
174	Node 77	ST-37	Junction	Junction -with top inlet	3:01	896.16	32	905.72	9.56	Top Inlet 3	905.72			905.72	8	3	4.00	0.67
175	Node 947	A-9	Junction	Junction -with top inlet	1:02	936.75	3.14	940.06	3.31	Top Inlet 4	940.06			940.05	6.28	3	3.14	0.67
176	Node 19	A-9	Junction	Junction -with top	4:00	936.82	3.14	939.8	2.98	Top Inlet 4	939.8			939.82	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
177	Node 310	A-9	Start	US Junction -with top inlet	0:01	936.86	3.14	939.66	2.80	Top Inlet 4	939.66			939.66	6.28	3	3.14	0.67
178	Node 527	A-9	Start	US Junction -with top inlet	0:01	934.08	3.14	937.06	2.98	Top Inlet 4	937.06			937.08	6.28	3	3.14	0.67
179	Node 861	A-9	Start	US Junction -with top inlet	0:01	936.83	3.14	939.83	3.00	Top Inlet 4	939.83			939.83	6.28	3	3.14	0.67
180	Node 657	A-9	Start	US Junction -with top inlet	0:01	936.87	3.14	939.38	2.51	Top Inlet 4	939.38			939.38	6.28	3	3.14	0.67
181	Node 827	A-7	Junction	Junction -with top inlet	1:01	946.76	3.14	949.75	2.99	Top Inlet 4	949.75			949.76	6.28	3	3.14	0.67
182	Node 568	A-5	Start	US Junction -with top inlet	0:01	945.26	3.14	947.57	2.31	Top Inlet 4	947.57			947.56	6.28	3	3.14	0.67
183	Node 712	A-5	Junction	Junction -with top inlet	2:01	937.44	3.14	940.47	3.03	Top Inlet 4	940.47			940.44	6.28	3	3.14	0.67
184	Node 280	A-5	Start	US Junction -with top inlet	0:01	937.78	3.14	940.81	3.03	Top Inlet 4	940.81			940.78	6.28	3	3.14	0.67
185	Node 139	A-5	Junction	Junction -with top inlet	3:01	945.2	3.14	948.16	2.96	Top Inlet 4	948.16			948.2	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
186	Node 939	A-10	Culvert Opening	DS Culvert opening	1:00	943.73	0	943.46	-0.27									
187	Node 335	A-9	Junction	Junction -with top inlet	2:01	937.15	3.14	940.17	3.02	Top Inlet 4	940.17			940.15	6.28	3	3.14	0.67
188	Node 751	A-9	Junction	Junction -with top inlet	1:01	937.46	3.14	939.18	1.72	Top Inlet 4	939.18			939.46	6.28	3	3.14	0.67
189	Node 729	A-9	Junction	Junction -with top inlet	1:01	937.25	3.14	940.26	3.01	Top Inlet 4	940.26			940.25	6.28	3	3.14	0.67
190	Node 1077	A-8	Junction	Junction -with top inlet	1:01	944.83	3.14	947.81	2.98	Top Inlet 4	947.81			947.83	6.28	3	3.14	0.67
191	Node 549	A-7	Start	US Junction -with top inlet	0:01	947.91	3.14	950.5	2.59	Top Inlet 4	950.5			950.51	6.28	3	3.14	0.67
192	Node 1078	A-5	Start	US Junction -with top inlet	0:01	948.06	3.14	951.03	2.97	Top Inlet 4	951.03			951.06	6.28	3	3.14	0.67
193	Node 977	A-5	Junction	Junction -with top inlet	1:01	951.81	3.14	954.77	2.96	Top Inlet 4	954.77			954.81	6.28	3	3.14	0.67
194	Node 565	A-5	Start	US Junction -with top inlet	0:01	951.92	3.14	954.88	2.96	Top Inlet 4	954.88			954.92	6.28	3	3.14	0.67
195	Node 707	A-10	Start	US Junction -with top	0:01	952.53	3.14	955.41	2.88	Top Inlet 4	955.41			955.43	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
196	Node 34	A-10	Junction	Junction -with top inlet	1:01	952.42	3.14	955.43	3.01	Top Inlet 4	955.43			955.42	6.28	3	3.14	0.67
197	Node 246	ST-37	Junction	Junction -with top inlet	2:01	904.63	32	908.38	3.75	Top Inlet 3	908.38			908.38	8	3	4.00	0.67
198	Node 1014	ST-37	Junction	Junction -with top inlet	1:01	905.19	20	909.67	4.48	Top Inlet 1	909.67			909.53	5	3	2.50	0.67
199	Node 86	ST-37	Start	US Junction -with top inlet	0:01	907.43	20	912.14	4.71	Top Inlet 1	912.14			912.15	5	3	2.50	0.67
200	Node 595	ST-37	Junction	Junction -with top inlet	1:01	910.97	20	914.13	3.16	Top Inlet 1	914.13			914.14	5	3	2.50	0.67
201	Node 181	ST-37	Junction	Junction -with top inlet	1:01	911.01	20	914.42	3.41	Top Inlet 1	914.42			914.41	5	3	2.50	0.67
202	Node 66	ST-37	Start	US Junction -with top inlet	0:01	911.1	20	913.25	2.15	Top Inlet 1	913.25			913.25	5	3	2.50	0.67
203	Node 802	A-9	Start	US Junction -with top inlet	0:01	938.87	3.14	941.87	3.00	Top Inlet 4	941.87			941.87	6.28	3	3.14	0.67
204	Node 401	A-9	Junction	Junction -with top inlet	1:01	938.55	3.14	941.55	3.00	Top Inlet 4	941.55			941.55	6.28	3	3.14	0.67
205	Node 191	A-9	Junction	Junction -with top	1:01	953.15	3.14	956.16	3.01	Top Inlet 4	956.16			956.15	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
206	Node 502	A-8	Junction	Junction -with top inlet	1:01	955.44	3.14	958.31	2.87	Top Inlet 4	958.31			958.44	6.28	3	3.14	0.67
207	Node 973	A-8	Junction	Junction -with top inlet	1:01	960.56	3.14	963.56	3.00	Top Inlet 4	963.56			963.56	6.28	3	3.14	0.67
208	Node 512	A-11	Culvert Opening	DS Culvert opening	1:00	968.33	0	967.77	-0.56						6.28	3	3.14	0.67
209	Node 914	ST-43	Start	US Junction -with top inlet	0:01	903	20	907.59	4.59	Top Inlet 1	907.59			907.6	5	3	2.50	0.67
210	Node 781	ST-45	Start	US Junction -with top inlet	0:01	902.47	6	907.45	4.98	Top Inlet 3	907.45			907.46	8	3	4.00	0.67
211	Node 494	ST-43	Junction	Junction -with top inlet	1:01	895	20	906.91	11.91	Top Inlet 1	906.91			906.91	5	3	2.50	0.67
212	Node 7	ST-45	Junction	Junction -with top inlet	1:01	902.25	25	907.56	5.31	Top Inlet 3	907.56			907.57	8	3	4.00	0.67
213	Node 8	ST-45	Culvert Opening	DS Culvert opening	1:00	890.79	0	888.46	-2.33									
214	Node 50	ST-45	Junction	Junction -with top inlet	2:01	893.77	25	908.37	14.60	Top Inlet 3	908.37			908.36	8	3	4.00	0.67
215	Node 76	ST-45	Start	US Junction -with top	0:01	903.44	25	908.12	4.68	Top Inlet 3	908.12			908.13	8	3	4.00	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
216	Node 610	A-9	Junction	Junction -with top inlet	2:01	964.14	3.14	967.16	3.02	Top Inlet 4	967.16			967.14	6.28	3	3.14	0.67
217	Node 946	A-9	Start	US Junction -with top inlet	0:01	964.27	3.14	967.28	3.01	Top Inlet 4	967.28			967.27	6.28	3	3.14	0.67
218	Node 536	A-8	Start	US Junction -with top inlet	0:01	960.62	3.14	963.56	2.94	Top Inlet 4	963.56			963.62	6.28	3	3.14	0.67
219	Node 148	A-11	Start	US Junction -with top inlet	0:01	968.4	3.14	971.97	3.57	Top Inlet 4	971.97			972.01	6.28	3	3.14	0.67
220	Node 780	ST-43	Culvert Opening	DS Culvert opening	1:00	889.65	0	888.72	-0.93									
221	Node 922	A-9	Junction	Junction -with top inlet	2:01	972.97	3.14	975.97	3.00	Top Inlet 4	975.97			975.97	6.28	3	3.14	0.67
222	Node 860	A-9	Junction	Junction -with top inlet	1:01	973.08	3.14	975.85	2.77	Top Inlet 4	975.85			975.88	6.28	3	3.14	0.67
223	Node 308	A-9	Start	US Junction -with top inlet	0:01	973.25	3.14	976.28	3.03	Top Inlet 4	976.28			976.25	6.28	3	3.14	0.67
224	Node 728	A-9	Junction	Junction -with top inlet	1:01	973.1	3.14	976.06	2.96	Top Inlet 4	976.06			976.1	6.28	3	3.14	0.67
225	Node	A-9	Junctio	Junction -with	2:01	974.36	3.14	977.38	3.02	Top	977.38			977.36	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
	654		n	top inlet						Inlet 4								
226	Node 240	A-9	Start	US Junction -with top inlet	0:01	974.68	3.14	977.68	3.00	Top Inlet 4	977.68			977.68	6.28	3	3.14	0.67
227	Node 949	A-9	Start	US Junction -with top inlet	0:01	986	3.14	989.03	3.03	Top Inlet 4	989.03			989	6.28	3	3.14	0.67
228	Node 769	ST-11	Start	US Junction -with top inlet	0:01	918.81	20	920.8	1.99	Top Inlet 1	920.8			920.81	5	3	2.50	0.67
229	Node 361	ST-11	Junction	Junction -with top inlet	1:01	918.07	20	920.84	2.77	Top Inlet 1	920.84			920.87	5	3	2.50	0.67
230	Node 908	ST-11	Junction	Junction -with top inlet	1:01	916.15	25	921.99	5.84	Top Inlet 3	921.99			922.02	8	3	4.00	0.67
231	Node 648	ST-11	Junction	Junction -with top inlet	1:01	917.55	25	920.79	3.24	Top Inlet 3	920.79			920.88	8	3	4.00	0.67
232	Node 276	ST-11	Culvert Opening	DS Culvert opening	1:00	914.39	0	909.59	-4.80									
233	Node 795	ST-15	Junction	Junction -with top inlet	2:01	932.2	3.14	935.31	3.11	Top Inlet 4	935.31			935.2	6.28	3	3.14	0.67
234	Node 448	ST-15	Start	US Junction -with top inlet	0:01	932.3	3.14	935.12	2.82	Top Inlet 4	935.12			934.99	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
235	Node 101	ST-15	Junction	Junction -with top inlet	1:01	923.34	3.14	926.34	3.00	Top Inlet 4	926.34			926.34	6.28	3	3.14	0.67
236	Node 830	ST-15	Junction	Junction -with top inlet	1:01	921.18	3.14	924.18	3.00	Top Inlet 4	924.18			924.18	6.28	3	3.14	0.67
237	Node 561	ST-15	Junction	Junction -with top inlet	3:01	920.61	3.14	923.6	2.99	Top Inlet 4	923.6			923.61	6.28	3	3.14	0.67
238	Node 430	ST-15	Start	US Junction -with top inlet	0:01	920.94	3.14	923.96	3.02	Top Inlet 4	923.96			923.94	6.28	3	3.14	0.67
239	Node 690	ST-15	Start	US Junction -with top inlet	0:01	913.94	3.14	917.71	3.77	Top Inlet 4	917.71			917.74	6.28	3	3.14	0.67
240	Node 303	ST-15	Junction	Junction -with top inlet	2:01	911.31	20	917.71	6.40	Top Inlet 1	917.71			917.68	5	3	2.50	0.67
241	Node 984	ST-15	Junction	Junction -with top inlet	1:01	910.38	25	916.88	6.50	Top Inlet 3	916.88			916.88	8	3	4.00	0.67
242	Node 1022	ST-15	Culvert Opening	DS Culvert opening	1:00	908.68	0	908.14	-0.54						6.28	3	3.14	0.67
243	Node 545	ST-15	Junction	Junction -with top inlet	1:01	922.42	3.14	925.43	3.01	Top Inlet 4	925.43			925.42	6.28	3	3.14	0.67
244	Node 123	ST-15	Junction	Junction -with top inlet	1:01	922.09	3.14	925.09	3.00	Top Inlet 4	925.09			925.09	6.28	3	3.14	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
245	Node 535	ST-15	Start	US Junction -with top inlet	0:01	922.24	3.14	927.24	5.00	Top Inlet 4	927.24			927.24	6.28	3	3.14	0.67
246	Node 857	ST-15	Start	US Junction -with top inlet	0:01	968.26	3.14	971.24	2.98	Top Inlet 4	971.24			971.26	6.28	3	3.14	0.67
247	Node 680	ST-15	Junction	Junction -with top inlet	1:01	968.17	3.14	971.41	3.24	Top Inlet 4	971.41			971.37	6.28	3	3.14	0.67
248	Node 248	ST-15	Junction	Junction -with top inlet	1:01	964.29	3.14	967.29	3.00	Top Inlet 4	967.29			967.29	6.28	3	3.14	0.67
249	Node 393	ST-15	Junction	Junction -with top inlet	1:01	952.21	3.14	955.19	2.98	Top Inlet 4	955.19			955.21	6.28	3	3.14	0.67
250	Node 596	ST-15B	Culvert Opening	DS Culvert opening	1:00	975.58	0	975.54	-0.04						6.28	3	3.14	0.67
251	Node 754	ST-15B	Junction	Junction -with top inlet	1:01	987.15	3.14	990.12	2.97	Top Inlet 4	990.12			990.15	6.28	3	3.14	0.67
252	Node 887	ST-15B	Start	US Junction -with top inlet	0:01	987.22	3.14	990.12	2.90	Top Inlet 4	990.12			990.12	6.28	3	3.14	0.67
253	Node 12	ST-19	Start	US Junction -with top inlet	0:01	990.44	20	993.47	3.03	Top Inlet 1	993.47			993.44	5	3	2.50	0.67
254	Node 13	ST-19	Junction	Junction -with top	1:01	988.17	20	991.17	3.00	Top Inlet 1	991.17			991.17	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
255	Node 15	ST-19	Start	US Junction -with top inlet	0:01	971.84	20	974.81	2.97	Top Inlet 1	974.81			974.84	5	3	2.50	0.67
256	Node 16	ST-19	Junction	Junction -with top inlet	2:01	971.68	20	974.67	2.99	Top Inlet 1	974.67			974.68	5	3	2.50	0.67
257	Node 21	ST-19	Start	US Junction -with top inlet	0:01	963.91	20	966.93	3.02	Top Inlet 1	966.93			966.91	5	3	2.50	0.67
258	Node 22	ST-19	Junction	Junction -with top inlet	2:01	963.28	20	966.28	3.00	Top Inlet 1	966.28			966.28	5	3	2.50	0.67
259	Node 23	ST-19	Junction	Junction -with top inlet	1:01	959.69	20	962.73	3.04	Top Inlet 1	962.73			962.69	5	3	2.50	0.67
260	Node 24	ST-19	Junction	Junction -with top inlet	1:01	954.23	20	958.25	4.02	Top Inlet 1	958.25			958.23	5	3	2.50	0.67
261	Node 31	ST-19	Start	US Junction -with top inlet	0:01	947.15	20	950.1	2.95	Top Inlet 1	950.1			950.15	5	3	2.50	0.67
262	Node 32	ST-19	Junction	Junction -with top inlet	1:01	941.56	20	944.52	2.96	Top Inlet 1	944.52			944.56	5	3	2.50	0.67
263	Node 33	ST-19	Junction	Junction -with top inlet	1:01	940.89	20	943.89	3.00	Top Inlet 1	943.89			943.89	5	3	2.50	0.67
264	Node 36	ST-19	Junction	Junction -with top	1:01	931.44	20	934.45	3.01	Top Inlet 1	934.45			934.44	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
265	Node 25	ST-19	Start	US Junction -with top inlet	0:01	943.56	20	946.55	2.99	Top Inlet 1	946.55			946.56	5	3	2.50	0.67
266	Node 26	ST-19	Start	US Junction -with top inlet	0:01	943.47	20	946.47	3.00	Top Inlet 1	946.47			946.47	5	3	2.50	0.67
267	Node 27	ST-19	Junction	Junction -with top inlet	2:01	935.99	20	947.47	11.48	Top Inlet 1	947.47			947.48	5	3	2.50	0.67
268	Node 28	ST-19	Junction	Junction -with top inlet	1:01	942.6	20	946.66	4.06	Top Inlet 1	946.66			946.6	5	3	2.50	0.67
269	Node 29	ST-19	Junction	Junction -with top inlet	2:01	934.81	20	947.48	12.67	Top Inlet 1	947.48			947.47	5	3	2.50	0.67
270	Node 30	ST-19	Junction	Junction -with top inlet	1:01	928.26	20	935.88	7.62	Top Inlet 1	935.88			935.9	5	3	2.50	0.67
271	Node 37	ST-19	Junction	Junction -with top inlet	1:01	920.52	20	927.97	7.45	Top Inlet 1	927.97			927.99	5	3	2.50	0.67
272	Node 38	ST-19	Junction	Junction -with top inlet	2:01	916.65	20	921.03	4.38	Top Inlet 1	921.03			921.05	5	3	2.50	0.67
273	Node 39	ST-19	Junction	Junction -with top inlet	1:01	922.54	20	925.58	3.04	Top Inlet 1	925.58			925.54	5	3	2.50	0.67
274	Node 35	ST-19	Junction	Junction -with top	1:01	934.96	20	937.97	3.01	Top Inlet 1	937.97			937.96	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
275	Node 43	ST-19	Junction	Junction -with top inlet	2:01	908.63	25	913.52	4.89	Top Inlet 3	913.52			913.46	8	3	4.00	0.67
276	Node 46	ST-19	Culvert Opening	DS Culvert opening	1:00	906.46	0	895.94	-10.52									
277	Node 47	ST-19	Junction	Junction -with top inlet	1:01	911.15	25	914.94	3.79	Top Inlet 3	914.94			914.92	8	3	4.00	0.67
278	Node 48	ST-19	Junction	Junction -with top inlet	1:01	911.65	20	916.19	4.54	Top Inlet 1	916.19			916.17	5	3	2.50	0.67
279	Node 49	ST-19	Start	US Junction -with top inlet	0:01	914.7	20	918.86	4.16	Top Inlet 1	918.86			918.87	5	3	2.50	0.67
280	Node 730	ST-23	Start	US Junction -with top inlet	0:01	912.06	20	913.38	1.32	Top Inlet 1	913.38			914.36	5	3	2.50	0.67
281	Node 883	ST-23	Culvert Opening	DS Culvert opening	1:00	900.68	0	899.06	-1.62									
282	Node 41	ST-23	Junction	Junction -with top inlet	1:01	903.36	20	913.42	10.06	Top Inlet 1	913.42			913.64	5	3	2.50	0.67
283	Node 332	ST-16	Culvert Opening	DS Culvert opening	1:00	905.93	0	904.69	-1.24									
284	Node 122	ST-27	Junction	Junction -with top	1:01	949.96	20	953	3.04	Top Inlet 1	953			952.96	5	3	2.50	0.67

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
				inlet														
285	Node 40	ST-27	Start	US Junction -with top inlet	0:01	949.99	20	952.88	2.89	Top Inlet 1	952.88			952.89	5	3	2.50	0.67
286	Node 792	ST-27	Start	US Junction -with top inlet	0:01	935.64	20	938.16	2.52	Top Inlet 1	938.16			938.04	5	3	2.50	0.67
287	Node 555	ST-27	Junction	Junction -with top inlet	2:01	935.54	20	938.56	3.02	Top Inlet 1	938.56			938.54	5	3	2.50	0.67
288	Node 978	ST-27	Junction	Junction -with top inlet	1:01	915.12	20	919.98	4.86	Top Inlet 1	919.98			919.97	5	3	2.50	0.67
289	Node 1021	ST-27	Culvert Opening	DS Culvert opening	1:00	895.5	0	895.7	0.20									
290	Node 65	ST-03	Junction	Junction -with top inlet	1:01	927.25	0	931.66	4.41	Top Inlet 3	931.66			932.09	8	3	4.00	0.67
291	Node 89 (1)	ST-12b	Culvert Opening	DS Culvert opening	1:00	910.33	0	909.19	-1.14									0.67
292	Node 5 (1)	ST-12b	Start	US Junction -with top inlet	0:01	914.37	10	920.75	6.38	Top Inlet 8	920.75			920.75	18	3	18.00	0.67
293	Node 59	ST-39	Culvert Opening	DS Culvert opening	1:00	889.23	0	889.19	-0.04									
294	Node 59 (1)	ST-39	Culvert Openin	DS Culvert openin	1:00	889.23	0	889.22	-0.01									

FID	Name	System Name	Node Type	Node Status	Conduit Connections (US:DS)	Invert Elevation	Base Area	Terrain Elevation	Depth	Top Inlet Type	Top Inlet Elevation	Side Inlet Type	Side Inlet Elevation	Drop Inlet Elevation	Drop Inlet Weir Length	Drop Inlet Weir Coef	Drop Inlet Orifice Area	Drop Inlet Orifice Coef
			g	g														
295	Node 45	ST-39	Start	US Junction -with top inlet - with side inlet	0:01	889.75	33	893.91	4.16	Top Inlet 10	897.75	Curb-Opening-Inlet-0.5x3ft (default)	893.75	897.75	23	3	33.00	0.67
296	Node 45 (1)	ST-39	Start	US Junction -with top inlet - with side inlet	0:01	889.75	33	894.09	4.34	Top Inlet 10	897.75	Curb-Opening-Inlet-0.5x3ft (default)	893.75	897.75	23	3	33.00	0.67

M.3.4 PROPOSED CONDITIONS HEC RAS CONDUIT ATTRIBUTE TABLE

Table M.3.4. Proposed Conditions Conduit Attribute Table

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
0	ST-05d	ST-05	Node 685	Node 1075	Hydraulic	124.94	10	Circular	1.5	1.5	0.024	927.74	926.31	0.01	None
1	ST-05a	ST-05	Node 799	Node 51	Hydraulic	84.57	10	Circular	2	2	0.024	924.91	919.3	0.07	None
2	ST-05c	ST-05	Node 1075	Node 390	Hydraulic	51.51	10	Circular	1.5	1.5	0.024	926.31	925.52	0.02	None
3	ST-05b	ST-05	Node 390	Node 799	Hydraulic	29.50	10	Circular	2	2	0.024	925.52	924.91	0.03	None
4	ST-09c	ST-09	Node 761	Node 237	Hydraulic	233.11	10	Circular	1.5	1.5	0.024	932.11	927.72	0.02	None
5	ST-09e	ST-09	Node 9	Node 389	Hydraulic	171.42	10	Circular	1.5	1.5	0.024	936.46	932.56	0.02	None
6	ST-09b	ST-09	Node 237	Node 893	Hydraulic	134.60	10	Circular	2	2	0.024	927.72	923.38	0.03	None
7	ST-09d	ST-09	Node 389	Node 761	Hydraulic	31.24	10	Circular	1.5	1.5	0.024	932.56	932.11	0.02	None
8	ST-09a	ST-09	Node 893	Node 81	Hydraulic	263.16	10	Circular	3	3	0.024	923.38	915.64	0.03	None
9	ST-16i	ST-16	Node 484	Node 359	Hydraulic	36.79	10	Circular	2	2	0.009	909.94	909.66	0.01	None
10	ST-16d	ST-16	Node 534	Node 129	Hydraulic	262.85	10	Circular	2	2	0.015	918.14	913.46	0.02	None
11	ST-16j	ST-16	Node 620	Node 484	Hydraulic	26.41	10	Circular	1.5	1.5	0.009	910.77	909.94	0.04	None
12	ST-16k	ST-16	Node 100	Node 484	Hydraulic	141.48	10	Circular	2	2	0.009	911.67	909.94	0.01	None
13	ST-16m	ST-16	Node 358	Node 760	Hydraulic	25.70	10	Circular	1.5	1.5	0.009	912.16	911.68	0.02	None
14	ST-16h	ST-16	Node 359	Node 222	Hydraulic	27.54	10	Circular	2	2	0.009	909.66	911.13	-0.07	None
15	ST-16n	ST-16	Node 742	Node 760	Hydraulic	43.44	10	Circular	2	2	0.009	911.84	911.68	0.00	None
16	ST-16l	ST-16	Node 760	Node 100	Hydraulic	8.51	10	Circular	2	2	0.009	911.68	911.67	0.00	None
17	ST-16c	ST-16	Node	Node	Hydraulic	26.10	10	Circular	1.5	1.5	0.015	913.63	913.46	0.01	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
			1081	129											
18	ST-16o	ST-16	Node 1045	Node 742	Hydraulic	25.69	10	Circular	1.5	1.5	0.009	912.22	911.84	0.02	None
19	ST-16b	ST-16	Node 129	Node 222	Hydraulic	101.50	10	Circular	2	2	0.015	913.46	911.13	0.02	None
20	ST-16e	ST-16	Node 963	Node 534	Hydraulic	25.21	10	Circular	1.5	1.5	0.015	918.24	918.14	0.00	None
21	ST-16g	ST-16	Node 658	Node 247	Hydraulic	25.37	10	Circular	1.5	1.5	0.015	919.92	919.74	0.01	None
22	ST-16f	ST-16	Node 247	Node 534	Hydraulic	70.71	10	Circular	2	2	0.015	919.74	918.14	0.02	None
23	ST-20s	ST-20	Node 540	Node 75	Hydraulic	107.88	10	Circular	2.5	2.5	0.024	922.11	920.64	0.01	None
24	ST-20q	ST-20	Node 75	Node 1052	Hydraulic	276.73	10	Circular	2.5	2.5	0.024	920.64	919.26	0.01	None
25	ST-20u	ST-20	Node 70	Node 972	Hydraulic	33.02	10	Circular	2	2	0.024	922.4	922.3	0.00	None
26	ST-20t	ST-20	Node 972	Node 540	Hydraulic	100.22	10	Circular	2	2	0.024	922.3	922.11	0.00	None
27	ST-20r	ST-20	Node 1035	Node 75	Hydraulic	158.98	10	Circular	1.25	1.25	0.024	923.6	920.64	0.02	None
28	ST-20m	ST-20	Node 1052	Node 640	Hydraulic	197.99	10	Circular	2.833333	2.833333	0.024	919.26	918.42	0.00	None
29	ST-20o	ST-20	Node 411	Node 371	Hydraulic	16.60	10	Circular	1.25	1.25	0.024	919.91	919.45	0.04	None
30	ST-20l	ST-20	Node 640	Node 766	Hydraulic	148.09	10	Circular	2.833333	2.833333	0.024	918.42	916.41	0.01	None
31	ST-20n	ST-20	Node 371	Node 1052	Hydraulic	40.59	10	Circular	1.5	1.5	0.024	919.45	919.26	0.01	None
32	ST-20g	ST-20	Node 509	Node 903	Hydraulic	259.93	10	Circular	3	3	0.024	911.32	907.57	0.01	None
33	ST-20i	ST-20	Node 206	Node 641	Hydraulic	59.32	10	Circular	1.25	1.25	0.024	912.98	911.42	0.03	None
34	ST-20k	ST-20	Node 354	Node 766	Hydraulic	42.06	10	Circular	1.25	1.25	0.024	916.48	916.41	0.00	None
35	ST-20j	ST-20	Node 766	Node 641	Hydraulic	188.93	10	Circular	3	3	0.024	916.41	911.42	0.03	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
36	ST-20h	ST-20	Node 641	Node 509	Hydraulic	32.09	10	Circular	3	3	0.024	911.42	911.32	0.00	None
37	ST-20e	ST-20	Node 11	Node 333	Hydraulic	56.85	10	Circular	2	2	0.024	907.99	907.89	0.00	None
38	ST-20c	ST-20	Node 767	Node 1025 (1)	Hydraulic	201.41	10	Circular	3.333333	3.333333	0.024	907.07	902.93	0.02	None
39	ST-20b	ST-20	Node 614 (1)	Node 1025 (1)	Hydraulic	45.53	10	Circular	1.5	1.5	0.024	905.64	902.93	0.07	None
40	ST-20f	ST-20	Node 903	Node 767	Hydraulic	158.16	10	Circular	3	3	0.024	907.57	907.07	0.00	None
41	ST-20d	ST-20	Node 333	Node 767	Hydraulic	42.77	10	Circular	1.5	1.5	0.024	907.89	907.07	0.02	None
42	ST-20a	ST-20	Node 1025 (1)	Node 84	Hydraulic	344.37	10	Circular	4	4	0.024	902.93	901	0.01	None
43	ST-22	ST-22	Node 192	Node 863	Hydraulic	47.63	10	Circular	1.25	1.25	0.024	908.6	900	0.21	None
44	ST-22 1	ST-22	Node 863	Node 447	Hydraulic	54.14	10	Circular	2	2	0.015	900	897.64	0.05	Flap
45	ST-26 2	ST-26	Node 3	Node 85	Hydraulic	158.74	10	Circular	1.5	1.5	0.015	906.65	900	0.04	None
46	ST-26 3	ST-26	Node 104	Node 3	Hydraulic	62.03	10	Circular	1.25	1.25	0.015	907.01	906.65	0.01	None
47	ST-45 2	ST-45	Node 76	Node 50	Hydraulic	82.11	10	Circular	2	2	0.015	903.44	893.77	0.13	None
48	ST-45 4	ST-45	Node 781	Node 7	Hydraulic	123.35	10	Circular	2	2	0.015	902.47	902.25	0.00	None
49	ST-43 2	ST-43	Node 914	Node 494	Hydraulic	50.73	10	Circular	1.25	1.25	0.024	903	895	0.18	None
50	A-10b	A-10	Node 707	Node 34	Hydraulic	22.49	10	Circular	1.5	1.5	0.024	952.53	952.42	0.01	None
51	A-10a	A-10	Node 34	Node 939	Hydraulic	155.02	10	Circular	1.5	1.5	0.024	952.42	943.73	0.06	None
52	A-6c	A-6	Node 67	Node 587	Hydraulic	61.04	10	Circular	1.5	1.5	0.024	946.09	944.58	0.03	None
53	A-7b	A-7	Node 549	Node 827	Hydraulic	51.95	10	Circular	1.25	1.25	0.024	947.91	946.76	0.02	None
54	A-6b	A-6	Node 168	Node 587	Hydraulic	31.20	10	Circular	1.5	1.5	0.024	945.33	944.58	0.03	None
55	A-6a	A-6	Node 587	Node 743	Hydraulic	150.91	10	Circular	1.5	1.5	0.024	944.58	932.32	0.08	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
56	A-7a	A-7	Node 827	Node 145	Hydraulic	173.42	10	Circular	1.25	1.25	0.024	946.76	930.7	0.09	None
57	A-2b	A-2	Node 57	Node 1033	Hydraulic	35.72	10	Circular	1.25	1.25	0.024	934.92	934.99	0.00	None
58	A-2e	A-2	Node 454	Node 868	Hydraulic	32.58	10	Circular	1.25	1.25	0.024	943.78	943.68	0.00	None
59	A-2d	A-2	Node 868	Node 127	Hydraulic	65.37	10	Circular	1.25	1.25	0.024	943.68	940.11	0.06	None
60	A-2a	A-2	Node 1033	Node 1020	Hydraulic	44.14	10	Circular	1.25	1.25	0.024	934.99	933.28	0.04	None
61	A-2c	A-2	Node 127	Node 1033	Hydraulic	206.30	10	Circular	1.25	1.25	0.024	940.11	934.99	0.03	None
62	A-4a	A-4	Node 704	Node 916	Hydraulic	66.69	10	Circular	1.25	1.25	0.024	925.61	921.47	0.06	None
63	ST-37	ST-37	Node 128	Node 905	Hydraulic	27.09	10	Circular	1.25	1.25	0.015	904.49	904.4	0.00	None
64	ST-37 1	ST-37	Node 1050	Node 1079	Hydraulic	213.90	10	Circular	1.25	1.25	0.015	909.09	903.5	0.03	None
65	ST-37 2	ST-37	Node 884	Node 464	Hydraulic	37.59	10	Circular	1.5	1.5	0.024	905.7	905.65	0.00	None
66	ST-37 3	ST-37	Node 492	Node 618	Hydraulic	106.57	10	Circular	2	2	0.015	906.75	906.11	0.01	None
67	ST-37 4	ST-37	Node 905	Node 385	Hydraulic	57.76	10	Circular	3	3	0.015	904.4	904.33	0.00	None
68	ST-37 5	ST-37	Node 370	Node 902	Hydraulic	119.77	10	Circular	1.25	1.25	0.015	908.94	908.28	0.01	None
69	ST-37 6	ST-37	Node 83	Node 491	Hydraulic	77.74	10	Circular	2	2	0.015	905.83	905.68	0.00	None
70	ST-37 7	ST-37	Node 275	Node 188	Hydraulic	47.08	10	Circular	1.25	1.25	0.015	904.71	904.65	0.00	None
71	ST-37 8	ST-37	Node 1038	Node 385	Hydraulic	316.18	10	Circular	2	2	0.024	905.6	904.33	0.00	None
72	ST-37 9	ST-37	Node 777	Node 902	Hydraulic	42.40	10	Circular	1	1	0.015	908.37	908.28	0.00	None
73	ST-37 10	ST-37	Node 350	Node 905	Hydraulic	64.63	10	Circular	3	3	0.015	904.44	904.4	0.00	None
74	ST-37 11	ST-37	Node 615	Node	Hydraulic	33.18	10	Circular	1.25	1.25	0.024	905.75	905.65	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
				464											
75	ST-37 12	ST-37	Node 1054	Node 369	Hydraulic	30.32	10	Circular	1.25	1.25	0.015	908.54	908.47	0.00	None
76	ST-37 14	ST-37	Node 618	Node 83	Hydraulic	53.00	10	Circular	2	2	0.015	906.11	905.83	0.01	None
77	ST-37 15	ST-37	Node 385	Node 828	Hydraulic	222.37	10	Circular	3.5	3.5	0.024	904.33	903.47	0.00	None
78	ST-37 16	ST-37	Node 369	Node 233	Hydraulic	91.83	10	Circular	1.25	1.25	0.015	908.47	907.11	0.02	None
79	ST-37 17	ST-37	Node 812	Node 905	Hydraulic	61.08	10	Circular	2.5	2.5	0.024	904.5	904.4	0.00	None
80	ST-37 18	ST-37	Node 491	Node 915	Hydraulic	28.72	10	Circular	3	3	0.015	905.68	905.65	0.00	None
81	ST-37 19	ST-37	Node 246	Node 669	Hydraulic	248.33	10	Circular	1.5	1.5	0.015	904.63	901.12	0.01	None
82	ST-37 20	ST-37	Node 233	Node 491	Hydraulic	93.13	10	Circular	1.25	1.25	0.015	907.11	905.68	0.02	None
83	ST-37 21	ST-37	Node 495	Node 884	Hydraulic	35.57	10	Circular	1.5	1.5	0.024	905.75	905.7	0.00	None
84	ST-37 22	ST-37	Node 989	Node 275	Hydraulic	58.27	10	Circular	1.25	1.25	0.015	904.82	904.71	0.00	None
85	ST-37 23	ST-37	Node 464	Node 1038	Hydraulic	43.21	10	Circular	1.75	1.75	0.024	905.65	905.6	0.00	None
86	ST-37 24	ST-37	Node 758	Node 83	Hydraulic	36.58	10	Circular	1.25	1.25	0.015	905.87	905.83	0.00	None
87	ST-37 25	ST-37	Node 915	Node 188	Hydraulic	35.26	10	Circular	3	3	0.015	905.65	904.65	0.03	None
88	ST-37 26	ST-37	Node 188	Node 350	Hydraulic	202.30	10	Circular	3	3	0.015	904.65	904.44	0.00	None
89	ST-37 27	ST-37	Node 828	Node 783	Hydraulic	15.15	10	Circular	3.5	3.5	0.024	903.47	903.43	0.00	None
90	ST-37 28	ST-37	Node 899	Node 812	Hydraulic	99.09	10	Circular	1.25	1.25	0.024	904.55	904.5	0.00	None
91	ST-37 29	ST-37	Node 1079	Node 828	Hydraulic	56.45	10	Circular	1.5	1.5	0.024	903.5	903.47	0.00	None
92	ST-37 30	ST-37	Node 902	Node 492	Hydraulic	168.90	10	Circular	1.25	1.25	0.015	908.28	906.75	0.01	None
93	ST-37 31	ST-37	Node 669	Node 77	Hydraulic	49.92	10	Circular	1.5	1.5	0.024	901.12	896.16	0.11	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
94	ST-37 32	ST-37	Node 783	Node 77	Hydraulic	173.74	10	Circular	3.5	3.5	0.024	903.43	896.16	0.04	None
95	ST-37 33	ST-37	Node 177	Node 721	Hydraulic	260.18	10	Circular	2	2	0.015	898.05	896.97	0.00	None
96	ST-37 35	ST-37	Node 864	Node 177	Hydraulic	16.79	10	Circular	2	2	0.015	903.83	898.05	0.49	None
97	ST-37 37	ST-37	Node 42	Node 721	Hydraulic	127.02	10	Circular	1.5	1.5	0.015	898.92	896.97	0.02	None
98	ST-37 38	ST-37	Node 721	Node 77	Hydraulic	278.25	10	Circular	3	3	0.015	896.97	896.16	0.00	None
99	ST-37 39	ST-37	Node 634	Node 177	Hydraulic	144.70	10	Circular	2	2	0.015	904.38	898.05	0.05	None
100	ST-37 40	ST-37	Node 1051	Node 634	Hydraulic	36.21	10	Circular	1.25	1.25	0.015	904.43	904.38	0.00	None
101	ST-37 41	ST-37	Node 86	Node 246	Hydraulic	44.20	10	Circular	1.25	1.25	0.015	907.43	904.63	0.07	None
102	ST-37 42	ST-37	Node 1014	Node 246	Hydraulic	72.87	10	Circular	1.5	1.5	0.015	905.19	904.63	0.01	None
103	ST-37 43	ST-37	Node 66	Node 181	Hydraulic	74.10	10	Circular	1.25	1.25	0.015	911.1	911.01	0.00	None
104	ST-37 45	ST-37	Node 181	Node 595	Hydraulic	49.64	10	Circular	1.5	1.5	0.015	911.01	910.97	0.00	None
105	ST-37 46	ST-37	Node 595	Node 1014	Hydraulic	285.33	10	Circular	1.5	1.5	0.015	910.97	905.19	0.02	None
106	A-1i	A-12	Node 403	Node 1084	Hydraulic	231.45	10	Circular	1.5	1.5	0.024	969.9	962.04	0.03	None
107	A-1l	A-12	Node 170	Node 249	Hydraulic	54.51	10	Circular	1.25	1.25	0.024	990.35	990.06	0.01	None
108	A-1j	A-12	Node 862	Node 403	Hydraulic	63.24	10	Circular	1.25	1.25	0.024	972.06	969.9	0.04	None
109	A-1g	A-12	Node 1084	Node 318	Hydraulic	39.36	10	Circular	1.25	1.25	0.024	962.04	951.61	0.29	None
110	A-1k	A-12	Node 249	Node 403	Hydraulic	332.43	10	Circular	1.5	1.5	0.024	990.06	969.9	0.06	None
111	A-1h	A-12	Node 451	Node 1084	Hydraulic	54.74	10	Circular	1.25	1.25	0.024	962.19	962.04	0.00	None
112	A-8c	A-8	Node 973	Node 502	Hydraulic	144.97	10	Circular	1.5	1.5	0.024	960.56	955.44	0.04	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
113	A-8b	A-8	Node 502	Node 1077	Hydraulic	238.69	10	Circular	2	2	0.024	955.44	944.83	0.04	None
114	A-8a	A-8	Node 1077	Node 312	Hydraulic	324.37	10	Circular	2	2	0.024	944.83	929.46	0.05	None
115	A-8d	A-8	Node 536	Node 973	Hydraulic	28.96	10	Circular	1.5	1.5	0.024	960.62	960.56	0.00	None
116	A-11a	A-11	Node 148	Node 512	Hydraulic	96.17	10	Circular	1.25	1.25	0.024	968.4	968.33	0.00	None
117	A-3h	A-3	Node 17	Node 700	Hydraulic	79.18	10	Circular	1.25	1.25	0.024	961.35	957.99	0.04	None
118	A-3p	A-3	Node 1073	Node 290	Hydraulic	34.13	10	Circular	1.25	1.25	0.024	977.42	976.16	0.04	None
119	A-3d	A-3	Node 590	Node 997	Hydraulic	45.31	10	Circular	1.25	1.25	0.024	961.58	959.2	0.05	None
120	A-3c	A-3	Node 997	Node 944	Hydraulic	42.93	10	Circular	1.25	1.25	0.024	959.2	957.94	0.03	None
121	A-3m	A-3	Node 979	Node 556	Hydraulic	86.70	10	Circular	1.25	1.25	0.024	975.53	972.03	0.04	None
122	A-3j	A-3	Node 429	Node 808	Hydraulic	65.46	10	Circular	1.25	1.25	0.024	967.61	967.52	0.00	None
123	A-3f	A-3	Node 700	Node 944	Hydraulic	17.98	10	Circular	1.25	1.25	0.024	957.99	957.94	0.00	None
124	A-3i	A-3	Node 808	Node 17	Hydraulic	123.34	10	Circular	1.25	1.25	0.024	967.52	961.35	0.05	None
125	A-3n	A-3	Node 674	Node 979	Hydraulic	40.59	10	Circular	1.25	1.25	0.024	977.14	975.53	0.04	None
126	A-3b	A-3	Node 944	Node 329	Hydraulic	109.67	10	Circular	3	3	0.024	957.94	953.42	0.04	None
127	A-3l	A-3	Node 556	Node 808	Hydraulic	52.26	10	Circular	1.25	1.25	0.024	972.03	967.52	0.09	None
128	A-3g	A-3	Node 301	Node 700	Hydraulic	31.10	10	Circular	1.25	1.25	0.024	958.06	957.99	0.00	None
129	A-3k	A-3	Node 20	Node 429	Hydraulic	31.87	10	Circular	1.25	1.25	0.024	967.7	967.61	0.00	None
130	A-3o	A-3	Node 290	Node 979	Hydraulic	34.32	10	Circular	1.25	1.25	0.024	976.16	975.53	0.02	None
131	A-3a	A-3	Node 329	Node 811	Hydraulic	70.73	10	Circular	3	3	0.024	953.42	953.13	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
132	A-9	A-9	Node 335	Node 19	Hydraulic	49.93	10	Circular	2	2	0.024	937.15	936.82	0.01	None
133	A-9 1	A-9	Node 401	Node 751	Hydraulic	169.16	10	Circular	2	2	0.024	938.55	937.46	0.01	None
134	A-9 2	A-9	Node 802	Node 401	Hydraulic	36.38	10	Circular	2	2	0.024	938.87	938.55	0.01	None
135	A-9 3	A-9	Node 310	Node 19	Hydraulic	24.02	10	Circular	2	2	0.024	936.86	936.82	0.00	None
136	A-9 4	A-9	Node 527	Node 14	Hydraulic	37.51	10	Circular	2	2	0.024	934.08	932.98	0.03	None
137	A-9 5	A-9	Node 657	Node 947	Hydraulic	44.62	10	Circular	2	2	0.024	936.87	936.75	0.00	None
138	A-9 6	A-9	Node 861	Node 19	Hydraulic	38.06	10	Circular	2	2	0.024	936.83	936.82	0.00	None
139	A-9 7	A-9	Node 579	Node 14	Hydraulic	17.04	10	Circular	2	2	0.024	938.09	932.98	0.34	None
140	A-9 8	A-9	Node 550	Node 14	Hydraulic	20.67	10	Circular	2	2	0.024	937.83	932.98	0.26	None
141	A-9 9	A-9	Node 14	Node 260	Hydraulic	29.12	10	Circular	2	2	0.024	932.98	932.86	0.00	None
142	A-9 10	A-9	Node 947	Node 14	Hydraulic	57.03	10	Circular	2	2	0.024	936.75	932.98	0.07	None
143	A-9 11	A-9	Node 610	Node 191	Hydraulic	207.58	10	Circular	1.5	1.5	0.024	964.14	953.15	0.05	None
144	A-9 12	A-9	Node 654	Node 922	Hydraulic	148.04	10	Circular	1.25	1.25	0.024	974.36	972.97	0.01	None
145	A-9 13	A-9	Node 425	Node 442	Hydraulic	39.75	10	Circular	1.25	1.25	0.024	958.44	953.95	0.12	None
146	A-9 14	A-9	Node 831	Node 550	Hydraulic	106.42	10	Circular	1.5	1.5	0.024	947.29	937.83	0.09	None
147	A-9 15	A-9	Node 860	Node 922	Hydraulic	30.74	10	Circular	1.5	1.5	0.024	973.08	972.97	0.00	None
148	A-9 16	A-9	Node 751	Node 335	Hydraulic	141.15	10	Circular	2	2	0.024	937.46	937.15	0.00	None
149	A-9 17	A-9	Node 406	Node 442	Hydraulic	76.12	10	Circular	1.25	1.25	0.024	961.92	953.95	0.11	None
150	A-9 18	A-9	Node 922	Node 610	Hydraulic	300.97	10	Circular	1.5	1.5	0.024	972.97	964.14	0.03	None
151	A-9 19	A-9	Node 949	Node 654	Hydraulic	239.53	10	Circular	1.25	1.25	0.024	986	974.36	0.05	None
152	A-9 20	A-9	Node 442	Node 831	Hydraulic	56.71	10	Circular	1.25	1.25	0.024	953.95	947.29	0.12	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
153	A-9 21	A-9	Node 308	Node 728	Hydraulic	32.82	10	Circular	1.25	1.25	0.024	973.25	973.1	0.00	None
154	A-9 22	A-9	Node 729	Node 335	Hydraulic	17.39	10	Circular	1.5	1.5	0.024	937.25	937.15	0.01	None
155	A-9 23	A-9	Node 191	Node 729	Hydraulic	326.66	10	Circular	1.5	1.5	0.024	953.15	937.25	0.05	None
156	A-9 24	A-9	Node 240	Node 654	Hydraulic	36.41	10	Circular	1.25	1.25	0.024	974.68	974.36	0.01	None
157	A-9 25	A-9	Node 947	Node 19	Hydraulic	53.48	10	Circular	1.25	1.25	0.024	936.75	936.82	0.00	None
158	A-9 26	A-9	Node 265	Node 425	Hydraulic	176.99	10	Circular	1.25	1.25	0.024	969.72	958.44	0.06	None
159	A-9 27	A-9	Node 728	Node 860	Hydraulic	33.50	10	Circular	1.5	1.5	0.024	973.1	973.08	0.00	None
160	A-9 28	A-9	Node 946	Node 610	Hydraulic	30.75	10	Circular	1.25	1.25	0.024	964.27	964.14	0.00	None
161	A-9 29	A-9	Node 153	Node 831	Hydraulic	30.88	10	Circular	1.25	1.25	0.024	947.35	947.29	0.00	None
162	A-5 1	A-5	Node 712	Node 6	Hydraulic	160.66	10	Circular	3	3	0.024	937.44	923.67	0.09	None
163	A-5 2	A-5	Node 566	Node 61	Hydraulic	58.00	10	Circular	3	3	0.024	920.09	918.55	0.03	None
164	A-5 3	A-5	Node 139	Node 712	Hydraulic	106.88	10	Circular	3	3	0.024	945.2	937.44	0.07	None
165	A-5 5	A-5	Node 565	Node 977	Hydraulic	31.44	10	Circular	1.25	1.25	0.024	951.92	951.81	0.00	None
166	A-5 6	A-5	Node 6	Node 566	Hydraulic	102.47	10	Circular	3	3	0.024	923.67	920.09	0.04	None
167	A-5 7	A-5	Node 977	Node 139	Hydraulic	70.09	10	Circular	1.25	1.25	0.024	951.81	945.2	0.10	None
168	A-5 9	A-5	Node 1078	Node 139	Hydraulic	48.22	10	Circular	2	2	0.024	948.06	945.2	0.06	None
169	A-5 11	A-5	Node 280	Node 712	Hydraulic	35.12	10	Circular	1.25	1.25	0.024	937.78	937.44	0.01	None
170	A-5 12	A-5	Node 568	Node 139	Hydraulic	30.00	10	Circular	1.25	1.25	0.024	945.26	945.2	0.00	None
171	A-5 13	A-3	Node 10	Node 944	Hydraulic	60.95	10	Circular	1.25	1.25	0.024	956.84	957.94	-0.02	None
172	ST-02	ST-02	Node 52	Node 1	Hydraulic	121.55	10	Circular	2	2	0.015	932.66	929.52	0.03	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
173	ST-02 1	ST-02	Node 53	Node 52	Hydraulic	39.93	10	Circular	2	2	0.015	933.2	932.66	0.02	None
174	ST-02 2	ST-02	Node 55	Node 52	Hydraulic	104.54	10	Circular	2	2	0.015	935.22	932.66	0.03	None
175	ST-02 3	ST-02	Node 54	Node 55	Hydraulic	39.16	10	Circular	2	2	0.015	936.17	935.22	0.03	None
176	ST-02 4	ST-02	Node 56	Node 54	Hydraulic	370.93	10	Circular	2	2	0.015	951.56	936.17	0.04	None
177	ST-03 6	ST-03	Node 63	Node 62	Hydraulic	189.88	10	Circular	1.25	1.25	0.015	932.17	929.99	0.01	None
178	ST-03 3	ST-03	Node 62	Node 60	Hydraulic	36.25	10	Circular	1.5	1.5	0.024	929.99	924.42	0.18	None
179	ST-03 4	ST-03	Node 64	Node 62	Hydraulic	54.36	10	Circular	1.25	1.25	0.024	930.73	929.99	0.02	None
180	ST-03 5	ST-03	Node 72	Node 64	Hydraulic	112.32	10	Circular	1.25	1.25	0.015	931.26	930.73	0.00	None
181	ST-10 1	ST-10	Node 2	Node 4	Hydraulic	41.96	10	Circular	2	2	0.015	912.59	912.03	0.01	Flap
182	ST-45 3	ST-45	Node 7	Node 50	Hydraulic	35.60	10	Circular	2	2	0.015	902.25	893.77	0.28	None
183	ST-02 5	ST-02	Node 80	Node 79	Hydraulic	86.35	10	Circular	2	2	0.015	1008.02	1002.93	0.06	None
184	ST-02 6	ST-02	Node 79	Node 78	Hydraulic	203.38	10	Circular	2	2	0.015	1002.93	991.81	0.06	None
185	ST-02 7	ST-02	Node 78	Node 73	Hydraulic	183.21	10	Circular	2	2	0.015	991.81	978.6	0.07	None
186	ST-02 8	ST-02	Node 74	Node 73	Hydraulic	38.96	10	Circular	2	2	0.015	978.7	978.6	0.00	None
187	ST-02 9	ST-02	Node 73	Node 58	Hydraulic	95.96	10	Circular	2	2	0.015	978.6	972.15	0.07	None
188	ST-02 10	ST-02	Node 58	Node 56	Hydraulic	325.78	10	Circular	2	2	0.015	972.15	951.56	0.06	None
189	Conduit 1	ST-37	Node 77	Node 18	Hydraulic	98.55	10	Circular	5	5	0.015	896.16	895.33	0.01	None
190	ST-26 1	ST-26	Node 85	Node 152	Hydraulic	60.18	10	Circular	2	2	0.015	900	897.6	0.04	Flap
191	ST-20 1	ST-20	Node 84	Node 44	Hydraulic	90.17	10	Circular	4	4	0.015	901	898.52	0.03	Flap
192	ST-09 1	ST-09	Node 81	Node 415	Hydraulic	63.91	10	Circular	4	4	0.015	915.64	914.76	0.01	Flap
193	ST-05 1	ST-05	Node 51	Node 854	Hydraulic	29.31	10	Circular	3	3	0.015	919.3	918.98	0.01	Flap
194	ST-43 1	ST-43	Node 494	Node 780	Hydraulic	118.40	10	Circular	2	2	0.015	895	889.65	0.05	Flap
195	ST-10 2	ST-10	Node 541	Node 2	Hydraulic	432.06	10	Circular	1.5	1.5	0.015	919.41	912.59	0.02	None
196	ST-45 1	ST-45	Node 50	Node 8	Hydraulic	61.34	10	Circular	2	2	0.015	893.77	890.79	0.05	Flap
197	ST-02 11	ST-02	Node 1	Node 87	Hydraulic	141.19	10	Circular	2	2	0.015	929.52	924.3	0.04	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
198	ST-03 2	ST-03	Node 60	Node 65	Hydraulic	23.30	10	Circular	2	2	0.015	924.42	927.25	-0.14	None
199	ST-12 1	ST-12	Node 5	Node 89	Hydraulic	65.09	10	Circular	4	4	0.015	914.37	910.33	0.06	Flap
200	Conduit 271	ST-07	Node 707 (1)	Node 334	Hydraulic	97.16	10	Circular	1.5	1.5	0.015	929.87	927.72	0.02	None
201	Conduit 47	ST-07	Node 826	Node 707 (1)	Hydraulic	51.24	10	Circular	1.5	1.5	0.015	930.4	929.87	0.01	None
202	Conduit 818	ST-07	Node 702	Node 707 (1)	Hydraulic	9.98	10	Circular	1.5	1.5	0.015	929.96	929.87	0.02	None
203	ST-11a	ST-11	Node 769	Node 361	Hydraulic	32.01	10	Circular	1.5	1.5	0.015	918.81	918.07	0.03	None
204	ST-11d	ST-11	Node 908	Node 276	Hydraulic	169.57	10	Circular	3	3	0.024	916.15	914.39	0.01	None
205	ST-11b	ST-11	Node 361	Node 648	Hydraulic	54.16	10	Circular	2	2	0.015	918.07	917.55	0.01	None
206	ST-11c	ST-11	Node 648	Node 908	Hydraulic	87.36	10	Circular	3	3	0.024	917.55	916.15	0.02	None
207	ST-15c	ST-15	Node 690	Node 303	Hydraulic	34.06	10	Circular	1.25	1.25	0.024	913.94	911.31	0.09	None
208	ST-15o	ST-15	Node 857	Node 680	Hydraulic	44.81	10	Circular	1.25	1.25	0.024	968.26	968.17	0.00	None
209	ST-15g	ST-15	Node 535	Node 545	Hydraulic	106.43	10	Circular	3	3	0.024	922.24	922.42	0.00	None
210	ST-15i	ST-15	Node 101	Node 830	Hydraulic	92.41	10	Circular	2	2	0.024	923.34	921.18	0.02	None
211	ST-15f	ST-15	Node 123	Node 561	Hydraulic	43.45	10	Circular	3	3	0.024	922.09	920.61	0.04	None
212	ST-15l	ST-15	Node 393	Node 795	Hydraulic	180.81	10	Circular	1.5	1.5	0.024	952.21	932.2	0.11	None
213	ST-15k	ST-15	Node 448	Node 795	Hydraulic	33.32	10	Circular	1.25	1.25	0.024	932.3	932.2	0.00	None
214	ST-15f 1	ST-15	Node 545	Node 123	Hydraulic	36.14	10	Circular	3	3	0.024	922.42	922.09	0.01	None
215	ST-15n	ST-15	Node 680	Node 248	Hydraulic	148.28	10	Circular	1.5	1.5	0.024	968.17	964.29	0.03	None
216	ST-15e	ST-15	Node 430	Node 561	Hydraulic	71.21	10	Circular	1.25	1.25	0.024	920.94	920.61	0.00	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
217	ST-15m	ST-15	Node 248	Node 393	Hydraulic	127.27	10	Circular	1.5	1.5	0.024	964.29	952.21	0.10	None
218	ST-15b	ST-15	Node 303	Node 984	Hydraulic	77.88	10	Circular	3.5	3.5	0.024	911.31	910.38	0.01	None
219	ST-15a	ST-15	Node 984	Node 1022	Hydraulic	112.70	10	Circular	3.5	3.5	0.024	910.38	908.68	0.02	None
220	ST-15h	ST-15	Node 830	Node 561	Hydraulic	54.16	10	Circular	2	2	0.024	921.18	920.61	0.01	None
221	ST-15d	ST-15	Node 561	Node 303	Hydraulic	203.41	10	Circular	3.5	3.5	0.024	920.61	911.31	0.05	None
222	ST-15j	ST-15	Node 795	Node 101	Hydraulic	185.58	10	Circular	2	2	0.024	932.2	923.34	0.05	None
223	ST-15Ba	ST-15B	Node 887	Node 754	Hydraulic	34.52	10	Circular	1.25	1.25	0.015	987.22	987.15	0.00	None
224	ST-15Bb	ST-15B	Node 754	Node 596	Hydraulic	134.06	10	Circular	1.25	1.25	0.015	987.15	975.58	0.09	None
225	ST-19 5	ST-19	Node 12	Node 13	Hydraulic	53.89	10	Circular	2	2	0.024	990.44	988.17	0.05	None
226	ST-19 6	ST-19	Node 13	Node 16	Hydraulic	342.81	10	Circular	2	2	0.024	988.17	971.68	0.05	None
227	ST-19 7	ST-19	Node 15	Node 16	Hydraulic	24.68	10	Circular	2	2	0.024	971.84	971.68	0.01	None
228	ST-19 8	ST-19	Node 16	Node 22	Hydraulic	338.60	10	Circular	2	2	0.024	971.68	963.28	0.03	None
229	ST-19 9	ST-19	Node 21	Node 22	Hydraulic	27.23	10	Circular	2	2	0.024	963.91	963.28	0.03	None
230	ST-19 10	ST-19	Node 22	Node 23	Hydraulic	73.92	10	Circular	2	2	0.024	963.28	959.69	0.05	None
231	ST-19 11	ST-19	Node 23	Node 24	Hydraulic	48.46	10	Circular	2	2	0.024	959.69	954.23	0.13	None
232	ST-19 12	ST-19	Node 24	Node 28	Hydraulic	165.78	10	Circular	2	2	0.024	954.23	942.6	0.07	None
233	ST-19 13	ST-19	Node 25	Node 27	Hydraulic	145.77	10	Circular	2	2	0.024	943.56	935.99	0.05	None
234	ST-19 14	ST-19	Node 26	Node 27	Hydraulic	19.60	10	Circular	2	2	0.024	943.47	935.99	0.51	None
235	ST-19 15	ST-19	Node 27	Node 29	Hydraulic	33.82	10	Circular	2	2	0.024	935.99	934.81	0.04	None
236	ST-19 16	ST-19	Node 28	Node 29	Hydraulic	16.38	10	Circular	2	2	0.024	942.6	934.81	0.69	None
237	ST-19 17	ST-19	Node 29	Node 30	Hydraulic	186.28	10	Circular	3.16	3.16	0.024	934.81	928.26	0.04	None
238	ST-19 18	ST-19	Node 31	Node 32	Hydraulic	91.97	10	Circular	2	2	0.015	947.15	941.56	0.06	None
239	ST-19 19	ST-19	Node 32	Node 33	Hydraulic	27.10	10	Circular	2	2	0.015	941.56	940.89	0.03	None
240	ST-19 20	ST-19	Node 33	Node 35	Hydraulic	196.75	10	Circular	2	2	0.015	940.89	934.96	0.03	None

FID	Name	System Name	US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n	US Elevation	DS Elevation	Slope	DS Flap Gate
241	ST-19 23	ST-19	Node 30	Node 37	Hydraulic	220.40	10	Circular	3.16	3.16	0.024	928.26	920.52	0.04	None
242	ST-27	ST-27	Node 792	Node 555	Hydraulic	54.20	10	Circular	1.25	1.25	0.024	935.64	935.54	0.00	None
243	ST-27 1	ST-27	Node 122	Node 555	Hydraulic	319.83	10	Circular	1.5	1.5	0.024	949.96	935.54	0.05	None
244	ST-27 2	ST-27	Node 40	Node 122	Hydraulic	53.64	10	Circular	1.25	1.25	0.024	949.99	949.96	0.00	None
245	ST-27 3	ST-27	Node 555	Node 978	Hydraulic	306.58	10	Circular	1.5	1.5	0.024	935.54	915.12	0.07	None
246	ST-19	ST-19	Node 49	Node 48	Hydraulic	36.92	10	Circular	1.25	1.25	0.015	914.7	911.65	0.10	None
247	ST-19 1	ST-19	Node 48	Node 47	Hydraulic	29.18	10	Circular	1.25	1.25	0.015	911.65	911.15	0.02	None
248	ST-19 2	ST-19	Node 47	Node 43	Hydraulic	176.04	10	Circular	1.25	1.25	0.015	911.15	908.63	0.01	None
249	ST-19 3	ST-19	Node 43	Node 46	Hydraulic	98.33	10	Circular	3.16	3.16	0.024	908.63	906.46	0.02	None
250	ST-19 4	ST-19	Node 38	Node 43	Hydraulic	203.72	10	Circular	3.16	3.16	0.024	916.65	908.63	0.04	None
251	ST-19 21	ST-19	Node 35	Node 36	Hydraulic	75.64	10	Circular	2	2	0.015	934.96	931.44	0.05	None
252	ST-19 22	ST-19	Node 36	Node 39	Hydraulic	128.32	10	Circular	2	2	0.015	931.44	922.54	0.07	None
253	ST-19 24	ST-19	Node 37	Node 38	Hydraulic	217.95	10	Circular	3.16	3.16	0.024	920.52	916.65	0.02	None
254	ST-19 25	ST-19	Node 39	Node 38	Hydraulic	101.39	10	Circular	2	2	0.015	922.54	916.65	0.06	None
255	ST-23	ST-23	Node 730	Node 41	Hydraulic	27.49	10	Circular	1.25	1.25	0.024	912.06	903.36	0.39	None
256	ST-23 1	ST-23	Node 41	Node 883	Hydraulic	76.23	10	Circular	3	3	0.024	903.36	900.68	0.04	Flap
257	ST-27 4	ST-27	Node 978	Node 1021	Hydraulic	76.47	10	Circular	1.5	1.5	0.024	915.12	895.5	0.27	None
258	ST-16 1	ST-16	Node 222	Node 332	Hydraulic	141.22	10	Circular	2	2	0.015	911.13	905.93	0.04	Flap
259	ST-03 1	ST-03	Node 65	Node 88	Hydraulic	123.18	10	Circular	2	2	0.015	927.25	922.01	0.04	Flap
260	ST-12 2	ST-12b	Node 5 (1)	Node 89 (1)	Hydraulic	63.29	10	Circular	4	4	0.015	914.37	910.33	0.07	Flap
261	ST-39 2	ST-39	Node 45	Node 59	Hydraulic	213.71	10	Circular	4	4	0.015	889.75	889.23	0.00	Flap
262	ST-39 1	ST-39	Node 45 (1)	Node 59 (1)	Hydraulic	231.87	10	Circular	4	4	0.015	889.75	889.23	0.00	Flap

APPENDIX M.4 – DRAINAGE STRUCTURE INFORMATION

PROPOSED CONDITIONS DRAINAGE STRUCTURE DATA TABLE

M INTERIOR DRAINAGE

M.4 DRAINAGE STRUCTURE INFORMATION

M.4.1 PROPOSED CONDITIONS DRAINAGE STRUCTURE DATA TABLE

14 drainage structures are designed to go under or though the levee or floodwall line of protection. Coordinates are in the North Kansas State Plane projection. Landside (LS) and Riverside (RS) denote what side of the line of protection is being referenced.

Pipe lengths and slopes in this table reflect information from hydraulic modelling, not civil design. Hydraulic pipe lengths should be near the value provided by civil, slopes may vary due to differences in pipe length. These deviations are not expected to significantly impact results as pipe nodes work within the larger 2D cell structure.

Limitations in hydraulic modelling of pipe networks may also lead to some deviations from civil design values, specifically for the detention basin outlet inverts. These deviations are not expected to significantly impact results as the slope is maintained.

M INTERIOR DRAINAGE

Table M.4.1. Proposed Conditions Drainage Structures Data Table

Drainage Structure	Diameter (ft)	LS Invert (ft)	RS Invert (ft)	Length (ft)	Slope	LS X Coordinate	LS Y Coordinate	RS X Coordinate	RS Y Coordinate	Maximum Velocity (ft/s) for 0.1 AEP
ST-03	2	927.25	922.01	123.18	4.25%	2,249,266.65	265,613.61	2,249,270.77	265,492.86	10.3
ST-05	3	919.3	918.98	29.31	1.09%	2,249,683.81	265,553.47	2,249,692.38	265,520.90	11.2
ST-09	4	915.64	914.76	63.91	1.38%	2,250,111.25	265,941.99	2,250,199.69	265,923.50	9.0
ST-10	2	912.59	912.03	41.96	1.33%	2,250,445.46	266,094.72	2,250,440.97	266,133.72	8.0
ST-12	4	914.37	910.33	65.09	6.21%	2,250,601.15	266,241.52	2,250,547.09	266,272.87	13.5
ST-12B	4	914.37	910.33	56.16	7.19%	2,250,604.16	266,246.71	2,250,550.10	266,278.06	13.5
ST-16	2	911.13	905.93	141.22	3.68%	2,250,879.40	266,654.85	2,250,816.20	266,755.94	13.6
ST-20	4	901	898.52	90.17	2.75%	2,251,293.83	267,647.39	2,251,235.87	267,675.90	10.7
ST-21	Triple Box 7' x 7'	905.65	900.23	436.92	1.24%	2,251,719.49	267,690.75	2,251,284.80	267,734.85	6.1
ST-22	2	900	897.64	54.14	4.36%	2,251,313.88	267,800.46	2,251,262.43	267,800.10	9.3
ST-23	3	903.36	900.68	76.23	3.52%	2,251,126.03	267,859.94	2,251,195.01	267,862.17	10.3
ST-26	2	900	897.6	60.18	3.99%	2,251,288.47	268,064.36	2,251,231.71	268,065.87	10.6
ST-39	4	889.75	889.23	198.19	0.26%	2,251,247.34	269,371.44	2,251,114.73	269,478.97	2.4
ST-39B	4	889.75	889.23	215.52	0.24%	2,251,247.34	269,371.44	2,251,114.73	269,478.97	2.7
ST-43	2	895	889.65	118.4	4.52%	2,251,445.19	269,688.42	2,251,454.96	269,806.96	11.4
ST-45	2	893.77	890.79	61.34	4.86%	2,251,643.79	269,729.14	2,251,645.35	269,790.76	13.4

This page was intentionally left blank for duplex printing.

APPENDIX N – REAL ESTATE MAPS

This page was intentionally left blank for duplex printing.