Mouse River Enhanced Flood Protection Project

Phase WC-1 Tierrecita Vallejo
Basis of Design Report—60% Design Submittal

Prepared for the Souris River Joint Board

BARR

ACKERMAN ESTVOLD

Basis of Design Report - 60% Design Submittal
MREFPP - Phase WC-1 Tierrecita Vallejo

May 2019

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List of Acronyms, Abbreviations, and Units

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AEP	Annual Exceedance Probability
APE	Area of Potential Affect
ASTM	American Society for Testing and Materials
BDR	Basis of Design Report
BFE	Base Flood Elevation
BMP	Best Management Practice
CADD	Computer-aided Drafting and Design
CFS	Cubic feet per second
CLSM	Controlled Low Strength Material
CMP	Corrugated Metal Pipe
DEM	Digital Elevation Model
DMT	Dilatometer
DTM	Digital Terrain Model
EA	Environmental Assessment
EIS	Environmental Impact Statement
ESSA	Effective Stress Stability Analysis
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Survey
GIS	Geographic Information System
HEI	Houston Engineering, Inc
HPTRM	High Performance Turf Reinforcement Mat
HTRW	Hazardous, Toxic and Radioactive Waste
JCSNWR	J. Clark Salyer Nation Wildlife Refuge
LiDAR	Light Detection and Ranging
mm/s	millimeters per second
MREFPP	Mouse River Enhanced Flood Protection Project
NAD	North American Datum
NAVD	North American Vertical Datum
NAWS	Northwest Area Water Supply
NEC	National Electrical Code
NED	National Elevation Data
NEXRAD	Next Generation Radar
NDDH	State of North Dakota Department of Health
NDDOT	North Dakota Department of Transportation
NDGF	State of North Dakota Game and Fish Department
NDSWC	North Dakota State Water Commission
NEPA	National Environmental Policy Act

NFIP	National Flood Insurance Program
NGVD	National Geodetic Vertical Datum
NLCD	National Land Cover Database
NDPDES	North Dakota Pollutant Discharge Elimination System
NRCS	National Resources Conservation Service
NWI	National Wetlands Inventory
NWS	National Weather Service
PER	Preliminary Engineer's Report
psf	pounds per square foot
PLC	Programmable Logic Controller
O&M	Operations and Maintenance
OHWL	Ordinary High Water Level
PL	Public Law
Pump Station	Refers to the entire facility associated with conveying water from the stormwater system to gatewell including all buildings, walls, pump and motor equipment, valve and meter vaults, and discharge pipes connecting to the gatewell.
RCP	Reinforced Concrete Pipe
RMSE	Root Mean Square Error
SCADA	Supervisory Control and Data Acquisition
SET	Soil Engineering Testing, Inc.
SHPO	State Historic Preservation Office
SRJB	Souris River Joint Water Resource Board, or Souris River Joint Board
SSURGO	Soil Survey Geographic Database
StARR	Structure Acquisition, Relocation or Ring Dike Program
SWE	Snow Water Equivalent
SWPPP	Stormwater Pollution Prevention Plan
TRM	Turf Reinforcement Mat
XP-SWMM	XP Solutions Storm Water Management Model
USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
USEPA	U.S. Environmental Protection Agency
USFWS	U.S. Fish and Wildlife Service, or U.S. Department of the Interior, Fish and Wildlife Service
USGS	United States Geologic Survey
USNWR	U.S. National Wildlife Refuge
USSA	Undrained Strength Stability Analysis
WRDA	Water Resource Development Act
WSS	Web Soil Survey
WTP	Water Treatment Plant

Certifications

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Executive Summary

This basis of design report (BDR) contains information related to the design of the Mouse River Enhanced Flood Protection Project (MREFPP) (Project)—Phase WC-1 Tierrecita Vallejo (Phase WC-1), located west of Minot, North Dakota. The report contains relevant design basis analysis for the flood risk management system to satisfy the U.S. Army Corps of Engineers (USACE) Section 408 permission requirements. This document reflects a 60% submittal level of design.

The MREFPP is part of an overall basin-wide effort of the Souris River Joint Board (SJRB) to address water issues within the Mouse (a.k.a. Souris) River valley. In the immediate aftermath of the record flood of 2011 the SRJB and the North Dakota State Water Commission (NDSWC) focused their attention on the developed areas of the valley in an effort to develop a plan as quickly as possible to give flooded homeowners the information they needed to make personal decisions on whether to rebuild their flooded homes. The purpose of the MREFPP was to develop a flood risk reduction project that could pass the flood of record. Project objectives included protecting as many homes as possible, minimizing the Project footprint, and minimizing impacts to unprotected features. Following significant technical analysis, stakeholder and community input, and environmental considerations, the Preliminary Engineering Report (PER) for developed areas in the basin was published in February 2012 and adopted by SRJB and Minot.

This report establishes the design basis for Phase WC-1 for flood risk management features that encompasses the Tierrecita Vallejo subdivision located immediately west of Minot, North Dakota. The north (upstream) section of the system starts north of the Canadian Pacific Railroad extending south and east along the Mouse River encompassing the southern (downstream) section of the subdivision and connecting to MREFPP Phase MI-2 at the U.S. Highway 83 Bypass.

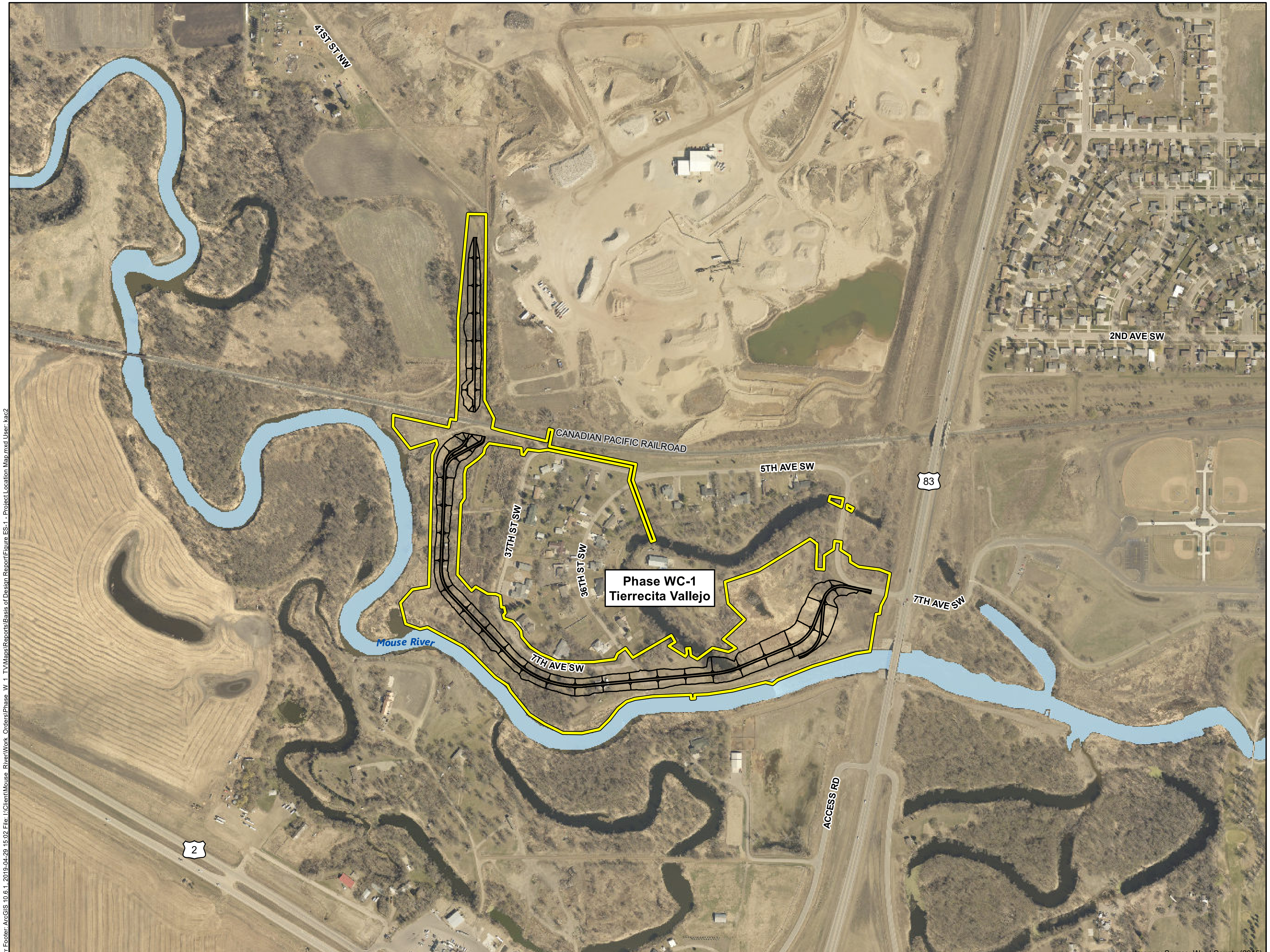
Figure ES-1 identifies the location of Phase WC-1. Major design features associated with Phase WC-1 are listed below.

- Approximately 4,700 feet of new levee
- New gatewell control structure at Mouse River and modifications to existing gatewell within oxbow
- Levee ramps for access, maintenance, and inspections
- A stoplog railroad closure with floodwall sections at the Canadian Pacific Railroad crossing
- Seepage mitigation measures including levee fill trench
- Overbank excavation adjacent to the northern bank of the Mouse River from Station 30+00F to the southbound U.S. Highway 83 bypass bridge at Station 57+00F.
- Bank and levee erosion protection adjacent to the Mouse River and existing oxbow.

- Municipal infrastructure modifications and improvements, including water main, storm sewer, and street reconstruction
- Levee work near the existing Northwest Area Water Supply (NAWS) pipe corridor near U.S. Highway 83 Bypass.

Also in planned system improvements are corrective measures and work items identified during the U.S. Army Corps of Engineers (USACE) September 2017 routine inspection:

- Unwanted vegetation growth
- Structure corrections
- Encroachments
- Bank Erosion Repair
- General Infrastructure



Phase WC-1 Construction Limits

0 250 500 1,000
Feet

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Figure ES-1

PROJECT LOCATION MAP

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

Pertinent Data

Original Project Authorization and Purpose

The Project for local flood risk-management improvements on the Souris (Mouse) River at Minot, North Dakota, was developed by the USACE over several years in three separate Congressional actions:

- Flood Control Act of 1965 (P.L. 89-298)
- USACE Chief of Engineers in House Document 286, 87th Congress, 2d Session
- USACE Chief of Engineers in House Document 321, 91st Congress, 2d Session

Project Purpose

The purpose of the Project is to meet the following goals:

- Reduce the risk of property damage and loss of life in the most densely populated reach of the river due to floods that approach the size of the 2011 flood (i.e., 27,400 cubic feet per second (cfs)), regardless of where the precipitation occurs in the Souris River Basin.
- Keep critical elements of the public transportation system operating during and after a flood similar to the 2011 flood in size.
- Design and construct a flood risk-reduction system for a 27,400 cfs flood event that meets current USACE standards and the Federal Emergency Management Agency (FEMA) requirements for accreditation.

Type of Project—MREFPP Phase WC-1

This is a local flood risk-management project consisting of levees, interior drainage facilities, gatewells, seepage control, interceptor ditches, a railroad closure structure, overbank excavation, and bank/levee erosion protection.

Hydrology and Hydraulics

Drainage area	31,200 square miles
Existing flood risk reduction capacity	5,000 cfs
Phase WC-1 design flood flow	27,400 cfs
Channel capacity (discharge at which river banks overflow)	1,150 cfs

Principal Items of Work

Levee

Existing Levee Alignment

North side of Mouse River	1,670 feet
Emergency Levee	1,740 feet

New Levee Alignment

Type	Compacted levee fill

Length	4,700 feet
Side slopes	3H:1V
Maximum height	29.5 feet
Average height	12.5 feet
Top crest width	10 feet
Stage Uncertainty	1.3 – 1.9 feet
Settlement Overbuild	12 inches
Superiority Overbuild	1.1 – 1.7 feet
Ramps	
Number of access ramps	2
Seepage Correction	
Exploration/levee fill trench (6′ depth)	940 feet
Exploration/levee fill trench (10′ depth)	2,240 feet
Sheet pile cut-off	1,300 feet
Levee and Bank Erosion Protection	
Turf reinforcement mat	95,500 square yards
Closure Structures	
Canadian Pacific Railroad closure	100 feet
Interior Drainage Facilities	
Ponding	
Tierrecita Vallejo oxbow (storage volume)	37 acre-feet at 1,557.7
Interceptor Ditches and Gatewells	
Interceptor ditches	
Length	TBD
Side slopes	TBD
Gatewells	
Tierrecita Vallejo Gatewell (new)	1
U.S. Highway 83 Gatewell (existing)	1
Outlet size	
Tierrecita Vallejo Gatewell	36-inch RCP
U.S. Highway 83 Gatewell	60-inch RCP
USACE Inspection Work Items Corrected (# of deficiencies)	
Unwanted vegetation growth	6
Structure corrections	2
Encroachments	5
Bank erosion repair	4
General infrastructure	6

Property Acquisition

Existing levee right-of-way (from USACE drawings)	7.3 acres
Existing easement in Project area to be vacated	0.0 acres
New permanent easement in Project area	23.9 acres
Net permanent easement in Project area	31.2 acres
Temporary construction easement in Project area	11.3 acres

Project Cost Share

Federal share	0 percent
Local share	100 percent

1.0 Introduction

This report establishes the design basis for Mouse River Enhanced Flood Protection Project (MREFPP)—Phase WC-1 (Phase WC-1 or Project) of the Mouse River, located near Minot, North Dakota. Throughout the remainder of this design report MREFPP - Phase WC-1 Tierrecita Vallejo is also known as Phase WC-1. The report also contains relevant hydrology and hydraulic analysis for the Mouse River to satisfy the U.S. Army Corps of Engineers (USACE) Section 408 permission requirements, including risk and uncertainty, superiority and impacts analysis.

The Phase WC-1 flood risk management alignment starts north of the Canadian Pacific Railroad extending south and east along the Mouse River encompassing the southern (downstream) section of the subdivision and connecting to MREFPP Phase MI-2 at the U.S. Highway 83 Bypass.

Phase WC-1 will impact elements of the existing federal project and requires Section 408 permission from the USACE prior to construction.

1.1 Proposed Project Background

The MREFPP is part of an overall basin-wide effort of the Souris River Joint Board (SJRB) to address water issues within the Mouse River Valley. In the immediate aftermath of the record flood of 2011 the SRJB and the North Dakota State Water Commission (NDSWC) focused their attention on the developed areas of the valley in an effort to develop a plan as quickly as possible to give flooded homeowners the information they needed to make personal decisions about rebuilding their flooded homes. The purpose of the MREFPP is to develop a flood risk reduction project that can pass the flood of record. Project objectives include protecting as many homes as possible, minimizing the Project footprint, and minimizing impacts to unprotected features. Significant stakeholder involvement was solicited in obtaining the Project constraints which include, but are not limited to:

- Minimizing property acquisitions.
- Minimizing impacts to the Northwest Area Water Supply (NAWS) water pipeline.
- Incorporating 3 feet of additional feature height to account for uncertainty and superiority.
- Maintaining functionality of critical transportation routes during a flood.
- Limiting observed (2011) increases to water surface elevations at the WTP.
- Maintaining key community resources.

After delivery of the Preliminary Engineering Report (PER) covering the developed portions of the basin, the SRJB shifted their attention to the rural reaches of the Mouse River Valley, where land use and flooding characteristics vary greatly from the developed areas. The resulting *Rural Flood Risk Reduction Alternatives Evaluation* (reference [1]) was completed by Barr in May 2013.

1.2 Preliminary Engineer's Report (PER)

The Mouse River has a history of flooding, including the record-breaking flood of 2011. The 2011 flood overwhelmed most levees and flood-fighting efforts along the entire reach of the Mouse River through North Dakota, causing extensive damage to homes, businesses, public facilities, infrastructure, and rural areas. Over 4,700 commercial, public, and residential structures in Ward and McHenry counties sustained an estimated $690 million in damages.

After the flood of record, residents of the Mouse River Valley requested plans for a project that could reduce the risk of flooding from events of similar magnitude. A team led by Barr Engineering Co. was selected by the NDSWC to develop plans for new flood risk-reduction features that would accommodate flows up to 27,400 cfs. The resulting *Preliminary Engineering Report (PER)* (reference [2]) was completed on February 29, 2012, and adopted by Minot through City Council action in April 2012 and adopted by the SRJB in December 2013. It serves as the master plan for flood risk reduction measures within Minot and surrounding communities.

1.3 Basis of Design Report Purpose and Scope

The purpose of this basis of design report (BDR) is to summarize Project design efforts for Phase WC-1. The BDR report text contains sufficient detail to describe the components and configuration of the design, along with rationale for decisions and recommendations associated with design development. Detailed supporting documentation is provided in the appendices, including review comments and responses, supporting reports and documents, design computations, and construction drawings. The BDR also contains relevant hydrology and hydraulic analysis for the Mouse River including risk and uncertainty, superiority and impacts analysis.

The SRJB retained the services of the Barr team to assess the existing levee system, design modifications, and prepare the design documentation necessary for flood system modifications that will meet the requirements for USACE Section 408 permissions and an array of state, local, and federal permits and support future revisions to the Federal Emergency Management Agency's (FEMA) Flood Insurance Rate Maps once future MREFPP phases are implemented.

1.4 Prior Reports and Studies

Efforts to address flooding problems in Tierrecita Vallejo and Minot started in the 1930s and have resulted in the implementation of several flood risk reduction projects. A brief summary of key past studies and resulting projects follows.

- **1930:** A USACE report recommended a study of flood control alternatives including reservoir storage near Foxholm, North Dakota, and a floodway through Minot.
- **1935:** A follow-up to the 1930 report conducted by the USACE concluded that neither reservoir storage nor local protection provided sufficient benefits to permit federal participation in flood risk reduction projects.

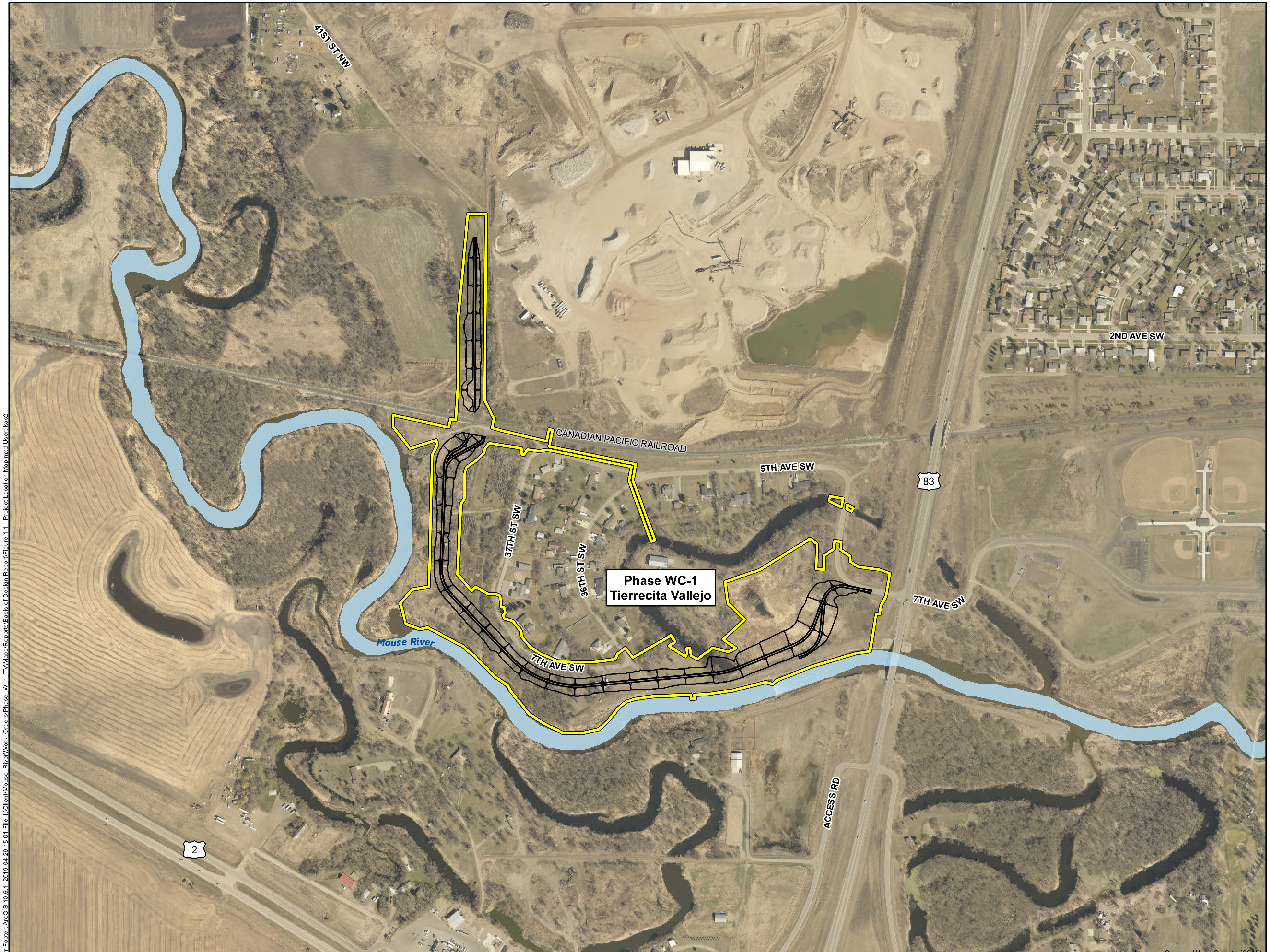
- **1957:** Additional studies were recommended in a USACE examination of the Mouse River in the vicinity of Minot.
- **1965:** The Flood Control Act (Public Law [P.L.] 89-298) authorized channel modifications and enlargement at Minot.
- **1969:** The USACE issued a report and draft environmental impact statement (EIS) which included a recommendation for early construction of the channel modifications and enlargement at Minot.
- **1970:** Senate (June 25) and House (July 14) Public Works Committee resolutions authorized the channel modifications and enlargement features at Minot, as recommended in the 1969 USACE report.
- **1971–1979:** Channel enlargements within Minot were designed for 5,000 cfs flow.

The SRJB is the sponsor for the MREFPP. The Project, which will be implemented in multiple phases, is based on the PER and will include alignments for new levees and other flood risk reduction measures (Figure 1-1). The SRJB is pursuing other measures to reduce the risk of flooding in the rural reaches of the valley including implementation of the structure acquisition, relocation, or ring dike (StARR) program and advocating for changes in reservoir operations.

A thorough review of the documents was conducted to gain a better understanding of the original design assumptions, subsequent project improvements, monitoring data, and current issues surrounding the flood risk reduction project. Below is a summary of known USACE documentation for the Tierrecita Vallejo levee project:

- *Design Memorandum No. 1*, July 1972, (reference [3])
- *Design Memorandum No. 2*, *Interior Drainage*, December 1973, (reference [4]
- *USACE As-Built Drawings – Burlington to Minot Improvements Stage 1 – Tierrecita Vallejo,* September 1991, (reference AA)
- *USACE As-Build Drawings – Channel Improvement – Reach E-1,* January 1979, (reference BB)
- U.S. Army Corps of Engineers, St. Paul District, *Flood Control Project, Souris River, Minot, North Dakota, Operation and Maintenance Manual*, 1981, (reference DD)
- U.S. Army Corps of Engineers, St. Paul District, *Flood Control Project, Souris River Basin, Burlington to Minot Stages 1-4, Ward County, North Dakota, Operation and Maintenance Manual,* 1993, (reference DD)
- *USACE Routine Inspection Report*, 2017, (reference CC)

The following list contains the sources of other data used in the development of this report. More information about the specific information used from these sources is provided throughout this report.



Phase WC-1 Construction Limits

0 250 500 1,000
Feet

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Figure 1-1

PROJECT LOCATION MAP

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

- *Flood Insurance Study, Ward County, North Dakota and Incorporated Areas,* FEMA, February 15, 2002, (reference [5])
- *City of Minot FIS, Flood Insurance Study Report Data,* Swenson Hagen & Company and Houston Engineering, June 3, 2002, (reference [6])
- *Mouse River Enhanced Flood Protection Plan: Preliminary Engineering Report,* Barr Engineering Co., February 29, 2012, (reference [2])
- *Mouse River Enhanced Flood Protection Plan: Erosion and Sedimentation Study,* Barr Engineering Co., January 18, 2013, (reference [7])
- *Mouse River Enhanced Flood Protection Plan: Hydrologic and Hydraulic Modeling Report,* Barr Engineering Co., April 30, 2013, (reference [8]).
- *Mouse River Enhanced Flood Protection Plan: Rural Flood Risk Reduction Alternatives Evaluation,* Barr Engineering Co., May 1, 2013, (reference [1])

1.5 Existing Flood Risk Reduction Systems Background

Numerous federal flood risk reduction projects have been constructed in the Mouse River Valley over the last 40 years to reduce flooding for developed areas along the Mouse River. These projects generally consist of upstream multi-purpose reservoirs, levees, channel modifications, and pump stations.

1.5.1 Existing System Authority

The existing flood risk management projects along the Mouse River (the Souris River Basin Project) were developed by the USACE over several years in three separate Congressional actions:

- Flood Control Act of 1965 (P.L. 89-298)
- USACE Chief of Engineers in House Document 286, 87th Congress, 2d Session
- USACE Chief of Engineers in House Document 321, 91st Congress, 2d Session

1.5.2 Existing System Description

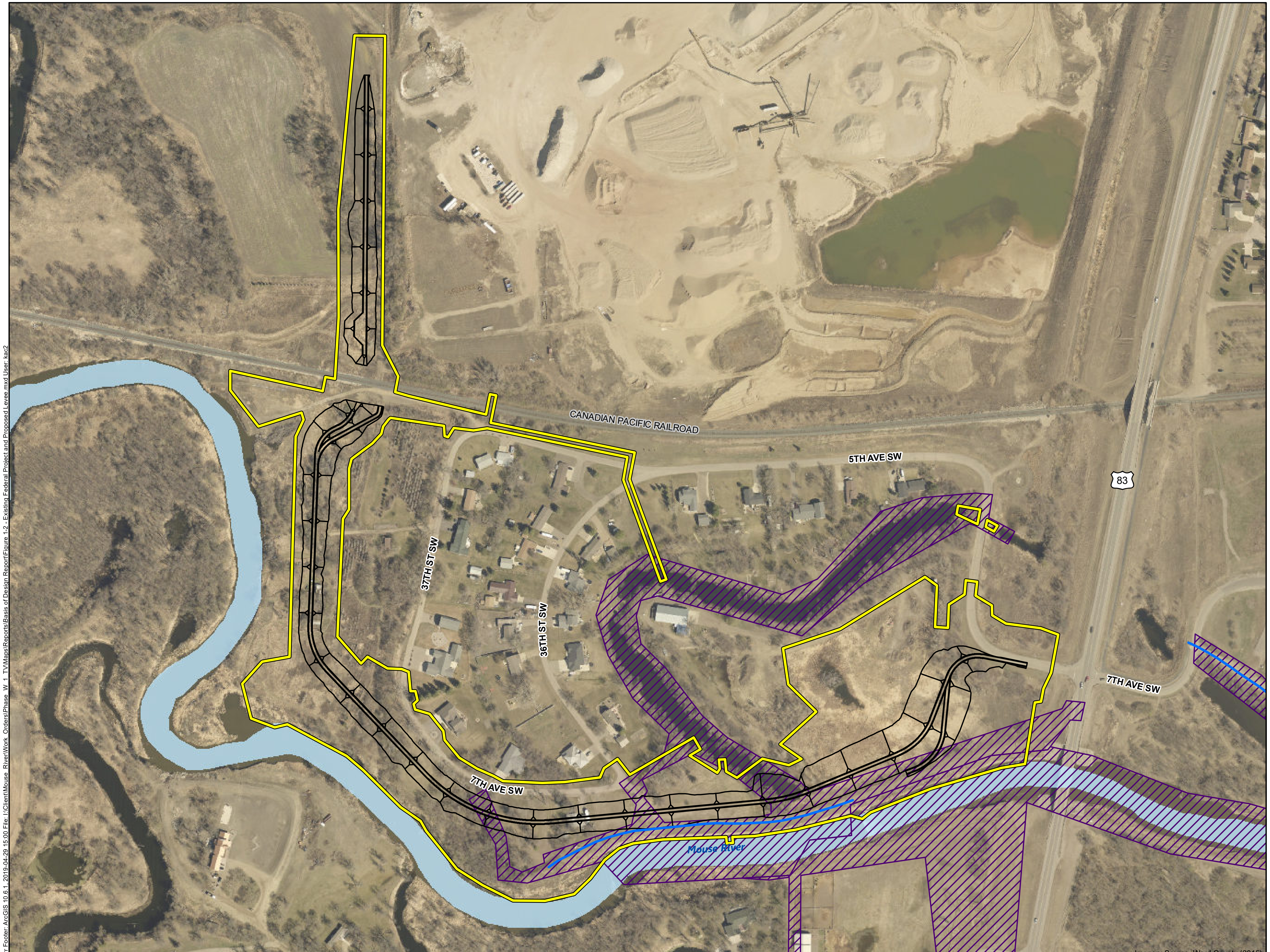
Flood risk reduction projects within the Mouse River Valley were constructed in three phases. The first phase was a channel modification project in Minot. The second phase was a levee project in Velva. The third phase included multiple features:

- Flood storage in Alameda and Rafferty Dams in Saskatchewan
- Construction of a gated spillway and flood storage at Lake Darling Dam
- Levees at Sawyer, Renville County Park (Mouse River Park), and six subdivisions between Burlington and Minot
- Structural and nonstructural measures for rural residents along the Souris River

- Modification of U.S. Fish and Wildlife Service (USFWS) structures in the Upper Souris and J. Clark Salyer National Wildlife Refuges
- Development of a flood warning system

Project features within Tierrecita Vallejo were constructed before and separate from any other components and designed to accommodate flows up to 5,000 cfs. The Project within Minot is operated and maintained by Minot. Tierrecita Vallejo Project features (Figure 1-2) consist of the following elements:

- 48 acres protected by the levee involves mainly low-density suburban residential property.
- Overall length of levee is approximately 1,670 feet with an additional 1,740 feet of emergency levee.
- Interior drainage is conveyed outside the Tierrecita Vallejo levee with a gatewell and 60-inch gravity outlet passing under U.S. Highway 83. A portable pump is mobilized to evacuate excess runoff and seepage that accumulates in the storage area.
- A gatewell at the penetration through the existing levee at the Mouse River houses a sluice gate to control stormwater flow through the existing oxbow. During flood periods, the sluice gate is to be closed to prevent backflow into the oxbow areas. Elevation at which to close the gatewell is 1553.2 (NAVD88).
- Stormwater drains to an existing oxbow through a system of ditches and culverts. Initial damage elevation is approximately 1557.2 (NAVD88).



Phase WC-1 Construction Limits

Proposed Levee Alignment

Exising Federal Levee Centerline

Existing Permanent USACE ROW

0 150 300 600

Feet

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Figure 1-2

EXISTING FEDERAL PROJECT AND PROPOSED LEVEE

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

1.6 Phase WC-1 Description

Phase WC-1 of the Project will provide flood risk-reduction for the Tierrecita Vallejo subdivision. The configuration of the proposed system is similar to that shown in the PER master plan.

Significant modifications to the existing levee segments are required to (1) accommodate the design flow increase from 5,000 to 27,400 cfs, (2) meet current USACE design criteria to obtain USACE Section 408 permission, and (3) obtain future FEMA accreditation for the flood risk management system. These modifications generally consist of the following:

- New levee alignment along the Mouse River from Station 10+00F to Station 57+00F.
- A stoplog railroad closure at Canadian Pacific Railroad with floodwall sections at Station 20+00F.
- Tierrecita Vallejo Gatewell control structure within the proposed levee at Station 45+20F to convey flow through the existing oxbow from the Mouse River.
- Existing river grade control structure (identified as Control Structure No. 24 in October 1976 USACE as-built plans) within the Mouse River and overbank excavation at Station 49+00F.
- Modifications to U.S. Highway 83 Gatewell (identified as Gatewell No. 2 in September 1991 USACE as-built plans) located within the oxbow to convey flow through the existing oxbow from the Mouse River to the MREFPP Phase MI-2 Bark Park Gatewell/Pump Station.
- Levee ramps for access, maintenance, and inspection at locations identified in the construction drawings.
- A Northwest Area Water Supply (NAWS) transmission line and water main upgrades for crossings within the USACE right-of-way at Station 55+50F.
- Overbank excavation adjacent to the north bank of the Mouse River from Station 30+00F to 57+00F.
- Bank erosion protection for the Mouse River channel and flood risk-management features at various locations.
- Municipal infrastructure modifications and improvements including water main, storm sewer, and street reconstruction.
- A dual purpose exploration for inspections and levee fill trench for seepage mitigation.

Below is a summary of corrective measures and work items identified during the USACE's 2017 routine inspection. These work items are in the planned system improvements within Phase WC-1.

- Unwanted vegetation growth
- Structure corrections

- Encroachments
- Bank Erosion Repair
- General Infrastructure

1.7 Feature Height Design Summary

Project features were designed to reduce the risk of flooding from a flood event similar to the 2011 flood of record by incorporating risk and uncertainty analysis and system superiority Figure 1-3 illustrates key design terms and elevations for levees and floodwalls. These terms are discussed further below.

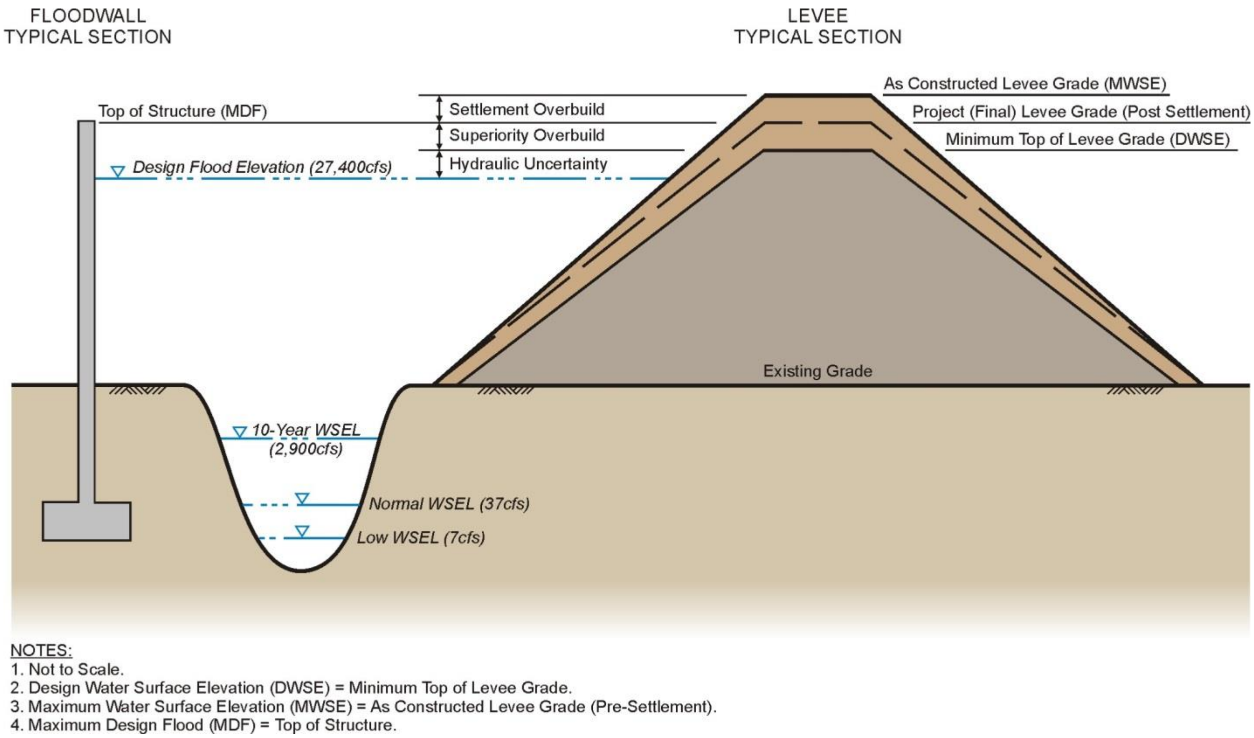


Figure 1-3 Design elevations for levees and floodwalls

Definition of terms for establishing design elevations for levees:

- **Design Flood Elevation** is the modeled water surface elevation for the 2011 flood hydrograph under with-Project conditions. The USGS measured a peak discharge of 27,400 cfs at Broadway bridge in Minot.
- **Hydraulic Uncertainty** accounts for natural variability and model parameter uncertainty associated with the Design Flood Elevation. Hydraulic Uncertainty defines the additional feature height needed to provide 95-percent probability that the design flood will not exceed the Minimum Top of Levee Grade. The calculation of Hydraulic Uncertainty is defined in Section 3.0 .
- **Design Water Surface Elevation (DWSE)** is the Minimum Top of Levee Grade**.** The DWSE is defined as the Design Flood Elevation plus Hydraulic Uncertainty.

- **Superiority Overbuild** is material added to some portions of the levee to control the location of overtopping of the levee system such that when it occurs it does so at a predetermined location. The overtopping location is typically at the downstream end of a levee system as it ties into high ground. Superiority overbuild varies by location. Superiority Overbuild heights are defined in Section 3.0 .

- **Project (Final) Levee Grade** is the Minimum Top of Levee Grade plus Superiority Overbuild. It is the anticipated finish grade of the levee system after long-term settlement of the levee.

- **Settlement Overbuild** is additional material placed on top of the levee when it is initially constructed to allow for settlement of the levee top to the desired Project (Final) Levee Grade. The anticipated amount of settlement for the levee systems is defined in Section 2.0.

- **As Constructed Levee Grade** is the Project (Final) Levee Grade plus Settlement Overbuild. The construction drawings will instruct the contractor to build the levee to this elevation.

- **Maximum Water Surface Elevation (MWSE)** is the top of the As Constructed Levee Grade. The MWSE is a factor in for design of levees as described in Section 2.0.

- **Top of Structure** is the as constructed top of a floodwall or closure structure. Floodwall and closure structure designs are described in Section 7.0.

- **10-year Water Surface Elevation** (10-year WSEL) is the water surface elevation for the 10-percent Annual Exceedance Probability flood event.

- **Normal Water Surface Elevation** (Normal WSEL) is the discharge with a 50-percent chance of daily exceedance. The Normal WSEL is used in the Geotechnical Analysis (Section 2.0). The calculation of Normal WSEL is discussed in Section 3.0.

- **Low Water Surface Elevation** (Low WSEL) is the discharge with a 75-percent chance of daily exceedance. The Low WSEL is used in the Geotechnical Analysis (Section 2.0). The calculation of Low WSEL is discussed in Section 3.0 .

1.8 Base Map Development and Project Datum

Data used to support the design and preparation of base maps is described in the following paragraphs. Each data set was projected to the appropriate horizontal datum (North Dakota State Plane, North Zone, U.S. feet, NAD83) and vertically adjusted to the NAVD88 vertical datum to provide a uniform base map along the entire length of Phase WC-1. All elevations are presented in NAVD88 unless otherwise noted. The conversion from NGVD29 to NAVD88 for the Project area is NGVD29 + 1.24 feet = NAVD88 (reference [9]). After projections and vertical adjustments were made, the resulting mapped data were compared to verify map accuracy. The Project datum is defined as follows:

Horizontal Datum: North Dakota State Plane, North Zone, U.S. feet, NAD83
Vertical Datum: NAVD88

1.8.1 Topographic Data and Features Survey

KBM, Inc. performed an aerial LiDAR survey of the Mouse River area within Minot in May 2014. The LiDAR topographic data states a root mean square error (RMSE) of 0.06 feet (0.72 inches), a maximum absolute error of 0.12 feet (1.44 inches), and a median absolute error of 0.02 feet (0.24 inches). Points classified as bare earth along with water edge 3D breaklines developed by KBM were used to develop the terrestrial surface within Minot. According to the vendor (KBM) the LiDAR was collected at a rate of 3–4 points per square meter within Minot. This data was used to create a surface model for design at a 3-foot resolution for Minot.

Outside of Minot, hydro breaklines were not available. Soundings and historical cross section data were used to define bathymetry along the entire modeled reach of the Mouse River by combining soundings and cross sections with the LiDAR surface. Bathymetry data was interpolated between cross sections of soundings and historical cross section data.

Ackerman-Estvold completed numerous detailed surveys to supplement LiDAR information and to acquire additional data specifically needed to develop the Project design. Additional bathometry within the cutoff meander, detailed topography of the railroad crossing, and verification of the LiDAR surface along the centerline of the proposed alignment was completed in December 2018 and January 2019.

Bathymetry surveys collected by the USACE, Houston Engineering (HEI), and Ackerman-Estvold formed the basis of the channel bathymetry.

The LiDAR topographic data, channel bathymetry surveys, and field topographic surveys were combined to create a single surface using the NAVD88 vertical datum.

The reasonableness of the project surface was verified through a QA/QC process. The surface was compared to survey data collected by Minot along the tops of existing levees. Over 90 percent of the top of levee survey points are within 0.4 feet of the LiDAR based digital terrain model and nearly 70 percent are within 0.2 feet. This is within the accuracy of a 3-foot grid terrain model. The few locations where there were larger differences were isolated and they did not suggest issues with the LiDAR surface. Cross sections cut from the new surface were compared to cross sections from previous models cut from other surfaces and differences were investigated.

1.8.2 Digital Terrain Model (DTM)

A digital terrain model (DTM) was compiled by merging LiDAR topographic data, field topographic data, and bathymetric survey information for use in designs and drawings. Autodesk Civil 3D software was used to process all LiDAR and bathymetric survey data and to create the DTM.

1.8.3 Parcel Data

Minot maintains a database of parcel information (including Tierrecita Vallejo) that has been supplied to the consulting teams. The parcel information is approximate in nature and should not be used to determine legal property boundaries.

In areas adjacent to project features, property surveys were completed through the Phase WC-1 reach to determine legal property boundaries. Property corners were recovered along the reach and property lines and parcel boundaries were established by North Dakota professional land surveyors in accordance with generally accepted practice and state law.

Easements for the existing federal project were retraced by conducting deed research at the Ward County courthouse. Generally, the recorded permanent easements for the existing federal project appear significantly smaller than the right-of-way indicated on the existing federal project recorded as-built plans. Temporary construction easements recorded for the existing federal project construction are generally consistent with the right-of-way indicated on the as-built plans for the existing federal project.

1.8.4 Franchise Utilities

The location of existing privately owned utilities such as electric, gas, cable, and telephone have been acquired and in the base maps. Coordination with Franchise utilities continues for relocation of existing lines and design proposed alignments within the Phase WC-1 corridor.

1.8.5 Wetland Delineation

Wetlands within the construction limits of Phase WC-1 were identified and delineated in the field in September/October 2018. Field surveys to determine the ordinary high-water level (OHWL) were completed in September/October 2018 and reports are in the appendices of this report. Delineation data was provided and integrated into the base map information.

1.9 Design Approach Summary

Geotechnical, hydrologic and hydraulic, civil, structural, and environmental design methods have been developed in accordance with the methods and references cited in USACE engineering manuals, technical letters, regulations, and other document types. The following report sections briefly describe the parameters and methods for the design. Detailed design calculations and supporting documentation are in the following appendices:

- Appendix A Agency Technical Review Report (not included in 60% Design Submittal)
- Appendix B Geotechnical Analysis
- Appendix C River Hydrology and Hydraulic Analysis
- Appendix D Interior Drainage Analysis
- Appendix E Civil Design
- Appendix F Structural Design
- Appendix G Pump Station Design (not included as part of Phase WC-1)
- Appendix H Permitting and Regulatory (not included as part of Phase WC-1)

- Appendix I Real Estate Summary
- Appendix J Opinion of Probable Cost
- Appendix K Construction Drawings
- Appendix L Project Manual
- Appendix M Independent External Peer Review (IEPR) (not included in 60% Design Submittal)
- Appendix N Project Design Guidelines
- Appendix O Environmental Studies
- Appendix P Operations and Maintenance Manual (not included in 60% Design Submittal)
- Appendix Q Quality Assurance and Quality Control
- Appendix R Risk-Informed Evaluation and Design Assessment

2.0 Geotechnical Analysis

2.1 Introduction

As part of the MREFPP – Phase WC-1 existing levees will be removed to a grade similar to the surrounding landside and riverside ground surface. New levees will be constructed along the Mouse River within the general corridor proposed in the *Preliminary Engineering Report* ([PER], reference [10]). A railroad closure, interior drainage components, and a gatewell will also be constructed.

To support the design of these elements, a subsurface investigation, laboratory testing, and geotechnical engineering analysis were performed. The geotechnical components of the Project are detailed in the sections below.

Geotechnical engineering components include the following:

- Development of a geologic profile
- Evaluation of soil stratigraphy based on field investigations
- Evaluation of soil parameters for settlement, seepage and stability modeling and analysis
- Evaluation of groundwater levels for seepage and stability modeling and analysis
- Modeling of seepage for the proposed levee system
- Underseepage mitigation
- Modeling of stability for the proposed levee system
- Evaluation of settlement for the proposed levee system
- Evaluation of bearing capacity for the Project structures, and
- Evaluation of settlement for the Project structures

All elevations in this section use the NAVD88 vertical datum.

2.2 Site Geology

2.2.1 General Geology Review

The Project area is near Minot along the Mouse River. It extends from north of the Canadian Pacific Railroad (the west/upstream end) to the U.S. Highway 83 Bypass (the east/downstream end). The site lies within the Williston Basin, one of the largest structural troughs in North America (reference [11]).

The area lies at the boundary between the Souris (Mouse) Plain physiographic region and the Glaciated Plains physiographic region (reference [11]). Most of the exposed surface sediment in Ward County was deposited by Late Wisconsinan Age glaciers about 25,000 to 12,000 years ago. Most of Minot lies within the Mouse Valley, which is a large proglacial lake spillway (reference [12]). Local relief outside of the floodplain is fairly gentle and numerous closed basins impound runoff water for significant periods (reference [11]).

Surficial geology mapping (reference [13]) indicates that subglacial glacial till surrounds the Mouse River Valley. The glacial till is described as poorly sorted, unbedded gravel, sand, silt, and clay soils, with cobbles to boulders. The average thickness of glacial till in Ward County, as determined by test-hole drilling is 165 feet. The thickness of the glacial till lessens to the southwest. The near-surface glacial till soils in the Minot area are considered medium-stiff to stiff. Stiffness generally increases with depth (reference [12]). The glacial till and glacial sediment reworked by flowing water has been named the Coleharbor Formation.

The surficial soils are underlain by bedrock of the Tertiary Age Fort Union Group. The Fort Union Group consists of Sentinel Butte, Bullion Creek, Ludlow, and Cannonball Formations (ordered from youngest to oldest). These rock units consist primarily of shale, sandstone, and siltstone, with lignite coal seams (reference [12]). At a few isolated locations within Ward County, the Fort Union Group is exposed in outcrops at the surface or along stream channels.

Hydrogeological studies of the area indicate that the majority of the groundwater is typically found in the Quaternary deposits. These consist of buried sand and gravel aquifers deposited in the Mouse River Valley, within glacial meltwater channels, or at stratigraphic transitions between glacial till layers. The depth, thickness, and distribution of these aquifers vary widely but serve as the major water supply for Minot and the surrounding communities. Groundwater from sand and coal beds within the bedrock typically has small yield and is generally not used for human consumption (reference [12]).

2.2.2 Souris River Valley Geological Review

The Mouse Valley itself is filled primarily with alluvial deposits. The alluvial deposits are described as fluvial channel and overbank sand, gravel, silt, and clay deposits. The alluvial sediments are commonly 100 feet or more in thickness within the floodplain. The glacial till and underlying Fort Union Group bedrock is typically not present within the river channel through the Project areas, and the levees will be founded on alluvial soils within the overall Mouse Valley, with the exception of the northern tie-in north of the Canadian Pacific Railroad. The subsurface below northern tie-in on the west end of the levee is mapped as terrace deposits and is likely the source of the materials being quarried north of the Canadian Pacific Railroad lines through this reach. During the subsurface explorations, weathered shale and thin coal seams of the Fort Union Group were encountered within the upper 50 feet below grade in this area.

The potential soil conditions were evaluated by reviewing published geological data, aerial photography, and previous boring logs for the Project, as well as performing a site visit prior to performing the investigations. These studies were used to evaluate for site features such as oxbows, point bars, terrace deposits, tributary/coulee deltas, or landslide areas which may affect seepage and stability of the levee.

Four primary sources of site geology were reviewed to evaluate site conditions:

- North Dakota Geological Survey – Geologic Investigation No. 46, Geology of the Minot Area (reference [13])
- U.S. Geological Survey Professional Paper 325, Geology of the Souris River Area North Dakota (reference [14])
- North Dakota Geological Survey – Bulletin 50-Part 1, Geology of Ward and Renville Counties (reference [11])
- Publicly available aerial imagery

The Geology of the Souris River Area study discussed the types of subgrade materials in the general area of the site, including detailed characteristics of the alluvial soils at "type locations" within the greater Mouse River area (reference [14]). The Geology of Minot study includes a map through Minot defining mapped regions of surficial geology, including detailed Quaternary soil types (reference [13]).

The majority of the levee footprint is mapped as recent alluvial deposits consisting primarily of sand, silt, and clay. The majority of the river margins are mapped as glacial till soils. Along the margins of the valley, particularly the cut-banks, landslide and colluvial fan deposits have been mapped. The landslide and colluvial fan deposits consist of reworked sand, silt, and clay soils of the glacial till or underlying bedrock (reference [13]). The landslides are due to loss of lateral confinement from the eroded materials and are thought to have occurred rapidly following carving of the Mouse River Valley (reference [12]). Some slope instability which is continually observed is caused by groundwater seepage in alternating coarse- and fine-grained layers in the Fort Union Group (reference [11]).

In other areas along the margin of the valley, river terrace deposits have been mapped. These consist of planar beds of sand and gravel with abundant cobbles and boulders (reference [13]). The terrace deposits (and potentially recent fill materials) are located beneath the levee footprint and the northern tie-in north of the Canadian Pacific Railroad.

Identified features of interest and their mapped/observed locations are discussed below:

- Oxbows are former river channels which are stranded from the main channel when the river cuts through a thin neck of a meander loop. Oxbows may contain a higher percentage of sand within the old flow channels and provide preferential seepage pathways. Aerial imagery indicates there is one main oxbows located just west of the U.S. Highway 83 Bypass. Indications of other oxbows/former river channels are apparent west of the levee and primarily south of the Canadian Pacific tracks. These appear to be west of the primary levee alignment and do not appear to extend beneath the levee. The main oxbow west of U.S. Highway 83 Bypass continues beneath the highway through culverts and extends through the Bark Park Pump Station within Phase 2-3 of the levee project to the east.
- Point bars are depositional features located on the deposition side of a meandering river channel. Point bars can consist of coarser sand and gravel deposits which could serve as a preferential seepage pathway beneath the levees. Two larger point-type features were identified in aerial

imagery along the reach, one near the south-west corner of the reach and one in the south-central area just west of the main oxbow. However, the boring logs did not indicate high concentrations of gravel associated with the potential point bars.

- Terrace deposits are described as planar bedded sands and gravels with abundant cobbles and boulders deposited in terrace-formed bars along the walls of the river. The deposits were laid down by meltwater during the last deglaciation of the area. The Geology of Minot study (reference [12]) indicates the gravel pit area north of the Canadian Pacific tracks is on a surficial terrace deposit.
- Coulee/tributary deltas form from sediment deposited by intermittent stream from lesser tributary streams entering the Mouse River. These deposits for fan-like features similar to deltas and are typically comprised of coarser sediments than found beneath the valley floor. Based on review of the geologic maps and aerial photography, there does not appear to be any large streams or coulees entering the river along the Project reach.
- Landslides and colluvial deposits are mapped on the southern side of the river opposite of the levee. These are not within the areas of the Project.

2.2.3 Review of Field Conditions and Investigation Locations

In October 2018, Barr Engineering lead geotechnical engineer and Ackerman Estvold lead civil engineer traveled to the site to review field conditions, review features identified by mapping, and identify soil boring and cross-section locations. Winter conditions had not set in yet during the field visit, allowing for a good view of the exposed ground surface and river banks.

The following is a summary of observations of note made during the field visit:

- The site visit identified the presence of the U.S. Highway 83 Bypass oxbow, low-head control structure in the river, and associated gatewell. The majority of the alignment between the Canadian Pacific Railroad and the main oxbow feature extended through residential areas of various use. One residential structure was still present in the vicinity of the levee footprint at the time of the field visit as well as a lesser developed rural property with a few buildings. There apparently were other residences that were removed from within the footprint of the levee. The presumed basements had been backfilled level with surrounding grades. Investigations were placed along the levee in this area, but not specifically among the removed structures. The exploration trenches for the levee will locate buried structure foundations, floor slabs, and utilities much more effectively than small-diameter borings.
- In addition, the gravel pit site was reviewed as a potential tie-in point for the western end of the levee. The current quarrying operations were located in the eastern part of the property, away from the potential tie-in on the western property boundary. However, the western portion of the gravel pit property had apparent fill materials in the existing higher ground. Borings were placed in the vicinity of the apparent fill materials to assess their extent and character. An alternative set of borings were placed along an alternative levee alignment and tie-in point in the event that use of the gravel pit property was not desired for the project.

- Site investigation locations were placed along the levee alignment. Cross-section locations were recommended at the areas where the river channel was nearest to the proposed levee alignment, as well as near the main oxbow for the reach, and well as for the west tie-in/gravel pit.

- Besides placing borings along these features of note, investigation locations were also placed at regular intervals across the Project proposed levee alignments to evaluate if there were other soil conditions not associated with identified features which may affect levee seepage or stability. Continuous sampling was generally performed in the soil borings to evaluate for the presence of thin soil layers and sand and gravel seams. DMT testing was also performed to evaluate settlement potential for the main oxbow and other areas of the proposed alignment.

2.3 Field Work

The main field investigation to collect data for the geotechnical analyses was performed December 2018. The site investigation consisted of soil borings, in-situ testing, and instrumentation installation. Geotechnical investigation locations completed in along the levee alignments are shown in Figure 2-1.

Figure 2-1 Geotechnical Investigation at Levee Alignments

Twenty-three conventional soils borings were completed. Four flat plat dilatometer tests (DMT) were performed. The exploration locations for the field investigation are numbered roughly sequentially across the site, starting with the lowest exploration number at the western end of the Project area and finishing with the highest exploration number at the eastern end of the Project area.

A review of historical borings was also done to fill in gaps between the current investigations. These historical boring logs have been provided in Appendix B.

Information gathered during the field work was used to develop an along-levee profile and six geotechnical cross-sections along the study area for the evaluation of under seepage, seepage, levee stability, and settlement. The cross-section locations are shown in Figure 2-2. These were selected to include critical locations where the levee is near the existing river channel and to spatially cover the work area defined for Phase WC-1 of the Project.

2.3.1 Soil Borings

A total of 23 borings were performed for the Project. The test locations were selected to cover the Phase WC-1 area, particularly along the cross-sections selected for analysis. Hollow-stem auger techniques were primarily used to advance the borings above the water table; mud-rotary methods were used below the water table where needed to counteract soil heave and the potential of measuring artificially low SPT N-values from disturbance. Soil boring logs are provided in Appendix B.

Because under seepage was anticipated to be a concern, continuous sampling of the borings was done to evaluate whether sand seams and interlayered deposits were present. Most borings were completed with sampling throughout the full depth. Sampling in the soil borings consisted of split-spoons, thin-wall Shelby tubes, and modified California brass-lined samplers. The samples obtained from drilling were sealed in the field for laboratory testing.

2.3.2 Previous Soil Borings

Information from several soil borings previously taken for the existing levee and bridges was reviewed and compared to borings taken for Phase WC-1. These previous boring logs were provided to evaluate the potential for soil variability and provide soil information where current borings were not performed. These previous borings are provided in Appendix B.

2.3.3 Vibrating Wire Piezometers and River Levels

Three vibrating-wire (VW) piezometers were installed to monitor pore-water pressure within the soils at the site. The piezometers tips were installed approximately 30 to 36 feet below existing grades in the selected soil borings to evaluate the fluctuation of the water levels. Data loggers were placed at each VW piezometer location to record daily readings during the design period. Total head readings are in Appendix B. Water levels from the VW piezometers were also compared to water levels in the river to evaluate whether a direct connection exists between groundwater and the river.

2.3.4 DMT Soundings

Flat-blade dilatometer (DMT) testing was performed at four of the soil boring locations. Testing was performed using the soil boring rig to advance the DMT test equipment. The test locations were selected to spatially cover the work area defined for Phase WC-1 of the Project.

The processed DMT data are in Appendix B. The constrained modulus derived from DMT testing was ultimately used to estimate settlement of the proposed levee sections where DMT testing was performed.

2.4 Laboratory Testing

Laboratory testing was performed by Soil Engineering Testing (SET) of Richfield, Minnesota or Materials Testing Services of Minot, North Dakota and included consolidation, triaxial shear, direct shear, unconfined compression, hydraulic conductivity (permeability), and index-property testing. Intact Shelby tube and modified California samples were used in strength and permeability testing of the clay soils and as much as possible for sand soils to minimize remolding of test specimens. The laboratory reports for the testing program are in Appendix B.

2.4.1 Index Properties

Index property testing, consisting of moisture content, unit weight, grain size analysis, and/or Atterberg Limit determinations, was performed on numerous samples from each soil boring. These index properties are useful when characterizing the soil. The plasticity index (PI) and clay-size fraction (CF) are commonly used to estimate soil behavior.

2.4.2 Shear Strength

2.4.2.1 Direct Shear Testing

Direct shear testing was performed primarily on sand, silty sand, and clayey sand soils to measure the drained failure envelope (i.e., internal angle of friction). Tests were performed on intact Shelby tube or Modified California brass liner samples or samples remolded to specific unit weight.

2.4.2.2 Laboratory Undrained Shear Strength Testing

Unconsolidated undrained (UU) triaxial shear testing was completed on selected samples to determine the undrained shear strength properties of the clay soils in the borings.

2.4.3 Compressibility

2.4.3.1 Laboratory Consolidation Testing

Consolidation tests were performed on selected samples to estimate soil compressibility, stress history (i.e., over-consolidation ratio), and evaluate settlement potential of the clay soils in the borings.

2.4.3.2 Compressibility from DMT Testing

The DMT data was used to obtain the one-dimensional constrained modulus M at all test locations as an initial estimate of settlement. This provides an in-situ compressibility profile for use in estimating levee settlement.

2.4.4 Hydraulic Conductivity (Permeability)

2.4.4.1 Hydraulic Conductivity from Laboratory Testing

Hydraulic conductivity tests were performed on selected samples to determine the permeability of the material for seepage analysis. The laboratory hydraulic conductivity results represent only vertical permeability, with water forced to flow from the top face to the bottom face of the sample. The hydraulic conductivity testing was performed in both the vertical and horizontal direction to determine anisotropy for design.

2.5 Groundwater Monitoring

Groundwater conditions were evaluated using observations from soil borings and installation and monitoring of the VW piezometers.

It is anticipated that groundwater plays a significant role in the stability of the riverbank slopes. This makes piezometer data an important component of the geotechnical analysis used for modeling and design of the levee system.

2.5.1 Groundwater Observations in Soil Borings

Groundwater was observed during and at the completion of drilling at boring locations. Typically, stabilization time is required for groundwater readings in boreholes (particularly in low-permeability cohesive soils) to accurately reflect static water levels. Observed groundwater levels are provided in Appendix B, but these should only be considered a general indication of water levels.

2.5.2 Groundwater Levels from VW Piezometer Monitoring

All VW piezometers are associated with a specific boring location. The piezometers were installed in the selected boreholes with their tips approximately 30 to 36 feet below existing grade. The water levels from the VW piezometers from January 2019 are provided in Appendix B. The water levels from the VW piezometers should provide the best indication of groundwater conditions over time.

2.6 Soil Stratigraphy and Parameters

Current understanding of site stratigraphy is based on review of available data from previous investigations, field investigations, laboratory testing, and knowledge of the geology of the Mouse River Valley. The site stratigraphy consists of a thin layer of topsoil, typically followed by shallow fill and existing levee materials underlain by sand with high content fines, sand with low content fines, lean silty/sandy clay, and fat clay soils. The layers of native soils appear to be highly interbedded, with thin layers interspersed within the main soil types. An along-levee soil profile was inferred from the current and previous explorations. The soil profile developed depicts a wide variability in layer thickness and order along the levee alignment and therefore it is not feasible to use one consistent soil profile for the Project.

2.6.1 Model Geometry (Cross-sections)

Six cross-sections for the Phase WC-1 levee were analyzed using GeoStudio 2012. The evaluated cross-sections were given the names of the boring locations at each transect across the levee. The cross-sections were generally cut perpendicular to the river channel and proposed levees. Locations of the cross-sections are shown in Figure 2-2.

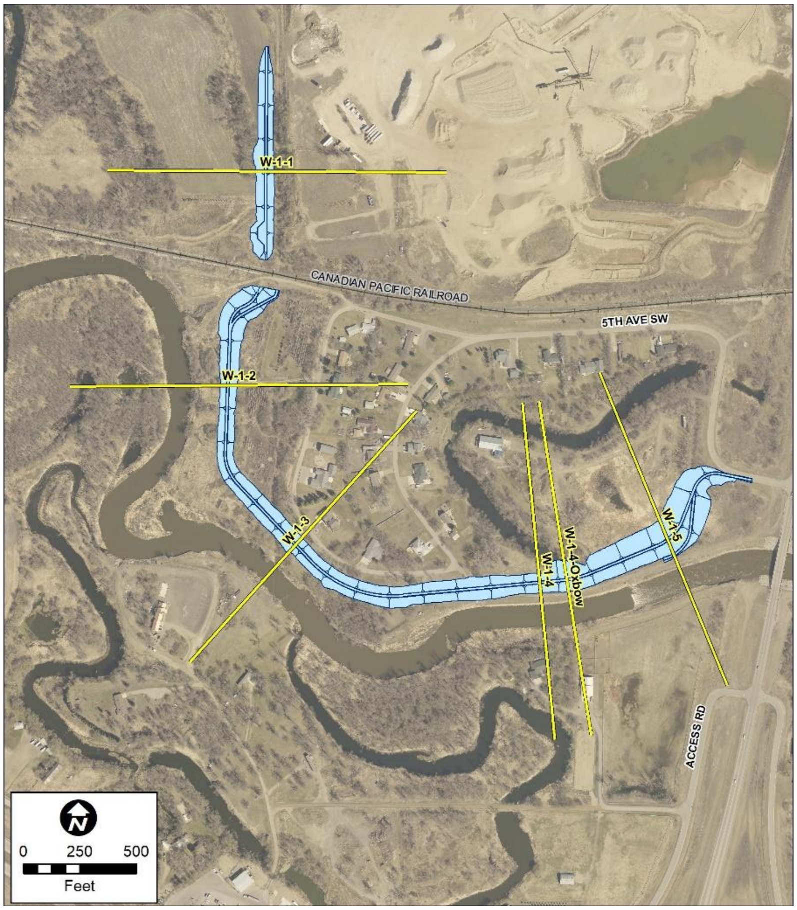


Figure 2-2 Cross-section Locations

The ground surface and river-bottom geometry used in the models were constructed using cross-sections developed from (1) a light-detecting and range survey (LiDAR) and (2) a river bathymetry survey. Results from these two surveys were merged into a surface to produce the cross-sections. The cross-section locations were selected to represent critical areas along the riverbank or to cover the extent of Phase WC-1.

2.6.2 Soil Stratigraphy

Soil stratigraphy was based on the borings logs, DMT sounding logs, and laboratory testing results from the current geotechnical investigation performed by Barr, supplemented with historical geotechnical information.

The contacts between various units were estimated by review of the soil types in each of the borings and inferred by CPT soil behavior type.

The main soil types encountered (and used as material types in the modeling) consist of:

- Low-fines content sand soils (SP and SP-SM).
- Higher-fines content silty/clayey sand soils (SM and SC)
- silty/sandy/lean clay soils (CL, CL-ML, and ML;
- Fat clay soils (CH)

Parameterization of each of these soils types will be needed to perform the subsequent modeling and analysis.

Significant interlayering and very thin soils were encountered within some sample intervals (i.e., several Shelby tube samples were found to contain up to three distinct soil types within one 2-foot sample). As determined from the investigations and along-levee soil profile, a typical ordered soil profile or consistent sequence/thickness of soil units was not identified. The modeled soil profile at each cross-section was estimated from the soils in each of the borings along the individual transects.

Review of deeper historical borings in the Phase WC-1 area indicated soils with high SPT N-values and descriptions of laminar clay soils (likely weathered shales of the underlying bedrock). A top elevation of 1,480 feet (approximately 70 to 80 feet below the ground surface) was incorporated in the modeling as a top of hard strata.

The geometry used at the specific cross-sections is shown in Appendix B.

2.6.3 Soil Parameters

2.6.3.1 Index Properties

MREFPP Phase WC-1 Atterberg limits for fat clay soils indicated liquid limits ranging from 50 to 75 percent, plastic limits ranging from 19 to 35 percent, and plasticity index values ranging from 9 to 47 percent. The natural moisture contents for the fat clay soils ranged from 23 to 75 percent. A grain size analysis indicated 1 percent gravel, a sand content of 6 percent, a silt content of 59 percent, and a clay content of 34 percent.

MREFPP Phase WC-1 Atterberg limits of the lean/silty clay soils indicated liquid limits ranging from 28 to 44 percent, plastic limits ranging from 14 to 23 percent, and plasticity index values ranging from 9 to 26 percent. The natural moisture contents ranged from 16 to 67 percent. Grain size analysis indicated up to 2 percent gravel, sand content ranging from 15 to 46 percent, and fines contents (silt and clay) ranging from 54 to 85 percent.

The MREFPP Phase WC-1 sand – high fines material grain size analysis indicated up to 18 percent gravel, 49 to 67 percent sand, and 18 to 52 percent fines (silt and clay). Moisture contents ranged from 5 to 39 percent.

The MREFPP Phase WC-1 sand – low fines material grain size analysis indicated up to 31 percent gravel, 65 to 98 percent sand, and 2 to 9 percent fines (silt and clay). Moisture contents ranged from 6 to 25 percent.

2.6.3.2 Unit Weight

MREFPP Phase WC-1 laboratory testing was performed on the intact Shelby tube samples and Modified California brass-lined samples where it was not practical top obtain Shelby tube samples. Laboratory dry density testing was performed in conjunction with moisture content testing to calculate in-situ unit weights and to estimate saturated unit weights for modeling. Saturated unit weights were derived using an assumed specific gravity and average dry unit weight values. Unit weights for modeling and analysis are indicated in Table 2-1.

Table 2-1 Moisture Content and Unit Weight by Soil Type

Material	Moisture Content		Dry Unit Weight		In-situ Unit Weight		Saturated Unit Weight
	Range	Average	Range	Average	Range	Average	Average
	(%)		(pcf)		(pcf)		(pcf)
Sand – Low Fines	6-25	17	103-121	113	127-140	132	116
Sand – High Fines	5-39	22	98-114	103	120-134	125	126
Lean/Silty Clay	16-37	28	87-108	95	117-130	122	124
Fat Clay	22-75	38	73-103	87	108-127	117	123

2.6.3.3 Borrow Materials

The same borrow area used for Phase 2-3 is likely the source of materials to be used for Phase WC-1.

2.6.3.4 Soil Strengths

Shear strengths for each soil stratigraphy were determined from laboratory and in-situ testing of boring soils. For materials exhibiting cohesive behavior, values were obtained from laboratory testing for both undrained and drained analysis conditions.

Direct Shear Testing of Sand/Silt and Clean Sand Soils

MREFPP Phase WC-1 direct shear testing was performed on granular (non-plastic behaving) materials. Testing results of all direct shear testing assumed no shear strength at zero effective normal stress (i.e., cohesion = 0 psf). A summary of MREFPP Phase WC-1 laboratory results is provided in Table 2-2. Data from MREFPP Phase 2, 3, 4, and WC-1 is plotted on Table 2-3.

Table 2-2 Direct Shear Testing

Boring ID	Soil Type by Lab	Shear Strength (psf)						
Normal Stress (psf)		**0**	**1000**	**1500**	**2000**	**3000**	**4000**	**6000**
SB-107-18 (depth 20.5–22 ft)	SP-SM	0	760	-	1,480	-	2,600	-
SB-115-18 (depth 32.5–34.5 ft)	SM	0	-	980	-	2,200	-	4,120
SB-120-18 (depth 18.5-19.5 ft)	SM	0	1,080	-	1,860	-	3,260	-

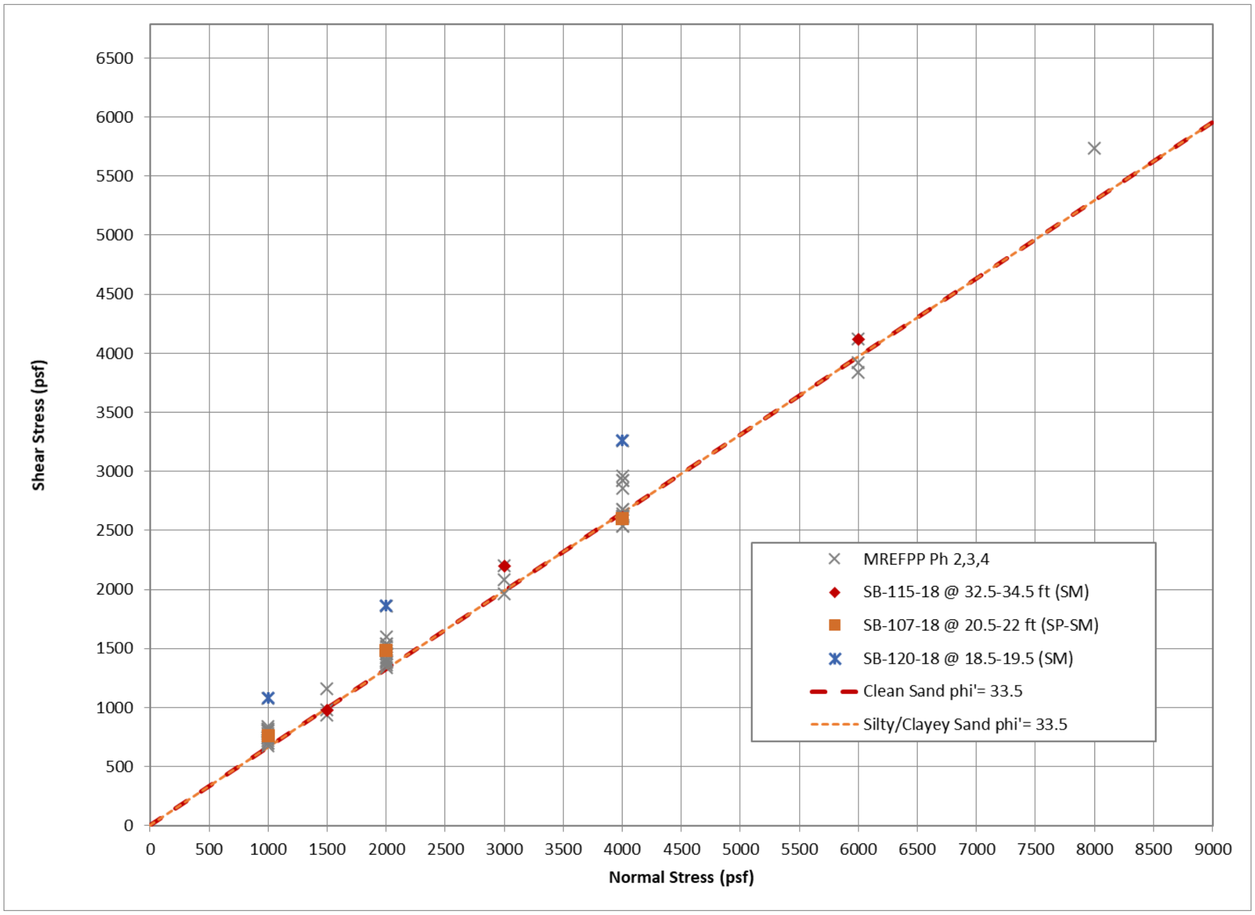


Figure 2-3 Friction Angle for Granular Soils from Direct Shear Testing

A friction angle of 33.5 degrees was indicated by a best-fit line plotted through the laboratory test data. This 33.5 degree friction angle was used for drained strength of the sand soils.

Undrained Strength of Clay Soils

Undrained strengths for the lean clay mix and fat clay soils were determined from triaxial unconsolidated undrained (UU) testing.

The laboratory test results indicated that the 33rd percentile undrained shear strength for the clay soils was about 1,214 psf for fat clay (CH) and about 1,393 psf for the lean/mixed clay soils. Values of 1,100 psf and 1,300 psf were selected for modeling of the fat clay and lean/mixed clay soils, respectively.

Laboratory test results were also compared to the undrained shear strengths from CPT testing at adjacent test locations. The laboratory test results were compared to an average undrained shear strength value over about 5 feet centered on the depth of the lab test sample (excluding different soil layers). Additional information concerning the correlation of CPT undrained shear strength and laboratory testing can be found in Appendix B.

A summary of undrained shear strength laboratory and in-situ testing is provided in Table 2-3.

Table 2-3 Undrained Shear Strengths from Laboratory Testing

Boring No.	Depth (feet)	Soil Type	Undrained Shear Strength (psf)	
			Lean/Silty Clay	Fat Clay
SB-102-18	20	CL	2,240	-
SB-104-18	27.5	CL	1,470	-
SB-108-18	20	CL and SP-SM	2,500	-
SB-123-18	35	CL	1,030	-
SB-109-18	40	CH	-	1,810
SB-112-18	40	CH	-	1,240
Minimum			1,030	1,240
Average			1,810	1,525
Maximum			2,500	1,810
33rd Percentile			1,466	1,428

Drained Strength of Clay Soils

Drained strengths for the silty to sandy lean clay and fat clay soils were estimated from triaxial consolidated undrained (CU) test results. These points were plotted together and a best fit line using the ⅓–⅔ principle (assuming zero shear strength at zero normal stress) was fit through the text data points. A friction angle of 33.5 and 35.5 degrees for the drained shear strength envelopes of the fat clay and lean clay soils, respectively. Triaxial testing results and shear strength envelopes are shown in Figure 2-4 and Figure 2-5.

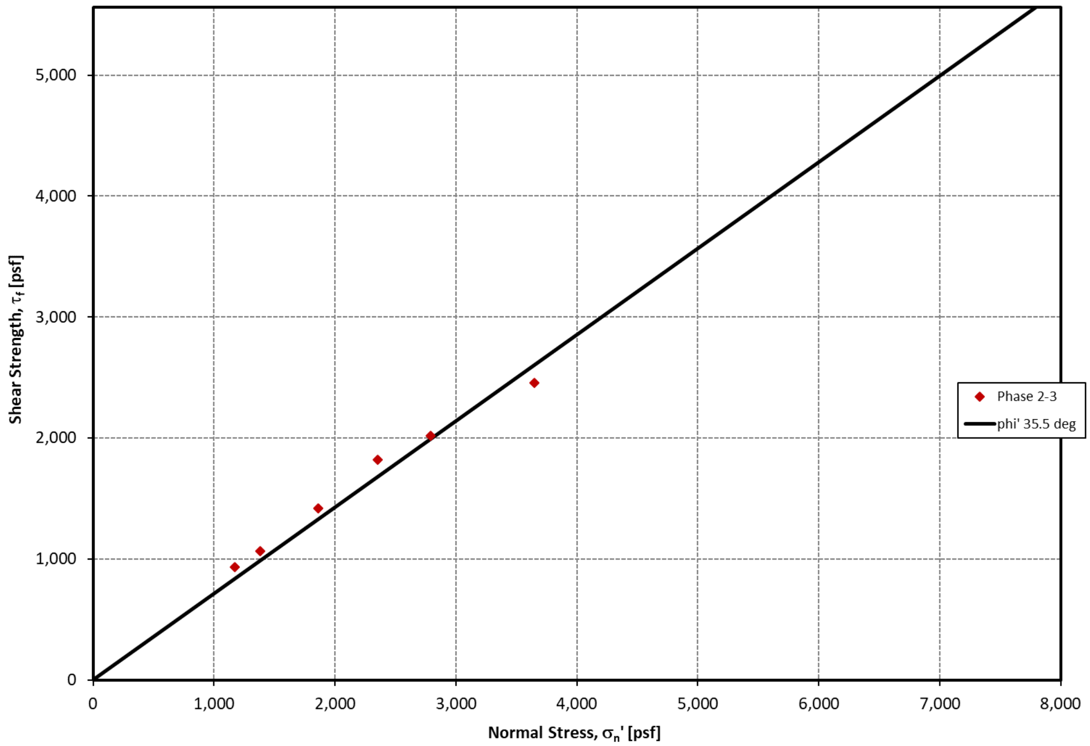


Figure 2-4 **Lean Clay Drained Shear Strength Envelope, Maximum Deviator Stress Failure Criterion**

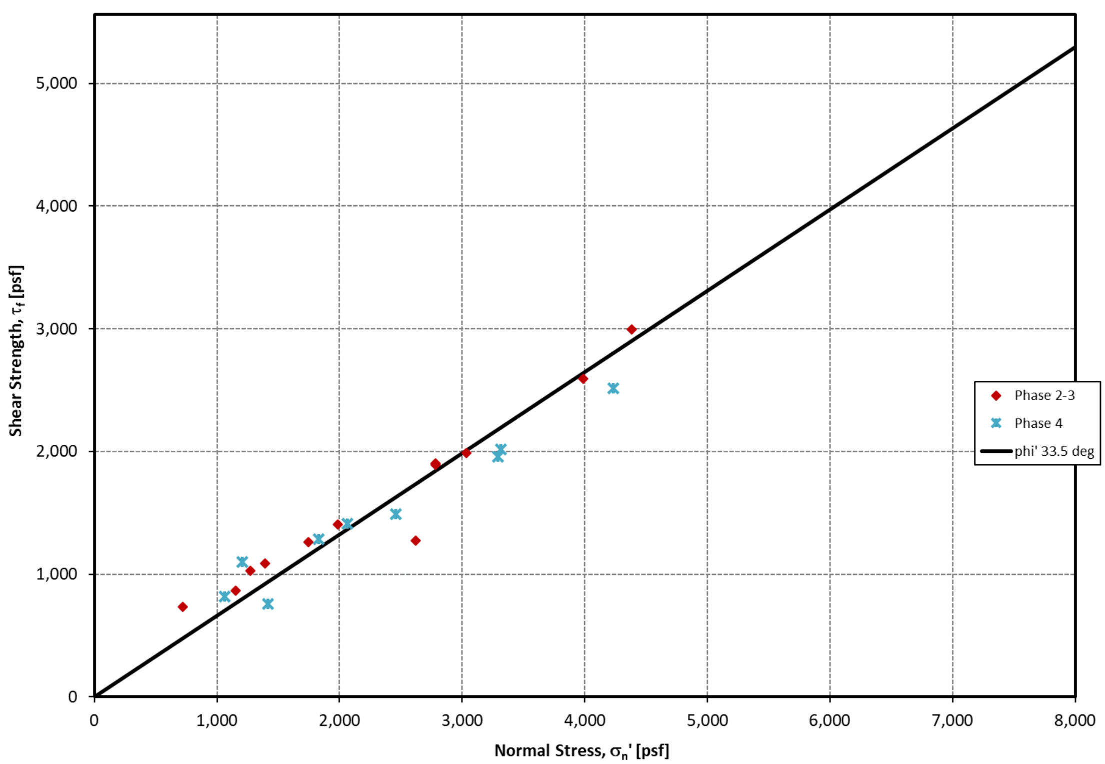


Figure 2-5 **Fat Clay Drained Shear Strength Envelope, Maximum Deviator Stress Failure Criterion**

Levee Borrow Material Strength

The undrained and drained shear strength of borrow material based on laboratory testing from Phase 2-3 of the project is shown on Figure 2-6 and Figure 2-7, respectively, All test samples were remolded bulk specimens of the lean clay glacial till compacted to approximately 95 percent of their maximum dry density based on standard Proctor testing. This is the borrow materials currently planned to be used for the Phase WC-1 portion of the project.

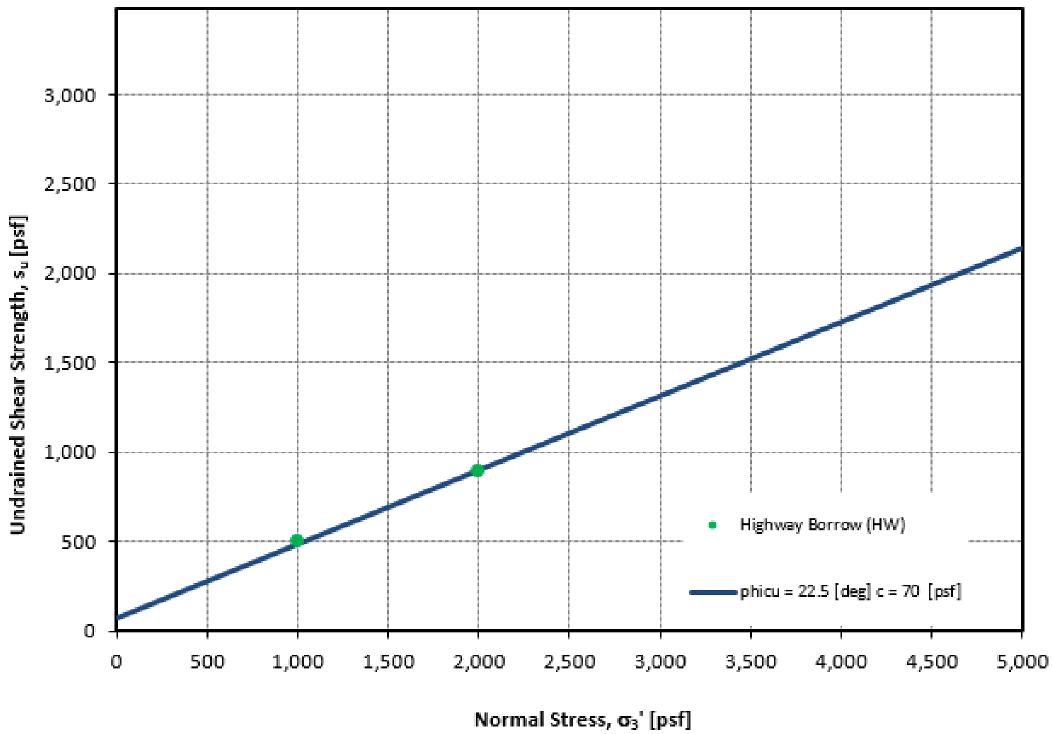


Figure 2-6 Borrow Undrained Shear Strength Envelope, Maximum Deviator Stress Failure Criterion

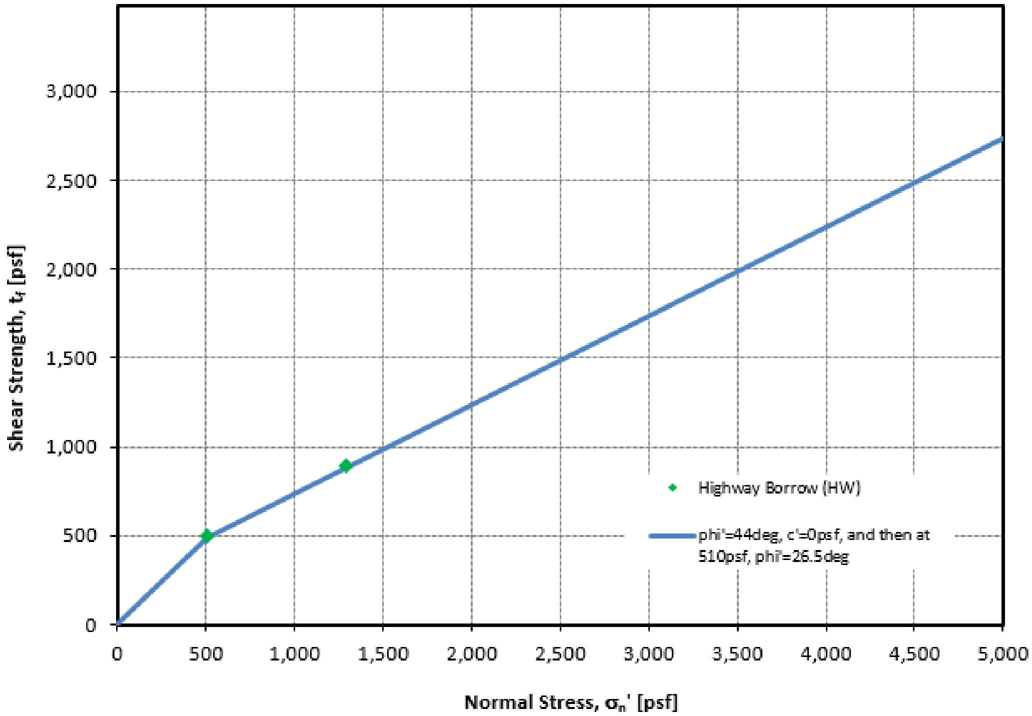


Figure 2-7 Borrow Drained Shear Strength Envelope, Maximum Deviator Stress Failure Criterion

2.6.3.5 Summary of Modeling Parameters

Material properties used for slope-stability analyses are summarized in Table 2-4. Unit weight and shear strength of sand/silt and clean sand materials were developed using the laboratory data (Figure 2-6 and Figure 2-7).

Table 2-4 Unit Weights and Shear Strength Parameters Used in Slope-Stability Analyses

Material	Unit Weight		ESSA - drained		USSA - undrained	
	Moist (pcf)	Sat (pcf)	c' (psf)	ϕ' (degrees)	c (psf)	ϕ (degrees)
Sand-Low Fines	130	125	33.5	-	-	-
Sand-High Fines	125	120	33.5	-	-	-
Lean/Silty Clay	125	120	32	-	1,300	-
Fat Clay	120	115	32	-	1,100	-
Fill - Cohesive	125	120	32	-	1,000	-
Fill - Granular	125	120	33.5	-	-	-
Levee Fill	128	122	c' = 0, ϕ' = 44 deg until σ' = 510 psf, ϕ' = 26.5 deg at higher σ'		100	21.5
Sand/Gravel	130	125	33.5	-	-	-
Riprap	130	125	45	-	-	-
Shale	120	115	32	-	3,000	-

2.6.3.6 Compressibility

Soil compressibility was characterized using the in-situ DMT results and laboratory one-dimensional consolidation test results. A summary of DMT constrained modulus values can be found in Appendix B.

Laboratory consolidation testing on Shelby tube soil samples was also completed to evaluate settlement potential of the underlying saturated clay strata. A summary of the laboratory consolidation test results is provided in Table 2-5.

Table 2-5 Laboratory Consolidation Test Results

Boring	Depth (ft)	Soil Type	Groundwater Depth(1) (feet)	Moist Unit Wt. (psf)	P_o (psf)	P_c (psf)	OCR	C_c	C_r	e_o
B-40-15	24	CL and SM Mix	10	119.9	2,004	6,000	3.0	0.17	0.03	0.790
B-53-15	42	CL-ML	8	118.8	2,868	6,000	2.1	0.17	0.02	0.824
SB-14-18	18	CL	11	114.3	1,621	1,260	1(2)	0.23	0.02	0.975
SB-105-18	27.5	CL	10	118.2	2,159	4,600	2.1	0.38	0.08	0.929
Lean/Silty Clay Averages							**2.1**	**0.24**	**0.04**	**0.880**
B-18-15	36	CH	24	118.9	3,532	2,600	1(2)	0.26	0.05	0.898
B-28-15	30	CH	14	113.9	2,419	1,060	1(2)	0.33	0.06	1.078
B-56-15	40	CH	10	116.5	2,788	3,200	1.1	0.28	0.06	0.960
SB-112-18	40	CH	10	116.2	2,777	3,400	1.2	0.38	0.09	1.014
Fat Clay Averages							**1.1**	**0.31**	**0.07**	**0.988**
SB-101-18	22.5	CH - Weathered Shale	30	126.6	3,317	17,800	5.4	0.13	0.03	0.640
Weathered Shale							**5.4**	**0.13**	**0.03**	**0.640**

(1) Groundwater depths are "while drilling" measurements in soil borings. Likely do not reflect static groundwater levels due to short stabilization times and addition of drilling fluid, but are slightly shallower than VW groundwater levels and slightly more conservative for settlement calculations.

(2) Test results indicate samples were likely disturbed. Assumed to be normally consolidated with an OCR=1.

2.6.3.7 Permeability

Laboratory permeability testing was performed on soil samples collected during the Phase WC-1 field investigation. The clay samples were undisturbed Shelby tube samples and the sand samples were either undisturbed in Shelby tubes or samples remolded to a density value obtained from Shelby tube or Modified California samples. Laboratory testing was completed in the vertical (k_y) and horizontal (k_h) direction.

Laboratory permeability testing and CPT pore-pressure dissipation testing from Phase 2, 3, 4, and WC-1 were used to establish vertical and horizontal permeabilities for the Project. CPT pore-water dissipation testing measures the change in pore-water pressure created as the cone is forced into the ground displacing soil and water. Excess pore-water pressure dissipates through the soil surrounding the cone tip and through the least restrictive flow path. In the instance of this Project, the least restrictive flow path is assumed to be in the horizontal direction. Using the CPT pore-water dissipation curve of pore pressure versus time, a horizontal permeability (k_h) was established. The ratio k_v/k_h, which describes anisotropy of permeability, was developed comparing laboratory testing results to those of the CPT dissipation testing (where test data was available) and horizontal laboratory permeability testing. Permeability values used in modeling are provided in Table 2-6 and illustrated in Figure 2-8.

Table 2-6 Permeabilities Used in Seepage Analysis

Material	k_v cm/sec	k_v ft/sec	k_h cm/sec	k_h ft/sec	k_v/k_h ratio
Sand-Low Fines	1.58E-03	5.19E-05	3.96E-03	1.30E-04	0.400
Sand-High Fines	3.86E-06	1.27E-07	5.92E-04	1.94E-05	0.006
Lean/Silty Clay	1.51E-07	4.94E-09	5.19E-05	1.70E-06	0.003
Fat Clay	2.64E-08	8.67E-10	9.20E-08	3.02E-09	0.300
Fill - Cohesive	5.19E-05	1.70E-06	5.19E-05	1.70E-06	1.000
Fill - Granular	5.92E-04	1.94E-05	5.92E-04	1.94E-05	1.000
Levee Fill	5.79E-08	1.90E-09	5.79E-08	1.90E-09	1.000
Sand/Gravel	1.00E-01	3.28E-03	1.00E-01	3.28E-03	1.000
Riprap	1.00E-01	3.28E-03	1.00E-01	3.28E-03	1.000
Shale	2.64E-08	8.67E-10	9.20E-08	3.02E-09	0.300

(1) Assumed Value
(2) Used in high permeability layer sensitivity analysis for mud loss/gravel layers where encountered and at the apparent oxbow feature near the U.S. Highway 83 Bypass.

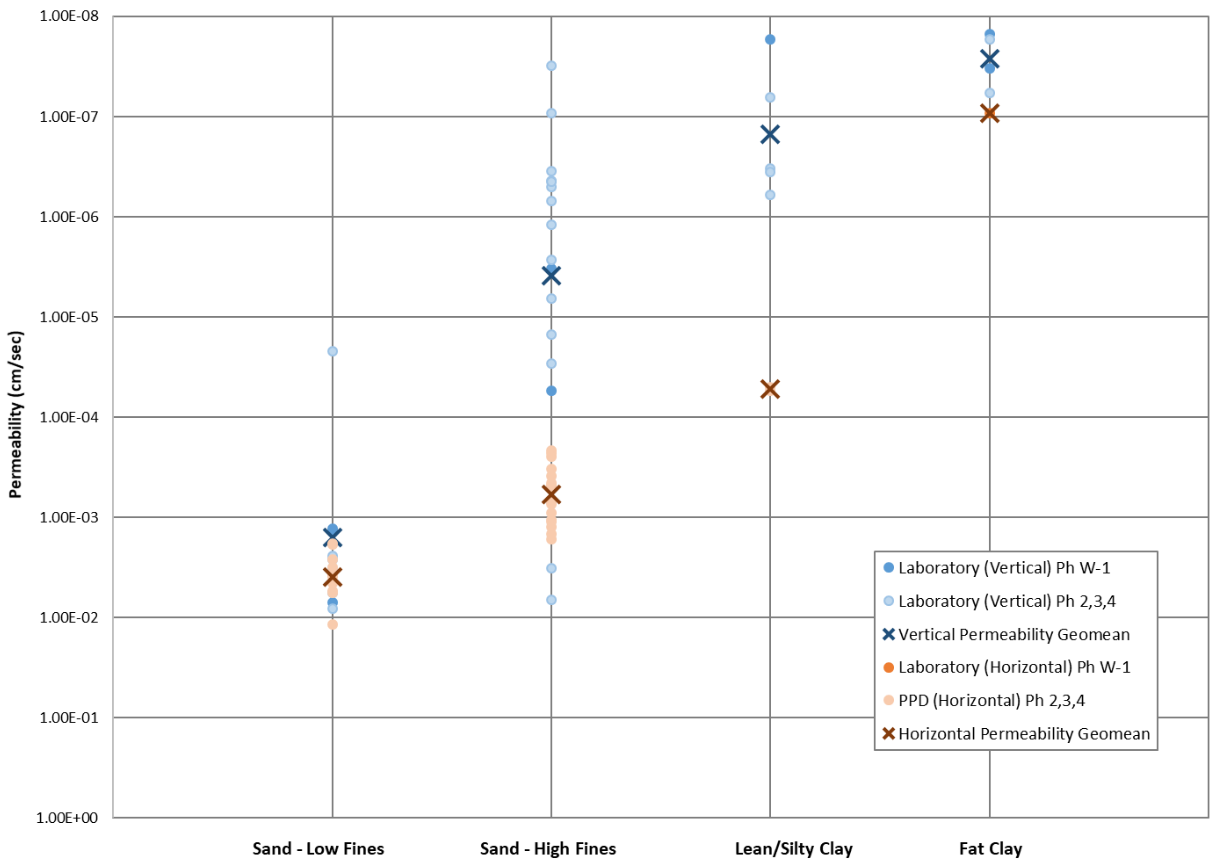


Figure 2-8 Comparison of Soil Permeability

2.7 Seepage and Stability Analysis

The seepage and stability analysis examined the hydraulic behavior and stability of existing and proposed levee configurations under various scenarios. The setback analyses considered low river flow under drained and undrained soil conditions as well as a rapid drawdown scenario. The proposed levee analysis examined levee stability under low river flow conditions both Landside and Riverside for drained and undrained soil conditions. Landside stability was analyzed during flooding events using the design water surface elevation (DWSE) and maximum water surface elevation (MWSE). Stability was evaluated using drained soil conditions for both DWSE and MWSE and undrained soil conditions for DWSE. Piping/erosion and heave factors of safety were developed for the upward gradient at the Landside toe of the levee for both the DWSE and MWSE events.

2.7.1 Modeling Method

The seepage conditions and slope stability of the levee embankment, foundation, and riverbanks were analyzed with software created by GEO-SLOPE International Ltd. The integrated software suite is called GeoStudio 2012 and includes SEEP/W and SLOPE/W. SEEP/W is a finite-element program that analyzes groundwater seepage within porous materials like rock and soil for traditional steady-state flow or transient analyses. The computed pore-water pressures and groundwater surface can then be imported into SLOPE/W, allowing the program to analyze complex saturated/unsaturated or transient conditions. SLOPE/W uses limit equilibrium methods to perform slope stability analyses.

2.7.2 Model Cross-sections

A total of six cross-sections were analyzed for Phase WC-1. Figure 2-2 shows the plan view of the site and the relative location of the geotechnical cross-sections.

The typical configuration of the levee prism consists of Riverside (wet side) and Landside (dry side) slopes of 3 horizontal units to 1 vertical unit (3H:1V). The levee crest width is 14 feet (10-foot crest with 2-foot shoulders).

2.7.3 Seepage Analysis

Seepage modeling was conducted to gain a better understanding of the groundwater conditions at each cross-section and incorporate seepage results into slope stability analyses. The seepage analysis included an estimate of the levee under seepage rate and an evaluation of the gradient at the Landside levee toe (piping/erosion evaluation). Heave calculations were also completed to verify that the total stress of soils overlying soil units were able to resist upward (vertical) water pressure in more permeable sand layers during flooding events. Piping/erosion and heave calculations initially assume that additional fill (buttress or seepage blanket) is not placed against the Landside of the levee.

2.7.3.1 Seepage Model Properties

The main parameter relevant to seepage analyses is hydraulic conductivity or permeability of the soils. The hydraulic conductivity is a measure of the resistance of flow through a saturated soil media based on a known hydraulic head differential. This parameter is used to evaluate the potential of groundwater moving through soils below the levee or soils used to construct the levee.

Other parameters which define the hydraulic conductivity function and volumetric water content functions, which apply the unsaturated region of the model above the phreatic surface were determined using correlations to material type or laboratory index test results.

2.7.3.2 Boundary Conditions

Hydraulic head conditions consistent with MWSE and DWSE water levels were applied along the ground surface from the river to the proposed levee. The normal and low-flow water surface elevations were applied to the river channel and low-lying areas directly connected to the river channel.

Boundary conditions were varied depending on the analysis performed. A summary of total head boundary condition is provided in Table 2-7. Boundary conditions consistent with MWSE and DWSE water levels were applied along the ground surface from the natural river bottom to the Riverside face of the proposed levee. The normal- and low-flow river water surface elevations were applied to the river channel and low-lying (floodplain) areas directly connected to the river channel.

The Landside far-field boundary condition was applied at a distance of at least 800 feet away from the centerline of the river determined from vibrating wire piezometer readings and engineering judgment, as shown in Table 2-7. Far-field total head conditions greater than the normal river flow elevations were selected for modeling based on the assumption that this reach of river is a gaining stream and proposed construction of a weir to control water flows within the Project's oxbow feature. A proposed weir elevation is 1550 feet. A far-field hydraulic boundary condition of 1550 feet was also used when modeling the flood events.

Table 2-7 Hydraulic Boundary Conditions

Cross-Section	MWSE (top of construction) Elevation	DWSE Elevation	Normal Flow Elevation	Low Flow Elevation	Nearest Piezometer	Far-field Total Head
	(ft)	(ft)	(ft)	(ft)		(ft)
W-1-1	1,573.6	1,570.9	1,550.3	1,550.1	SB-107-18	1550.0
W-1-2	1,573.5	1,570.0	1,550.3	1,550.1	SB-107-18	1550.0
W-1-3	1573.4	1,569.7	1,550.3	1,550.1	SB-114-18	1550.0
W-1-4	1,573.1	1569.3	1,550.3	1,550.1	SB-123-18	1550.0
W-1-4 Oxbow	1573.1	1569.2	1,550.3	1,550.1	SB-123-18	1550.0
W-1-5	1573.0	1569.1	1,548.7	1,547.1	SB-123-18	1550.0

2.7.3.3 Seepage Results

The piping/erosion factor of safety is only applied at cross-sections, with groundwater seepage at or near the ground surface of the Landside toe of the levee. The factor of safety for piping/erosion is estimated by dividing the critical gradient (buoyant soil unit weight/unit weight of water) by the exit gradient (change in head/distance between measured heads). The exit gradient was calculated (1) between the Landside toe of the levee and 2 feet below the Landside toe when the levee is founded on granular materials and (2) across the entire thickness of the uppermost clay layer when the levee is founded on cohesive materials.

The heave factor of safety is determined by dividing total vertical stress by pore-water pressure at the interface of a high permeable material and a low permeability material when the low permeability material overlies the high permeability material. If a lower point along the ground surface exists close to the levee toe, the heave factor of safety is determined for the lower ground surface point.

The minimum required factor of safety against heave at the Landside toe of the levee cross-sections is 1.6 for the DWSE and 1.3 for MWSE. The piping/erosion and heave factors of safety were found to be sufficient at the all of the cross-sections except W-1-5.

Table 2-8 provides the computed heave and piping/erosion factors of safety without seepage mitigation.

Table 2-8 Piping Erosion and Heave Factors of Safety without Seepage Mitigation

Model	Required FoS	W-1-1	W-1-2	W-1-3	W-1-4	W-1-4 Oxbow	W-1-5
5.0 Steady-State Seepage, Proposed (Case III - DSWE)							
Toe of Levee (Heave FoS)	1.6	1.92	1.86	2.63	2.74	2.01	**0.70**
Landside of Levee Toe (Heave FoS)	1.6	1.91	1.69	2.29	1.92	1.86	**0.59**
Toe of Levee (Piping/Erosion FoS)	1.6	Note 1	3.45	6.69	Note 1	Note 1	**1.43**
Landside of Levee Toe (Piping/Erosion FoS)	1.6	Note 1	7.31	Note 1	>10	Note 1	5.02
6.0 Steady-State Seepage, Proposed (Case III - MWSE)							
Toe of Levee (Heave FoS)	1.3	1.81	1.75	2.36	2.77	1.98	**0.63**
Landside of Levee Toe (Heave FoS)	1.3	1.79	1.59	2.11	1.89	1.83	**0.54**
Toe of Levee (Piping/Erosion FoS)	1.3	Note 1	2.40	5.02	Note 1	Note 1	**1.25**
Landside of Levee Toe (Piping/Erosion FoS)	1.3	Note 1	5.12	Note 1	>10	Note 1	5.02

Bold font factors of safety are below the required minimum factor of safety.
Note 1: Groundwater is at or below ground level surface.

2.7.3.4 Seepage Mitigation

As indicated in Table 2-8 most of the piping/erosion factors of safety are deficient. A number of seepage mitigation options were considered to control seepage near the Landside levee toe (Table 2-9).

Table 2-9 Evaluation of Seepage Mitigation Options

Option	Pro	Con
No additional seepage mitigation	• Minimal additional costs and work	• Does not reduce seepage or increase stability • Does not improve low factors of safety at toe • Would require a reduction of USACE factor of safety requirements
Seepage berm	• Lower costs • Waste local materials (if suitable) • Improve stability at toe • Would not require a specialty contractor	• Could not be performed in easement – requires large area behind levee • Does not reduce under-seepage
Zoned embankment	• Lower costs • Waste local excess materials (if suitable) • Would not require a specialty contractor	• Possibly more seepage • Does not improve under-seepage • Does not improve low factors of safety at toe
Relief wells	• Lower cost than cut off walls or sheet pile • Relief of pressure at toe • Improves stability at toe • Minimal space required • Likely does not require specialty contractor • Does not cut off interior drainage	• Does not reduce seepage, so must collect and divert water • Maintenance intensive • Would require many wells to adequately relieve pressure
Partially penetrating pressure relief trench	• Lower cost than cut off walls or sheet pile • Relief of pressure at toe • Improves stability at toe • Minimal space required • Likely does not require specialty contractor • Does not cut off interior drainage	• Does not reduce seepage, so must collect and divert water • Reliance upon graded filter design • Long-term maintenance will be required
Deep slurry cut off	• Reduces seepage • Improves stability at toe • Can be performed in easement	• High cost • Wide, low perm cutoff works best • Requires specialty contractor • Cuts off interior groundwater drainage to the river

Option	Pro	Con
Shallow slurry cut off wall	• Somewhat reduces seepage • Marginally improves stability at toe • Can be performed in easement • Lower cost than deep cutoff wall	• Shallower cutoff is less effective than deeper • May need to supplement with other methods to achieve USACE requirements • Potentially cuts off interior drainage
Sheet pile cutoff	• Reduces seepage • Improves stability at toe • Can be installed in easement	• High cost • May need specialty contractor • Depth of sheeting may be limited • Inhibits natural groundwater drainage to the river

Due to the relatively low topography and wide expanse of minimal development in areas where deficient factors of safety were identified, seepage berms/grade raises are anticipated to be best suited to mitigate the pore-water pressures at the levee toe and in underlying clean sand seams.

By adding a seepage berm which raises the ground surface and collects the water within the seepage blanket before it rises to the ground surface, the piping/erosion and heave factors of safety are increased to adequate levels. The summary of piping/erosion and heave factors of safety is in Table 2-10.

Table 2-10 Piping Erosion and Heave Factors of Safety with Seepage Mitigation

Model	Required FoS	W-1-1	W-1-2	W-1-3	W-1-4	W-1-4 Oxbow	W-1-5
5.0 Steady-State Seepage, Proposed (Case III - DSWE)							
Toe of Levee (Heave FoS)	1.6	-	-	-	-	-	1.66
Landside of Levee Toe (Heave FoS)	1.6	-	-	-	-	-	1.79
Toe of Levee (Piping/Erosion FoS)	1.6	-	-	-	-	-	Note 1
Landside of Levee Toe (Piping/Erosion FoS)	1.6	-	-	-	-	-	Note 1
6.0 Steady-State Seepage, Proposed (Case III - MWSE)							
Toe of Levee (Heave FoS)	1.3	-	-	-	-	-	1.55
Landside of Levee Toe (Heave FoS)	1.3	-	-	-	-	-	1.69
Toe of Levee (Piping/Erosion FoS)	1.3	-	-	-	-	-	Note 1
Landside of Levee Toe (Piping/Erosion FoS)	1.3	-	-	-	-	-	Note 1

Note 1: Groundwater is below ground level surface.
"-" indicates seepage mitigation was not required.

An update on seepage mitigation features will be provided in the 90 percent design submittal.

2.7.4 Slope Stability Analyses

The main objective of the slope stability analysis was to evaluate the stability of the levees and riverbanks under static conditions. Two types of stability analyses are typically performed for slopes: the Undrained Strength Stability Analysis (USSA) and the Effective Stress Stability Analysis (ESSA).

The USSA is performed to analyze the case in which loading or unloading is applied rapidly and excess pore-water pressures do not have sufficient time to dissipate during shearing. This scenario typically applies to loading from, for example, embankment construction where the loading takes place quickly relative to the permeability of the soils. It is often referred to as the "end-of-construction" case.

The ESSA is performed to account for much slower loading or unloading, or no external loading, in which the drained shear strength of the materials is mobilized and no excess pore-water pressures are allowed to develop. For example, a slowly moving landslide is best analyzed using the ESSA method. For this reason, the ESSA is often referred to as the "long-term" case.

Both the USSA and ESSA were performed as part of the slope stability analysis for Phases WC-1. The factor of safety was computed by incorporating the results of the seepage analysis under steady-state conditions. Incorporating the groundwater flow with the limit equilibrium calculations captures the effect of fluid/soil interaction on the factor of safety calculation. In this manner, emphasis was placed on evaluating the impact of groundwater flow on stability.

The stability of a slope is often reported using a factor of safety value. The factor of safety is the ratio of the summation of forces and moments that are resisting slope movement to the summation of forces and moments that cause slope movement. These forces and moments could result from increased loading or decreased resistance, which may be caused by variation in pore-water pressure and the buttressing effect induced by changes in river levels.

2.7.4.1 Slope Stability Model Properties

The key parameter associated with levee stability is shear strength. Material properties used for slope stability analyses are summarized in Table 2-4. Modeling parameters for the proposed materials and existing soil conditions were determined using laboratory test results.

2.7.4.2 Modeling Scenarios

Setback and proposed conditions were analyzed for slope stability. Analysis consisted of the following:

- The setback analysis included stability at low river flow (Riverside) and rapid drawdown (Riverside) conditions. Both drained and undrained soil conditions were considered for the setback analyses.

- The proposed conditions analyzed included low river flow conditions (Landside and Riverside), rapid drawdown (Riverside), DWSE flood event (Landside), and MWSE flood event (Landside). Drained soil conditions were considered for the proposed levee configuration at low river flows and DWSE and MWSE events. Undrained soil conditions were considered for the proposed levee configuration at normal river flows.

Required factors of safety are summarized in Table 2-11.

Table 2-11 Summary of Slope Stability Analyses and Minimum Required Factors of Safety

Model Name	Required Minimum Factor of Safety
1.0 Steady-State Seepage, Setback (Low Flow)	-
1.1 Slope Stability, Setback (ESSA - Riverside)	1.4
1.2 Slope Stability, Setback (USSA - Riverside)	1.3
2.1 Slope Stability, Setback (Sudden Drawdown - Riverside)	1.0
3.0 Steady-State Seepage, Proposed (Case I)	-
3.1.1 Slope Stability, Proposed (ESSA - Landside)	-
3.1.2 Slope Stability, Proposed (ESSA - Landside)	-
3.2.1 Slope Stability, Proposed (Case I - USSA - Landside)	1.3
3.2.2 Slope Stability, Proposed (Case I - USSA - Riverside)	1.3
4.1 Slope Stability, Proposed (Case II - Sudden Drawdown - Riverside)	1.0
5.0 Steady-State Seepage, Proposed (Case III - DSWE)	-
5.1 Slope Stability, Proposed (Case IIIa - ESSA - Landside)	1.4
5.2 Slope Stability, Proposed (Case IIIc - USSA - Landside)	1.3
6.0 Steady-State Seepage, Proposed (Case III - MWSE)	-
6.1 Slope Stability, Proposed (Case IIIb - ESSA - Landside)	1.3

2.7.4.3 Slope Stability Results

Model outputs for the slope stability analyses are in Appendix B.

The setback analysis provides the minimum offset distance necessary to achieve the Riverside stability factor of safety required for the Project. At a distance less than the reported offset value, the factor of safety is less than required. The setback is defined as the distance between the slope extending down to the river channel and the most landside potential slip surfaces having lower than required factors of safety, as shown on Figure 2-9.

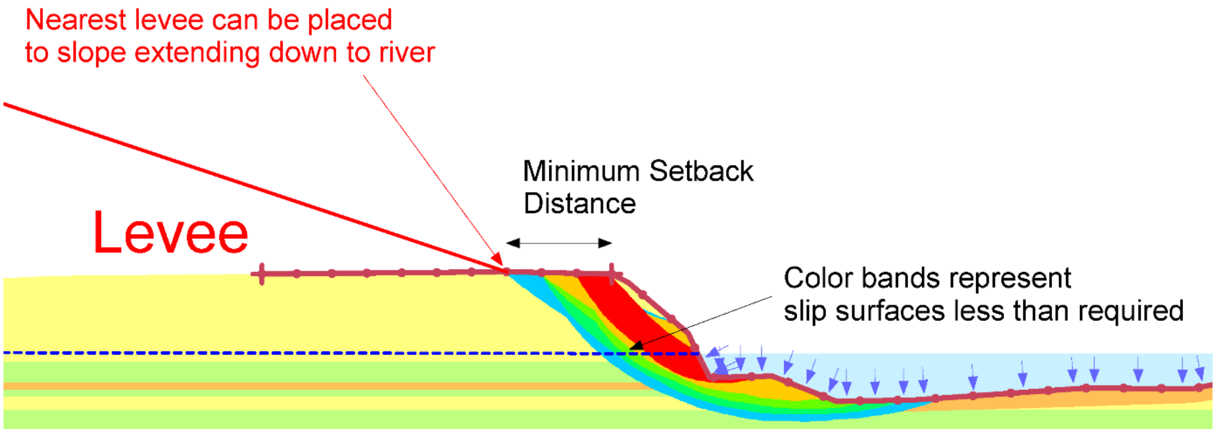


Figure 2-9 Depiction of Minimum Setback Distance

A summary of the minimum required distance between the Riverside levee toe and Riverside embankments is presented in Table 2-12. Factors of safety for the setback analysis are defined in Table 2-13. In general, slopes which were close to river or low lying area slopes (less than 30 feet), will be modified as a part of the design to improve stability, erodibility, or scour potential, and therefore did not require further setback analysis. With consideration to levee/embankment heights, soil, and potential hydraulic conditions, levee setback analysis were not considered necessary slopes farther than 30 feet away from the levee toe.

Table 2-12 Minimum Setback Distance Between Levee Toe and River Channel to Achieve Required Factors of Safety for Slope Stability

Cross-Section	Distance from Levee Toe to River Embankment (ft)	Alteration to River Embankment
W-1-1	-	Not Adjacent to River(1)
W-1-2	110	None
W-1-3	80	3.5H:1V Slope Overbank Excavation at toe of Levee
W-1-4	50	3.5H:1V Slope Overbank Excavation at toe of Levee
W-1-4 Oxbow	50	3.5H:1V Slope Overbank Excavation at toe of Levee
W-1-5	70	3.5H:1V Slope Overbank Excavation at toe of Levee

(1) Setback Analysis for Landside gravel pit currently being analyzed

Table 2-9 summarizes the conditions analyzed and the factors of safety associated with each condition. As shown in Table 2-13, some models are given the designation Case I, Case II, and Case III as described in USACE 1110-2-1913 (reference [15]). Additional guidance was provided by the St. Paul District of the USACE for development of the Case IIIb and Case IIIc conditions, which are associated with the MWSE.

Table 2-13 Levee Prism Factors of Safety for Slope Stability (with seepage remediation)

Summary of Slope Stability Factors of Safety							
Model	**Required FoS**	**W-1-1**	**W-1-2**	**W-1-3**	**W-1-4**	**W-1-4 Oxbow**	**W-1-5(2)**
3.0 Steady-State Seepage, Proposed (Case I)							
3.1.1 Slope Stability, Proposed (ESSA - Landside)	-	3.30	2.53	2.67	2.75(1)	2.17	2.86
3.1.2 Slope Stability, Proposed (ESSA - Riverside)	-	2.70	2.86(1)	2.33(1)	2.41(1)	2.53(1)	2.55(1)
3.2.1 Slope Stability, Proposed (Case I - USSA - Landside)	1.3	2.40	2.33	2.24	2.23(1)	1.71	1.92
3.2.2 Slope Stability, Proposed (Case I - USSA - Riverside)	1.3	2.37	2.49(1)	2.44(1)	2.29(1)	1.98(1)	1.84(1)
4.3.1 Slope Stability, Proposed (Case II - Sudden Drawdown - Landside	1.0	-	-	-	2.19(1)	1.45	-
4.3.2 Slope Stability, Proposed (Case II - Sudden Drawdown - Riverside	1.0	1.33	1.35(1)	1.09(1)	1.10(1)	1.18(1)	1.25(1)
5.0 Steady-State Seepage, Proposed (Case III - DSWE)							
5.1.1 Slope Stability, Proposed (Case III - ESSA - Landside)	1.4	2.52	1.88	2.06	2.72(1)	2.16	1.86
5.2.1 Slope Stability, Proposed (Case IIIc - USSA - Landside)	1.3	2.07	2.22	2.14	2.24(1)	1.71	1.39
6.0 Steady-State Seepage, Proposed (Case III - MWSE)							
6.1.1 Slope Stability, Proposed (Case IIIb - ESSA - Landside)	1.3	2.30	1.70	1.92	2.63(1)	2.16	1.47

(1) Entry-Exit search method used to determine Factor of Safety.
(2) Factor of Safety with seepage remediation.

2.8 Levee Settlement Analysis

Levee settlement was calculated using constrained modulus results from the DMT and laboratory compressibility testing results.

Stress introduced by the levee was computed for the mid-point of each layer for both the DMT and laboratory settlement analyses. The "Poulos and Davis Method," discussed in Appendix B, was used to calculate the stress within each soil layer. It was assumed that long-term consolidation settlement would not occur in the unsaturated zone above the water table. Geometry for settlement analysis was based on one of two configurations: a trapezoidal prism (new levee placed on relatively flat ground) and a trapezoidal fill area above a rectangular fill area (new levee fill placed over the existing levee trapezoid). For areas with highly complex geometry, both configurations were modeled to evaluate a potential range of settlement.

Based on the results of the DMT and laboratory calculations, the settlement is estimated to be on the order of approximately up to 17.2 inches for the Levee alignment. A summary of settlement results for the levee is provided in Table 2-14. It is recommended that the levee be overbuilt by these amounts to account for potential settlement. These overbuild heights would also be used to determine the Maximum Water Surface Elevations (MWSE) which are provided in each analyzed cross-section (Table 2-7). The MWSE represents the highest possible water level for analysis which is the top of the as constructed levee.

Table 2-14 Summary of Estimated Consolidation Settlement for Levee

Cross-section	Settlement from DMT (inches)		Settlement from Lab Consolidation Testing (inches)	
	Trapezoidal Prism(2)	Trapezoidal/ Rectangular Prism(2)	Trapezoidal Prism(2)	Trapezoidal/ Rectangular Prism(2)
W-1-2	1.4	-	1.9	-
W-1-3	2.1	-	0.5	-
W-1-4	5.2	-	1.5	-
W-1-4 Oxbow	--	7.0	-	5.5
W-1-5	11.6	-	17.2	-

(1) DMT not performed at cross-section. Settlement evaluation will be performed from laboratory test values

(2) Trapezoidal/Rectangular Prism geometry uses the settlement superposition technique to approximate settlement at cross-sections with uneven native ground surfaces. This technique to calculate settlement was only where existing/proposed geometry was appropriate. At cross-sections where ground was flat a more typical Poulos and Davis method was used for trapezoidal fill cross-sections.

NOTE:
For the 60 percent analysis, an initial settlement of 12 inches was applied to the alignment. The overbuild heights will be adjusted for the 90 percent analysis.

2.9 Bearing Capacity and Settlement Analysis for Tierrecita Vallejo Gatewell and Railroad Closure Structure

The peak strengths from field and laboratory testing were analyzed to evaluate allowable bearing capacity for the Tierrecita Vallejo gatewell and railroad closure structures. USACE guidance was used for bearing capacity calculations (EM 1110-1-1905, *Bearing Capacity of Soils*, reference [16]).

The compressibility of the clay was determined from the laboratory consolidation tests of site-specific soil borings and in-situ DMT testing, as discussed in Appendix B. USACE guidance, EM 1110-1-1904 (*Settlement Analysis*, reference [17]), was used for settlement calculations. Sandy soils should experience immediate settlement and will not contribute to long-term consolidation settlement.

The results of this analysis are summarized in the following sections of this report and detailed in Appendix B.

2.9.1 Bearing Capacity for Structures

The allowable bearing capacity for the railroad closure structure and Tierrecita Vallejo gatewell is anticipated to be 3,000 psf at the time of this report.

2.9.2 Tierrecita Vallejo Gatewell Structure

The magnitude of settlement for the levees along the WC-1 Project area levee is estimated at 12 inches. Because the gatewell will be located within the levee, it is assumed that the settlement of the structure will be similar to the ground beneath the levee— about 8 inches. This settlement will not be tolerated by the gatewell structure and settlement mitigation will be needed. Settlement mitigation will also be needed for the piping entering and leaving the structure to avoid adverse additional stresses due to differential settlement. Differential settlement is caused when spanning occurs between areas where settlement is allowed to occur naturally and areas where mitigation has been performed.

At the time of this report, it is assumed that preconsolidation will be used to reduce settlement for the Tierrecita Vallejo gatewell structure. Preconsolidation design will be performed for the Tierrecita Vallejo gatewell for the 90-percent design submittal.

2.9.3 Railroad Closure Structure

The Canadian Pacific (CP) Railroad Closure Structure is on roughly level ground in a gap in the levee. Relatively limited baring pressure will be applied by the closure structure. The subgrade soils in this area are largely sandy in nature which also will limit settlement of the levee surrounding this structure. Therefore settlement of the railroad closure structure is anticipated to be within tolerable limits. Settlement of the structure will be calculated upon final design of the structure and will be provided in the 90 percent design submittal.

2.9.4 Preconsolidation Design

Preconsolidation design will be performed for the oxbow gatewell for the 90-percent phase of the project.

2.10 Exploration Trench

An exploration trench to examine the subgrade immediately below the footprint of the levee prior to placement of levee fill are used to identify utilities or drainage pipes through developed areas and previously placed unsuitable fill materials. The exploration trench should be 10 or more feet deep in areas with housing or other foundations and utilities. In natural areas, 6-foot-deep trenches should be appropriate. The trench should be excavated with side slopes that meet OSHA guidelines. The side slopes of the exploration trench should also be benched to facilitate compaction of backfill layers during levee construction.

2.11 Levee Borrow Material and Shrinkage Factor

The proposed Price borrow site has been selected as the source of levee borrow materials for Phases 2 and 3 and is presumed to be the borrow source for the Phase WC-1 portion of the project. The soils encountered at the Price borrow area generally consist of lean clay soils (presumed clayey glacial till) above the Fort Union Group materials. The lean clay glacial till materials are recommended as the primary levee fill materials for Phase WC-1.

The permeability and shear strength characteristics of the clayey glacial till borrow materials from the Price site are discussed in Section 2.6.3.4. The shrinkage factor for levee fill material is estimated to be 5 to 10 percent. USACE guidance, EM 1110-2-1913 (*Design and Construction of Levees*, reference [15]) suggests that a shrinkage factor of at least 25 percent be included to account for material shrinkage during placement and material losses during excavation and hauling.

3.0 River Hydrology and Hydraulic Analysis

3.1 Overview

The hydrologic and hydraulic analysis discussed in this section of the BDR builds on previous reports for the MREFPP.

- Preliminary Engineering Report (PER) (reference [2]),
- MREFPP—Hydrologic and Hydraulic Modeling Report (2013 H&H Report) (reference [8]),
- MREFPP—Phases MI-2 and MI-3 Basis of Design Report (Phases MI-2 and MI-3 BDR) (reference [18]),
- MREFPP—Phase BU-1 Basis of Design Report (Phase BU-1 BDR) (reference [18]).

The purpose of river hydrology and hydraulic analysis is to:

- document existing hydraulic conditions
- calculate water surface elevations and velocities used to design Project features
- quantify Project impacts
- evaluate risk and uncertainty associated with interim hydraulic conditions when only some MREFPP segments are in place and with full MREFPP conditions

Hydraulic interdependencies require that the river hydrologic and hydraulic analysis check for impacts upstream and downstream of the proposed Tierrecita Vallejo levee system.

3.2 Hydrologic Analysis

The hydrologic analysis for Phase WC-1 used the same hydrology data that was used in the Phase MI-2 and MI-3 BDR. Below is a brief summary of the hydrologic data used in the hydraulic modeling. Additional information is available in Appendix C of this BDR. Full documentation is provided in the Phase MI-2 and MI-3 BDR (reference [18]).

3.2.1 Hydrologic Analysis from Prior Reports

Hydrographs from local drainage, or ungaged tributaries, associated with the 2009, 2010, and 2011 historic flood events were defined using a HEC-HMS model of the Mouse River Basin as part of the 2013 H&H Report (reference [8]). No updates were made to Mouse River Basin HEC-HMS model for this BDR.

Regulated, balanced hydrographs at the Foxholm, Minot, and Verendrye gaging stations and coincidental hydrographs at the inflow locations between the USGS gaging stations were developed for the Phase MI-2 and MI-3 BDR (reference [18]). Balanced hydrographs were developed to simulate intermediate flood peaks not represented by the 2009, 2010, and 2011 historic flood hydrographs. A discharge frequency was assigned to each of the balanced hydrographs based on the regulated discharge-frequency curve for the Mouse River (reference [5]).

Flood Hydrograph Name	Instantaneous Peak Flow at Minot (cfs)	Type	Source
2009	2,850	Historic	USGS stream gage measurements (1)
2010	820	Historic	USGS stream gage measurements (1)
2011	27,400	Historic	USGS flow measurements at Broadway bridge (1)
10-year	2,600	Synthetic (Balanced)	HEC-ResSim modeling (2)
25-year	5,000	Synthetic (Balanced)	HEC-ResSim modeling (2)
50-year	5,100	Synthetic (Balanced)	HEC-ResSim modeling (2)
75-year	6,800	Synthetic (Balanced)	HEC-ResSim modeling (2)
100-year	10,000	Synthetic (Balanced)	HEC-ResSim modeling (2)
200-year	14,200	Synthetic (Balanced)	HEC-ResSim modeling (2)
500-year	20,000	Synthetic (Balanced)	HEC-ResSim modeling (2)

(1) USGS Water Data for North Dakota (reference [19])
(2) Modeling documentation is in the Phase MI-2 and MI-3 BDR (reference [18])

3.3 Hydraulic Modeling

The hydraulic modeling for the Phase WC-1 builds on the modeling completed for previous design phases. The Phase MI-2 and MI-3 BDR (reference [18]) documents the calibration of the existing conditions model, the creation of created baseline scenarios for assessing impacts, and the simulation of future condition scenarios representing interim and full MREFPP conditions. The hydraulic modeling for Phase WC-1 focuses on the hydraulic impacts of design refinements for the Tierrecita Vallejo levee. Figure 3-1 is a location map showing hydraulic modeling extents. See Appendix C for additional details on the hydraulic modeling completed for this BDR.

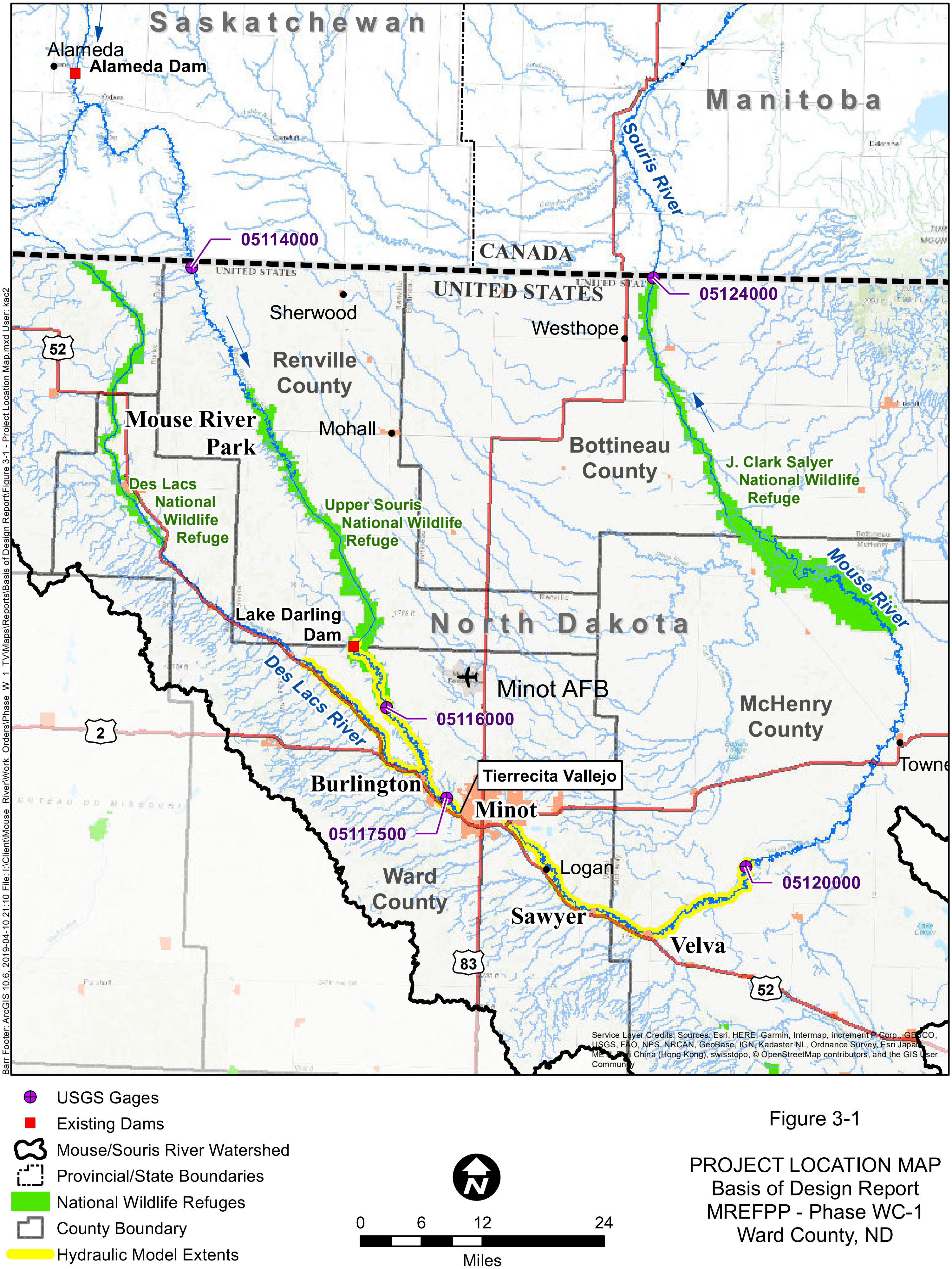


Figure 3-1

PROJECT LOCATION MAP
Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

3.3.1 Model History

Hydraulic modeling of the Mouse River dates back to the 1970s when the USACE began designing flood risk reduction levees for the river valley. The USACE updated its models as projects were designed and constructed. The USACE models were updated during the 2011 flood fight.

Barr built on the USACE models to create a steady state HEC-RAS model for the Preliminary Engineering Report in 2012 (reference [2]). In 2013, Barr converted the steady state model to an unsteady state model that simulated the Mouse River from where its upstream US/Canadian border crossing to its downstream border crossing (reference [8]). Unsteady flow modeling allows the project to evaluate both upstream and downstream impacts for future conditions hydraulic scenarios.

In 2016, Barr truncated the unsteady model to focus on the Lake Darling to Verendrye reach of the Mouse River as part of the Phase MI-2 and MI-3 BDR (reference [2]). The 2016 model was used for a programmatic Section 408 approval of the larger plan for all phases of the project. Subsequent iterations of the MREFPP models have been refinements that reflect hydraulic changes due to the design of individual Project phases for Burlington (Phase BU-1), Maple Diversion (Phase MI-4), and the east tie-back (Phase MI-5).

3.3.2 Existing Conditions Models

An existing conditions model representing conditions during the 2011 flood fight was calibrated and validated as part of the Phase MI-2 and MI-3 BDR (reference [18]). The 2011 MREFP model simulated the flood fighting that occurred during the 2011 flood. For the evaluation of impacts, the calibrated model was copied to create two baseline models representing existing conditions as of 2015. Baseline 1 represents existing conditions with no flood fighting. Baseline 2 represents existing conditions with a successful flood fight along existing federal levees for the 10,000 cfs flood event. Previous MREFPP reports evaluated impacts for both baseline scenarios. However, permitting agencies have focused on Baseline 1 impacts when reviewing previous design phases. Therefore, the modeling and analysis for Phase WC-1 will be limited to Baseline 1 scenarios.

3.3.3 With-Project Models

The MREFPP will take many years to be designed, permitted, and constructed. The Phase MI-2 and MI-3 BDR (reference [18]) defined Construction Stages that represent hydraulic conditions at milestones for completing major portions of the Project. Permitting of individual design segments has required creation of addition interim conditions scenarios to evaluate the impacts of constructing a specific design phase. Table 3-1 provides definitions for each Construction Stage. Appendix C provides a detailed matrix of the individual design phases that make up the major Construction Stages.

The hydraulic modeling for the Phase WC-1 BDR focused on two future conditions hydraulic scenarios, Construction Stage 1.5 and Construction Stage 4. Construction Stage 1.5 represents the floodplain hydraulics after completion of the North Minot levee system (Phases WC-1 and MI-1, 2, 3, 4, 5). This modeling scenario is needed to evaluate how completion of this independent levee system will affect flood risk for the community. Construction Stage 4 represents the floodplain hydraulics after all MREFPP segments have been constructed. The Construction Stage 4 modeling scenario is needed to define design elevations for Phase WC-1 and to confirm that design decisions for Phase WC-1 do not significantly change the design hydraulics for previously designed Project phases.

Table 3-1 Construction Stage Definitions

Construction Stage	Description
1	Phase MI-1, MI-2, MI-3, Broadway bridge and U.S. Highway 83 bypass bridge replacements
1.5	+ Phase BU-1, WC-1, MI-5, MI-4 (Burlington, Tierrecita Vallejo, East Tie-Back, Maple Diversion)
2	+ 27th Street Diversion, remaining North Minot levees
3	+ South Minot levees, Velva, Sawyer, some Ward County levees
4	+ remaining Ward County levees

3.3.4 Hydraulic Uncertainty Modeling

Hydraulic uncertainty modeling for the entire project was completed as part of the Phase MI-2 and MI-3 BDR (reference [18]). The hydraulic uncertainty modeling will be reviewed and updated, if necessary, to reflect the design of Phase WC-1 and in the 90% BDR.

3.4 Hydraulic Design

The HEC-RAS modeling was used to inform the design of the Phase WC-1 levee system. Design flood profile elevations were used to set design elevations for Project features and inform the geotechnical analysis of bank stability. Channel velocities were used to design bank stability and scour protection measures. The hydraulic design for the full project was documented in the Phase MI-2 and MI-3 BDR (reference [18]).

3.4.1 Design Flood Event

The design flood event for Phase WC-1 is the peak flow from the 2011 flood event, which was measured at 27,400 cfs at the Broadway bridge in Minot. The MREFPP hydraulic models use the unsteady flow routine in HEC-RAS to simulate the routing of flood hydrographs, so actual peak flows vary by location.

3.4.2 Uncertainty

In the flood-damage reduction planning of this Project, the following three types of uncertainty were considered: (1) hydrologic uncertainty, (2) natural hydraulic uncertainty or variability in the estimated rating curves, and (3) model hydraulic uncertainty arising from the use of a hydraulic model to describe complex hydraulic phenomena.

3.4.2.1 Hydrologic Uncertainty

The hydrologic uncertainty was estimated as part of the Phase MI-2 and MI-3 BDR (reference [18]) by performing a discharge-probability analysis on 76 years of unregulated flow data at the USGS gage above

Minot from a calibrated HEC-ResSim reservoir routing model. This is the continuous simulation method described in Table 4-1 of EM 1110-2-1619 (reference [20]).

The resulting discharge cumulative density function with confidence limits (uncertainty) was used at all locations along the Mouse River. The mean log, standard deviation, and skew of the fit to the 76 years of data (systematic record length) are 3.183, 0.508, and -0.128, respectively. These resulting values were directly entered into the HEC-FDA analysis to create the unregulated exceedance probability function with uncertainty (hydrologic uncertainty).

To represent hydrologic uncertainty in the HEC-FDA analysis for the regulated system, unregulated-regulated transform functions were developed for two distinct regions of the study area: Minot through Burlington based on the USGS gage above Minot, and downstream of Minot based on the USGS gage at Verendrye. Additional details are in Appendix C.

3.4.2.2 Hydraulic Uncertainty

Hydraulic uncertainty was evaluated as part of the Phase MI-2 and MI-3 BDR (reference [18]) following methods outlined in EM 1110-2-1619 (reference [20]). Proposed conditions hydraulic uncertainty will be reviewed and updated, if necessary, for the 90% BDR to reflect Phase WC-1 design. The uncertainties of the natural system and hydraulic models for both existing conditions and proposed conditions are discussed below.

Existing Conditions

Total hydraulic uncertainty for existing conditions is combination of natural uncertainty and model uncertainty. Results of the uncertainty estimates for existing conditions are summarized in Table 3-2.

Table 3-2 Standard Deviation or Uncertainty Estimates for Existing Conditions

Hydraulic Uncertainty	Burlington and Minot Area USGS Gaging Station above Minot		Sawyer and Velva USGS Gaging Station near Verendrye	
	σ for High Flows (feet)	σ for Lower Flows (feet)	σ for High Flows (feet)	σ for Lower Flows (feet)
Natural Uncertainty (feet)	0.16 (1)	0.93 (2)	0.24 (3)	0.69 (4)
Model Uncertainty (feet)	0.45 (5)	0.43 (6)	0.81 (7)	0.49 (8)
Total Uncertainty (9) (feet)	0.5	1.0	0.8	0.8

(1) Based on the fit of measured stage-flow data above Minot and for flows greater than 6,000 cfs
(2) Based on the fit of measured stage-flow data above Minot and for flows between 1,500 cfs and 6,000 cfs
(3) Based on the fit of measured stage-flow data at Verendrye and for flows greater than 6,000 cfs
(4) Based on the fit of measured stage-flow data at Verendrye and for flows between 1,500 cfs and 6,000 cfs
(5) Based on the fit of the calibrated HEC-RAS model to the 2011 HWMs upstream of Highway 2
(6) Based on the fit of the calibrated HEC-RAS model to the 2009 and 2010 HWMs upstream of Highway 2
(7) Based on the fit of the calibrated HEC-RAS model to the 2011 HWMs downstream of Highway 2
(8) Based on the fit of the calibrated HEC-RAS model to the 2009 and 2010 HWMs downstream of Highway 2
(9) Sum of variance; based on Equation 5-6 of EM 1110-2-1619 (reference [20])

Proposed Conditions

Total hydraulic uncertainty for proposed conditions is a combination of natural uncertainty and model uncertainty.

- **Natural uncertainty** was kept the same as it was defined for existing conditions because it is representative of natural variability which is not affected by the Project.

- **Model uncertainty** was accounted for with a sensitivity analysis of some of the calibrated model parameters. The Manning's n values and the weir coefficients (for all inline, lateral, and bridge structures) were modified, both +/- 20 percent, to see the impact that uncertainty in these parameters would have on the water surface profile.

  The model uncertainty along the Mouse River reach was compared to the suggested minimum values in Table 5-2 of EM 1110-2-1619 (reference [20]). A minimum model uncertainty of 0.3 feet was selected from the table because the hydraulic model is based on good LiDAR data, incorporated in-channel bathymetry data, and was calibrated to a substantial number of HWMs.

- **Total hydraulic uncertainty** at index stations for with-Project conditions was estimated as the sum of the variance as defined by Equation 5-6 of EM 1110-2-1619 (reference [20]).

The total with-Project hydraulic uncertainty (2σ) at the high-flow end of the rating curve (near the Project design flow) ranges from 0.5 feet to 2.2 feet (most under 2 feet) at the locations where the risk and uncertainty analysis was completed. Additional details of the hydraulic uncertainty are in the Phase MI-2 and MI-3 BDR (reference [18]).

3.4.3 Superiority

The consequences of overtopped flood risk reduction projects can be significant and costly. Projects designed for superiority allow initial, controlled overtopping to occur at a predetermined and least damaging location in the levee system.

Structural superiority for flood risk reduction systems generally involves adding height to project features to control the location of overtopping for a flood event that exceeds the capacity of the system. The difference between the overtopping flood profile and final design grade of the proposed levee system is superiority. The final design grade of the proposed levee system is the expected elevation of the levee crest along the levee alignment. The overtopping flood profile is defined based on the hydraulic uncertainty for the modeled system. Figure 3-2 illustrates how an overflow location is lower than other segments of the system, but higher than the design flood elevation to account for model uncertainty.

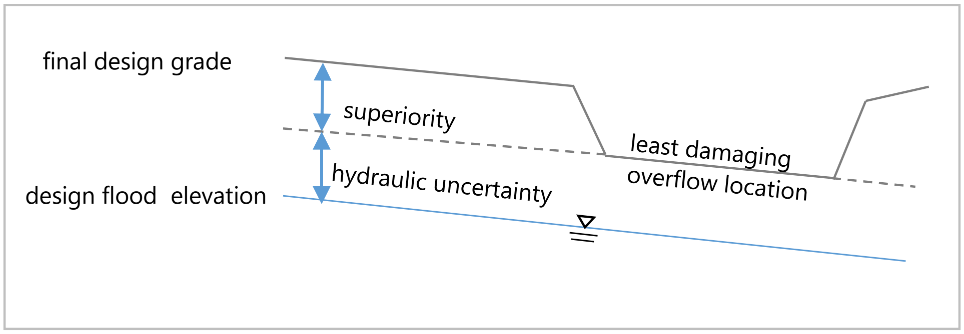


Figure 3-2 Superiority definition

The sponsor has provided the guidance that the top of levee, floodwall, and closure structure elevations be set at 3 feet above the design water flood elevation. Anything above the minimum top elevation defined by hydraulic uncertainty will be additional height used to establish superiority for the different flood risk reduction systems.

The Tierrecita Vallejo levee is one segment in the larger North Minot levee system -- consisting of Phases WC-1, MI-1, MI-2, MI-3, MI-4, and MI-5 -- that is on the north side of the Mouse River extending from the upstream end of the Tierrecita Vallejo development to high ground north of Roosevelt Park in Minot. The least damaging overtopping location is typically near the downstream end of a levee system so overtopping floodwaters must flow back up into the interior area. For the North Minot levee system, this would mean the least damaging overtopping location would be in the Phase MI-5 reach of the levee system, currently under design by others.

3.4.4 Design Water Surface Elevations

Design water surface elevations are used to set the minimum height for project features. The original project definition set the minimum design project final grade for levees, closures, and floodwalls at three feet above the design flood elevation. The hydraulic design verified that the minimum design project final grade allows for hydraulic uncertainty and superiority. Table 3-3 summarizes the different components that make up the minimum design project final grade elevations. Final design elevations may be greater than the minimum to allow for settlement and simplify civil grading.

Table 3-3 Design Water Surface Elevations for Phase WC-1

Hydraulic Model Cross-Section	Design Flood Elevations(1) (feet)	Hydraulic Uncertainty(2) (feet)	Minimum Top of Levee at 95% CNP(3) (feet)	Minimum Superiority(4) (feet)	Minimum Design Project Final Grade(5) (feet)	Location Reference
1221172	1570.9	1.1	1572.0	1.9	1573.9	
1214933	1570.0	1.5	1571.5	1.5	1573.0	RR bridge
1212369	1569.7	1.6	1571.3	1.4	1572.7	
1210972	1569.6	1.6	1571.1	1.4	1572.6	
1210061	1569.4	1.6	1570.9	1.4	1572.4	
1209372	1569.3	1.6	1570.9	1.4	1572.3	
1209143	1569.2	1.6	1570.8	1.4	1572.2	
1208985	1569.2	1.6	1570.7	1.4	1572.2	grade control str.
1208739	1569.1	1.6	1570.7	1.4	1572.1	
1208399	1568.9	1.6	1570.5	1.4	1571.9	U.S. Highway 83 bypass bridge
1208167	1568.8	1.7	1570.5	1.3	1571.8	

(1) Construction Stage 4 with-Project model version 3.1.0
(2) Hydraulic uncertainty is discussed in Section 3.4.2
(3) Minimum Top of Levee at 95% Conditional Non-Exceedance Probability (CNP) = current model design flood elevations plus hydraulic uncertainty.
(4) Minimum Superiority = Project (Final) Grade minus Minimum Top of Levee Grade (top of levee at 95% CNP).
(5) Minimum Design Project Final Grade, based on the Construction Stage 4 with-Project model plus 3 feet.

3.4.5 Slope Stability Water Surface Elevations

The MREFPP hydraulic models were used to calculate low, normal and design flood water surface elevations in support of the geotechnical slope stability analysis. Table 3-4 lists the water surface elevations associated with the geotechnical cross sections used for the slope stability analysis. The methodology for calculating the low and normal water surface elevations is described in Appendix C.

Table 3-4 Slope Stability Water Surface Elevations

Geotech Cross Section	HEC-RAS Cross Section	Water Surface Elevations (Feet) NAVD88		
		Low Water (Q=7 cfs)	Normal Water (Q=37 cfs)	Design Flood (Q=27,400 cfs)
W-1-1	1221172	1550.1	1550.3	1570.9
W-1-2	1214933	1550.1	1550.3	1570.0
W-1-3	1212369	1550.1	1550.3	1569.7
W-1-4	1209372	1550.1	1550.3	1569.3
W-1-4-oxbow	1209143	1550.1	1550.3	1569.2
W-1-5	1208739	1547.1	1548.7	1569.1

3.4.6 Erosion Protection and Scour Analysis

During the 2011 flood event, there was significant scour under the U.S. Highway 83 bypass bridge and on the south bank of the channel upstream of that bridge, which suggests that there is potential for scour and erosion to occur during the design event. After the 2011 flood, scour protection was installed to minimize the potential for future bridge scour. Upstream of the bridge, the channel banks were restored and armored with riprap. In 2018, the NDDOT replaced the U.S. Highway 83 bypass bridge with a design that has a larger hydraulic opening. The bridge replacement included riprap revetment under the bridge to minimize the potential for scour and bank erosion.

The potential for erosion and scour was analyzed as part of the civil design to determine the type and extents of erosion protection needed for this phase of the project. Velocities from the HEC-RAS Construction Stage 1.5 and Construction Stage 4 models were compared to existing conditions velocities to assess whether velocities are high enough to be a concern for erosion, whether the project would increase the potential for erosion, and whether erosion protection will be needed.

Average velocities within cross sections are highest for the design flood event (27,400 cfs). Under existing conditions as of 2015, average velocities range from three to seven feet per second, with the highest velocities occurring near the CP railroad bridge and U.S. Highway 83 bypass bridge. Away from the bridges, the overall average velocity is a little more than 4 fps.

Proposed conditions velocities were checked for Construction Stage 1.5 and Construction Stage 4. In both scenarios, average channel velocities range from 3 to 6 fps, which reflects a general decrease in velocities at most cross sections. The general reduction in velocities is due to the larger opening under the U.S. Highway 83 bypass bridge and from proposed overbank excavation on the north bank adjacent to the levee.

With velocities greater than 3 fps, some erosion protection is needed. Protection of the proposed levee from slope failure and lateral erosion of the stream banks during flood events will be accomplished through a combination of turf reinforcement mat, vegetation, and existing riprap.

Long term degradation of the channel is unlikely because of the existing grade control structure upstream of the U.S. Highway 83 bypass bridge. A series of grade control structures upstream and downstream of Phase WC-1 make it unlikely for headcutting of the channel to occur.

Channel scour is a possibility beneath the U.S. Highway 83 bypass bridge due to the contraction of flow during the design flood without adequate protection. The NDDOT completed a scour analysis as part of the bridge hydraulics report for the U.S. Highway 83 bypass bridge replacement project (reference [21]). The report recommended riprap in the channel beneath the bridge and extending 65 feet upstream of the bridge and 70 feet downstream of the bridge, which the NDDOT installed during bridge construction. Therefore, Phase WC-1 design did not complete a separate scour analysis.

3.4.7 Ice Jam Analysis

Ice jams were considered as part of the Phase MI-2 and MI-3 BDR (reference [18]) used for the programmatic Section 408 submittal and found not to be a major design consideration. Ice jams have been observed on the Mouse River in Burlington and Minot. However, few have been reported in developed areas since the USACE channelization project was completed in the 1970s and 80s. Mouse River ice jams are most likely to form for flows in the 1000 to 3000 cfs. The MREFP is designed for 27,400 cfs, which will provide levee systems that are 10 to 15 feet above the 3000 cfs flood profile. The recently replaced U.S. Highway 83 bypass bridge has an even larger hydraulic opening, which would further reduce the potential ice jams to occur along the Phase WC-1 levee system.

3.4.8 Design Modifications and Alternatives

The following modifications were made to the PER proposed features for Phase WC-1 that had the potential to affect the hydraulics of the design flood.

- Overbank excavation was eliminated from the south bank of the river across from the Tierrecita Vallejo levee.
- The north/south portion of the levee on the west side of Tierrecita Vallejo was shifted to the west, closer to the river.
- Minor shifts were made to the levee alignment near the U.S. Highway 83 bypass bridge.

The proposed conditions hydraulic models were updated to reflect these design changes. The Construction Stage 4 design flood profile did not significantly change as a result of these changes relative to Phase MI-2 and MI-3 BDR version of the full project model.

3.5 Impacts Analysis

The design of the Phase WC-1 levee has changed very little from what was presented in the Phase MI-2 and MI-3 BDR for the programmatic Section 408 submittal.

3.5.1 Regulatory Floodplain Analysis

The Phase WC-1 design reviewed potential impacts to both the effective and preliminary floodway maps. The FIS for Ward County, North Dakota, and incorporated areas was published in February 2002 (reference [6]). The NDSWC and FEMA are in the process of revising the Ward County FIS. The revised FIS will update the discharge-frequency curve such that the 1-percent AEP discharge increases from 5,000 to 10,000 cfs. The preliminary FIS and flood hazard mapping was published in 2017. As of spring 2019, the preliminary maps were being revised to address public comments. The revisions are expected to result in a regulatory floodway definition that does not extend landside of existing levees.

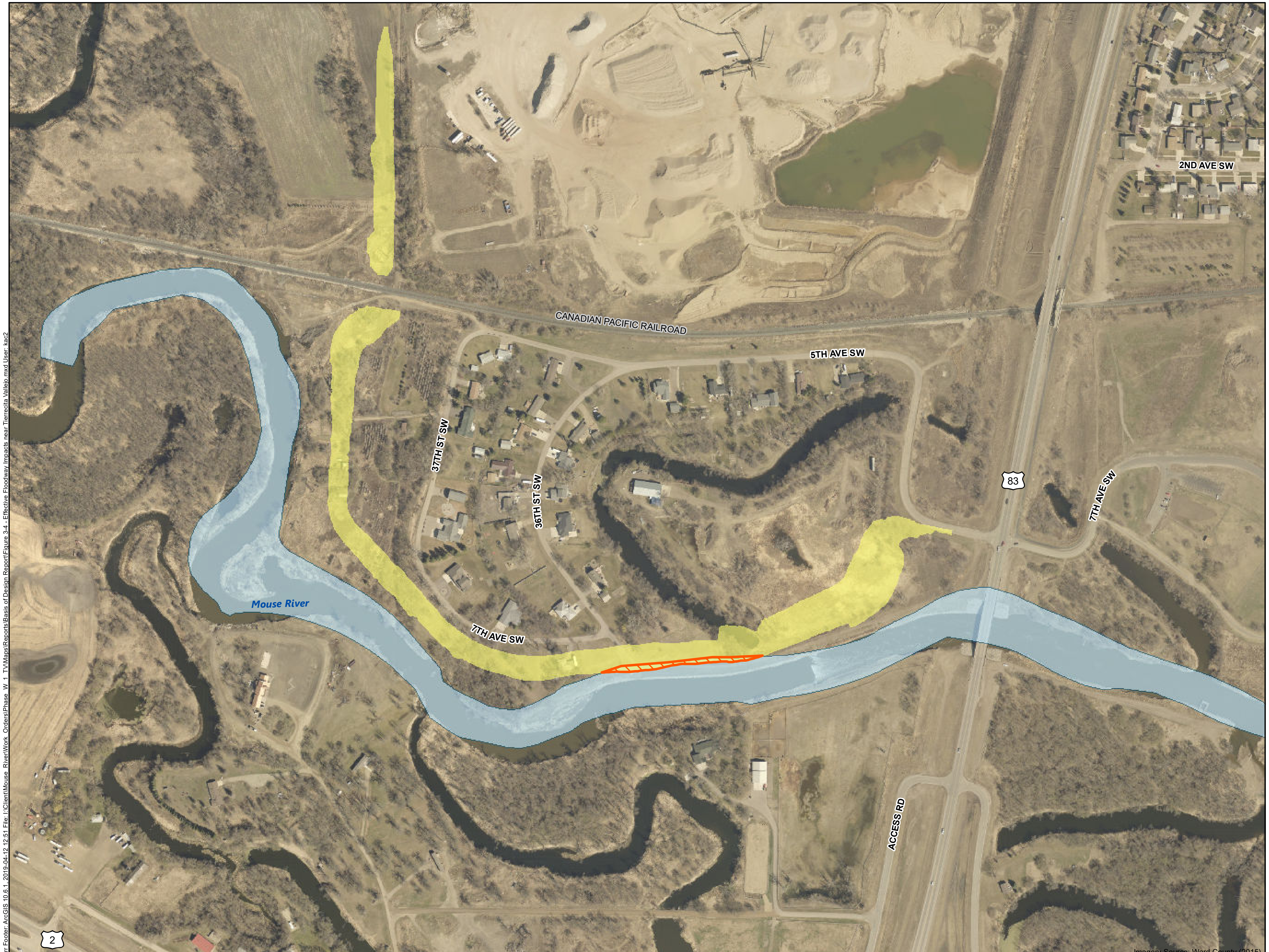
The local sponsor, USACE, FEMA, HEI, and Barr have discussed the potential impacts of the larger MREFPP on the regulatory floodway and floodplain. Typically projects are permitted with FEMA based on impacts to the effective floodway. However, given the substantial increase to the regulatory discharge, permitting of impacts for this project will be based on 10,000 cfs from the preliminary FIS rather than 5,000 cfs from the effective FIS.

A CLOMR was submitted to FEMA for Construction Stage 1.5, which includes anticipated impacts from Phase WC-1. The design of Phase WC-1 has not changed the anticipated impacts presented in the original CLOMR application.

Potential impacts to the effective floodway are shown in Figure 3-3. The proposed levee footprint overlaps a small portion the effective regulatory floodway. However, the aerial imagery in the figure shows that the floodway delineation for this reach of the Mouse River does not closely follow the actual channel alignment. If the floodway was centered over the channel, the levee footprint would not overlap the effective floodway.

Potential impacts to the preliminary regulatory floodway are shown in Figure 3-4. The initial mapping of the revised floodway extended the floodway behind the existing levee on the south side of Tierrecita Vallejo. FEMA has indicated that the on-going revisions to the floodway mapping will shift the floodway so that it does not extend behind the existing levee. The updated maps are expected to be available for the Phase WC-1 90% design submittal. Figure 3-4 will be updated to reflect the updated maps.

If there are still floodway impacts with the revised floodway delineation, the project will submit a CLOMR. Construction of the Project is not anticipated to begin until fall 2020 or later. The schedule for Project permitting and construction allows time to work through the CLOMR process with FEMA, if needed.



FEMA Effective Floodway

Potential Impacts to Effective Floodway

Levee System Footprint

N

0 175 350 700

Feet

BARR

ACKERMAN ESTVOLD

Figure 3-3

EFFECTIVE FLOODWAY IMPACTS NEAR TIERRECITA VALLEJO

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

FEMA Preliminary Floodway
Potential Impacts to Preliminary Floodway
Levee System Footprint

N

0 175 350 700
Feet

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ACKERMAN ESTVOLD

Figure 3-4

PRELIMINARY FLOODWAY IMPACTS NEAR TIERRECITA VALLEJO

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

3.5.2 Existing Conditions

The existing conditions model set the baseline flow, water level, and inundation conditions for comparison to future conditions scenarios. For the Phase WC-1 BDR, impacts were only assessed relative to Baseline 1, which represents 2015 existing conditions with no flood fighting.

3.5.3 Stage 1.5 Impacts

This section presents water surface profile impacts and discusses changes in risk and uncertainty for Construction Stage 1.5, which would complete the North Minot levee system (Phases WC-1 and MI-1, 2, 3, 4, 5). This hydraulic scenario was evaluated because completion of the Tierrecita Vallejo segment of the North Minot levee system is expected to happen at the same time as construction of the Maple Diversion. Together these segments would create a FEMA certifiable levee system for North Minot. The water surface profiles and the impacts for this hydraulic scenario are detailed in tabular and graphical form in Appendix C.

3.5.3.1 Water Surface Profile Impacts

The water surface profile comparison plot below shows how Mouse River hydraulic profiles will change relative to Baseline 1 once the Tierrecita levee and the other Construction Stage 1.5 features are in place. Figure 3-5 shows changes in the hydraulic profiles for the 2011 historic flood event and the 10-, 50-, 100-, and 200-year synthetic flood events.

Modeling suggests that impacts to water surface profiles will tend to occur adjacent to Project elements during large flood events.

Baseline 1 | Construction Stage 1.5 | Events: 2011, 10-, 50-, 100-, 200-Year

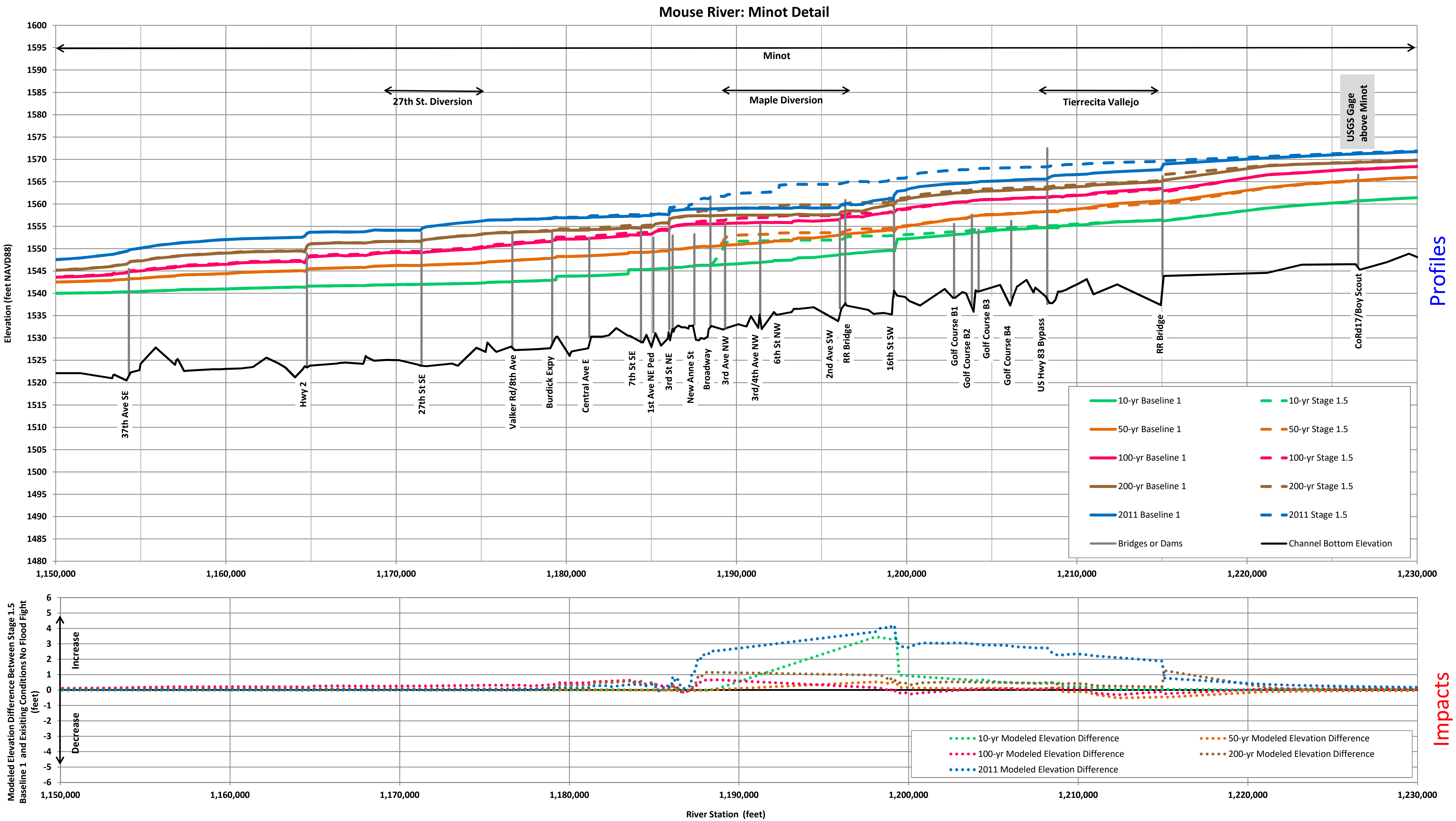


Figure 3-5 Water Surface Profile Comparison; Stage 1.5 vs. Baseline 1

P:\Mpls\34 ND\51\34511010 MREFP\WorkFiles\T09_W-1\Design\H_H\Profiles\S1.5_v2.4.3\S1.5 v2.4.3 vs B1 v2.1.9.xlsx
printed: 4/29/2019

3.5.3.2 Risk and Uncertainty Impacts

To evaluate the risk of levee overtopping with and without the Project and during construction stages, a risk and uncertainty analysis was performed using HEC-FDA as part of the Phase MI-2 and MI-3 BDR (reference [18]). During Construction Stage 1.5, the AEP (median and/or expected) increased at only a few locations in or immediately adjacent to Minot. Increases in the AEP ranged from 0.1 to 0.4 percent; most were 0.1 percent.

Conditional non-exceedance probability (CNP) is defined as the probability that an event will be contained by the Project. At locations where Project elements are in place during Construction Stage 1.5 assurance (CNP) for non-overtopping is 0.999 for all events up through 500 years. This is a significant improvement for index stations altered by the Project. Risk analysis did, however, identify locations where assurance of non-overtopping decreased—primarily in Minot. In Minot the decrease was approximately 1 percent for the 1% AEP event for most locations. Outside of Minot, decreases in CNP during the 1% AEP event were approximately 1 percent or less.

3.5.3.3 Inundation Area Impacts

The design profile and inundation area impacts have not changed substantially from the Phase MI-2 and MI-3 BDR (reference [18]). Copies of the Stage 1.5 inundation maps from the Phase MI-2 and MI-3 BDR are provided in Appendix C. These maps will be updated for the Phase WC-1 BDR 90% submittal. A summary of the inundation area impacts is provided below.

The assessment of inundation area impacts mapped how the Construction Stage 4 would change the location and extents of inundation for the 5,000 cfs, 10,000 cfs, and 27,400 cfs flood events. At 5,000 cfs, there would be little change in the total inundation area for each of the three areas analyzed because flows are generally contained within the channel. At 10,000 cfs, there would be a net decreases in the total inundation area because of the constructed levee segments. At 27,400 cfs, there are newly inundated areas on the order of 30 to 40 acres upstream of Minot, and newly inundated areas on the order of 20 to 60 acres within Minot. However these increases are offset by a reduction of over 600 acres upstream of Minot and nearly 2000 acres within Minot.

3.5.3.4 Structure and Parcel Impacts

The design of Phase WC-1 does not substantially change the design profile for the larger MREFPP, so the structure and parcel impacts analysis completed for as part of the Phase MI-2 and MI-3 BDR (reference [18]) is still representative of the expected impacts. Below is a brief summary of those impacts with figures highlighting the expected impacts at each major construction stage.

The structure impacts analysis quantified changes in the number of structures inundated by the 10,000 cfs and 27,400 cfs flood events over the course of Project implementation. A structure could be a home, a shed, a garage, or a business. Structures were identified using GIS and the 2015 LiDAR data. Some parcels have multiple structures. Structure impacts are presented based on number of structures inundated and number of parcels with at least one structure that is inundated.

Figure 3-6 shows the expected number of structures inundated within Minot. Figure 3-7 shows the number of Minot parcels with at least one inundated structure. For each construction stage, inundated structures and parcels are broken down into the number that would remain inundated and the number that would be newly inundated, relative it existing conditions. Impacts are show for both Baseline 1 (no flood fight) and Baseline 2 (10,000 cfs flood flight).

Additional discussion and results for structure and parcel impacts are in the Phase MI-2 and MI-3 BDR (reference [18]).

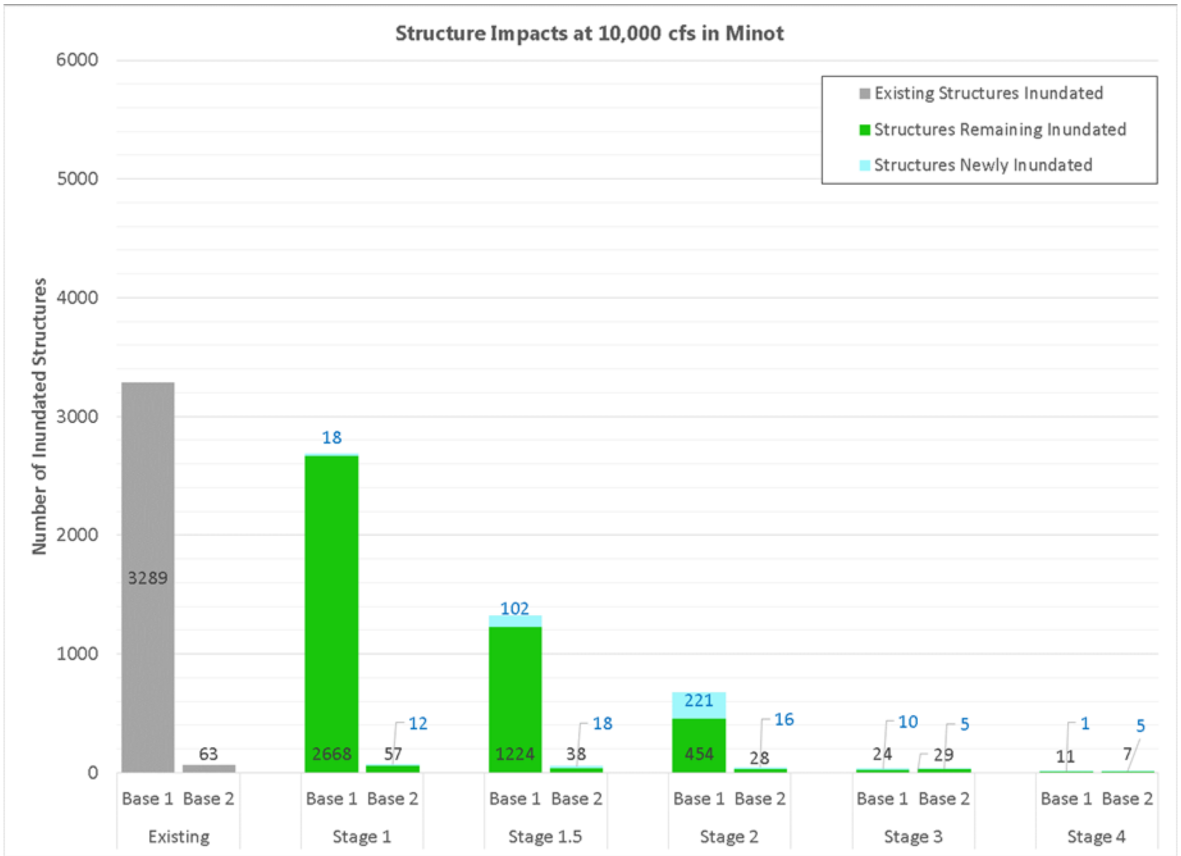


Figure 3-6 Inundated Structure at 10,000 cfs in Minot

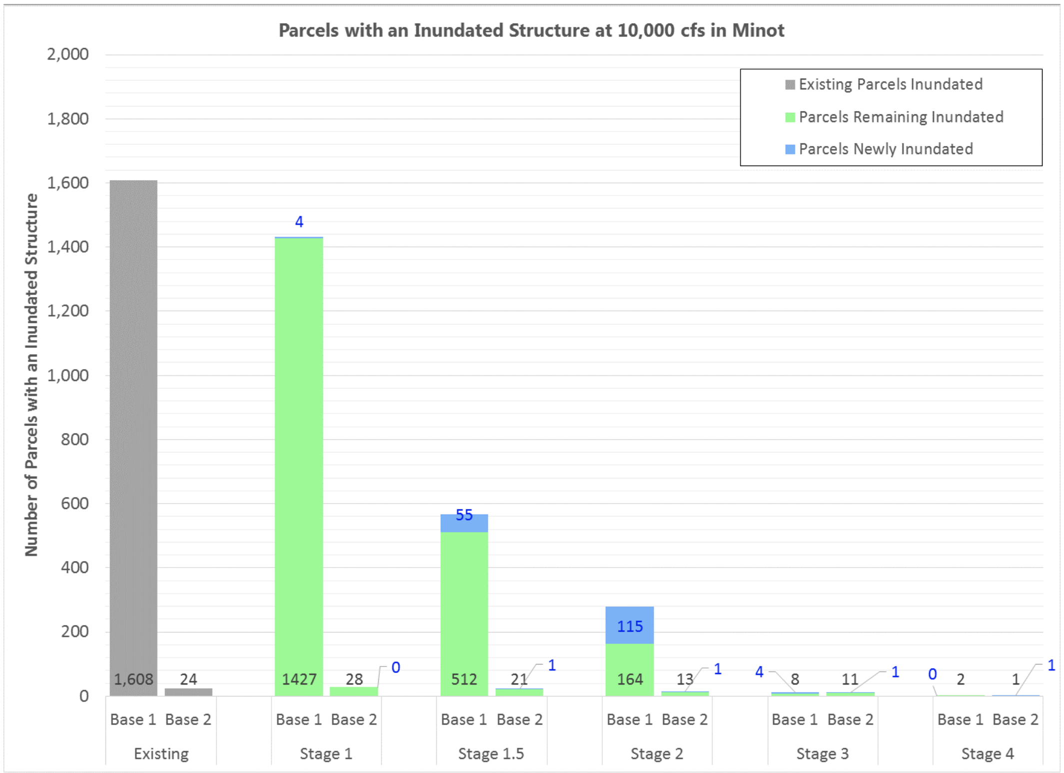


Figure 3-7 Parcels with an Inundated Structure at 10,000 cfs in Minot

3.5.3.5 Depth Duration Frequency Impacts

The design of Phase WC-1 does not substantially change the design profile for the larger project, so the depth duration frequency analysis completed as part of the Phase MI-2 and MI-3 BDR (reference [18]) is still valid. Below is a brief summary of that analysis.

Depth duration frequency curves show how the duration of inundation at a given elevation changes relative to a given flood event return frequency. Depth duration frequency plots were created at 19 locations associated with bridges over the Mouse River. Each set of curves shows plots for the 10-, 50-, 100-, 200-year events and the 2011 event. The solid lines represent existing conditions. The dashed lines represent proposed conditions for a given construction stage. Figure 3-8 shows the depth duration frequency curves for Sixteenth Street bridge.

Further discussion of depth duration frequency impacts and a full set of depth duration frequency curves at all 19 locations are in the Phase MI-2 and MI-3 BDR (reference [18]).

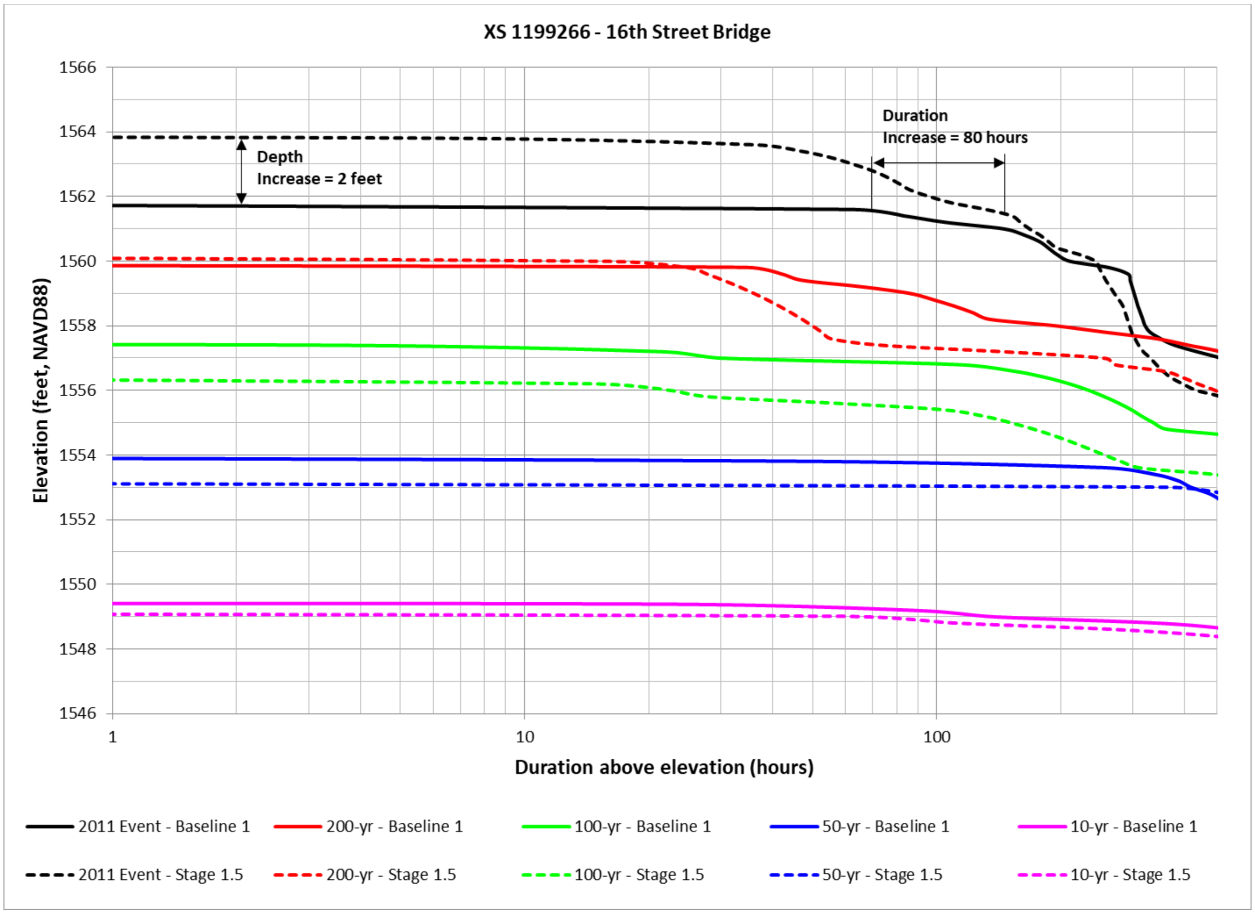


Figure 3-8 Depth Duration Frequency Curves; Sixteenth Street Bridge Stage 1.5

3.6 Uncertainties considered and related risks

Hydrologic and hydraulic analysis use measurements, data, and simulations to approximate how moving water responds to the natural environment. Many of the factors used to develop the numeric representation of these natural processes have some inherently unknown information and/or uncertainty associated with them. Even factors that are directly measured have some level of associated uncertainty (e.g., surveyed high-water marks). The methods by which each of these factors are individually handled are unique to the numeric methods and the goal or intent of the analysis. The unknowns, uncertainties, and potential inaccuracies associated with this analysis are described below to inform the SRJB, Minot and the community of the potential current and future risks associated with the design and future considerations. Other factors may exist, but were not considered in the work completed for this design.

- Standard error of flow frequency analysis
- Roughness values of the channel and overbanks
- Flow coefficients of inline weirs and bridges
- Accuracy of measured stage and flow, and developed rating curves

- Climate change effects on hydrology
- Land use changes affecting hydrology
- Planned reservoir operations
- Deviations from the planned reservoir operations

3.6.1 Hydrologic Uncertainty

The standard error of flow frequency analysis describes the confidence limits around the flow rate estimated for a given recurrence interval. The standard error is a function of the parameters of the distribution that fits the estimated annual peak flow rates and the number of annual peak flow estimates. As the number of annual peak flow estimates increases and more information is provided, the standard error decreases and the confidence of flow estimates for particular recurrence intervals increases. For example, for this study, the unregulated flow rate with only a 1-percent annual chance of being exceeded is about 20,700 cfs. However, confidence limits (5-percent to 95-percent) around this flow rate range between 14,200 cfs and 33,200 cfs, which is a large range. Therefore, if the intent of the Project was to design for an event with a particular recurrence interval (e.g., the 1-percent annual chance event) this range will have to be considered.

The confidence limits for FEMA's unregulated discharge frequency curve were considered and accounted for in the risk and uncertainty analysis (Section 3.4.2) in estimating the probability of the flood risk-reduction features overtopping based on the present understanding. However, the SRJB elected to commission a Project design based on conditions that occurred during the 2011 flood event, the flood of record. The risk associated with this uncertainty is that the Project may be designed for a flow rate that happens more frequently than expected. It is prudent to design the flood risk-reduction features with future adaptability in mind because as additional data are collected, the understanding of the Project's current design flow rate may change, which could result in a reduced or increased risk of the design flood risk reduction system overtopping.

Land use changes can have a similar effect on hydrology, though quantifying the future effect is extremely uncertain. Parts of the Souris River watershed will become more developed in the future if population increases. This will likely result in more developed impervious area and may result in increased runoff volumes and flow rates during storm events. However, with increasing awareness and education on this issue, there are opportunities to offset the development with stormwater best management practices. Therefore, increased development may not result in increased runoff volumes and flow rates if development is carefully managed. Because this factor in hydrology is highly uncertain, and is also somewhat controlled by future decisions, the effect was not accounted for in the hydrologic uncertainty. The risk, believed to be quite low, is that future flood flows will be higher due to increased population and development throughout the watershed. Similar to climate change, it is possible that the flood risk-reduction features of this Project may be tested or overtopped more frequently than we expect. Therefore, it will be prudent to design the flood risk-reduction features with future adaptability in mind.

However, it may be better to properly manage development and stormwater throughout the watershed, rather than rely on adaptation of flood risk reduction measures along the Mouse River.

The effect of climate change on future hydrology is potentially a significant uncertainty. The ongoing USACE feasibility study for the Souris River Basin completed a qualitative assessment of the potential for climate change impacts and non-stationarity for the Project area. The USACE analysis followed the Draft USACE ECB 2016-25 *Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects* (reference [22]), and USACE Engineer Technical Letter (ETL) 1100-2-3 *Guidance for Detecting Non-Stationarities* (reference [23]). The USACE analysis of non-stationarity and trends in the flow records of the Mouse River Basin were found to be not statistically significant (reference [24]).

3.6.2 Hydraulic Uncertainty

Roughness values of the river channel and the overbanks are inherently unknown because they cannot be easily measured. Therefore, estimation, professional judgment, and simulation calibration are necessary to select representative values. Still, a range around the estimated values should be used to capture the remaining uncertainties. The risk associated with incorrectly estimating the roughness values is a poorly estimated water level that directly affects the design elevations of flood risk-reduction features. Over-estimating the roughness will result in water levels that are too high for a given flow rate. Similarly, under-estimating the roughness will result in water levels that are too low. In particular, under-estimating the roughness values will result in lower flood risk-reduction elevations that may be overtopped at flow rates less than expected.

This risk was addressed through calibration of the model to measured water surface elevations (reference [18]) and by incorporating uncertainty ranges in the risk and uncertainty analysis (Section 3.4.2). Representative roughness values of the river channel were first estimated with hydraulic resources and judgment based on physical characteristics of the river, then calibrated by modeling events where flow was contained within the channel. Then roughness values of the overbank areas and channel were calibrated and modified respectively for intermediate and large events.

It is important to note that much of the floodplain area along the developed reaches of the Mouse River between Lake Darling and Verendrye have some form of existing flood risk reduction measures to consider. Therefore the areas on the landside of the features were simulated as storage areas rather than as conveyance area within a given cross-section. This was done to improve the ability of the model to represent emergency flood fight measures and reduce the risk of misrepresenting the complex flow distribution simulated between the overbanks and the channel.

Flow coefficients of inline weirs and for flows overtopping bridges are estimates because they cannot be measured. Professional judgment and calibration are necessary to develop estimates that are representative values. Uncertainties associated with these estimates can be captured by taking a range around the estimated values. The risk associated with estimating these coefficients incorrectly is a calculated water level that inaccurately reflects actual hydraulic conditions, particularly upstream of these features, which directly affects the design elevations of adjacent flood risk-reduction features. Under-estimating the coefficient will result in water levels that are too high for a given flow rate. Similarly, over-estimating the coefficient will result in water levels that are too low. In particular, over-estimating the coefficient will result in lower calculated flood risk-reduction elevations that may be exceeded at flow rates less than expected. This risk was addressed through calibration (reference [18]) and by incorporating uncertainty ranges in the risk and uncertainty analysis. The coefficients of the structures were first estimated with professional judgment, then calibrated by modeling low, intermediate, and high flow events. Uncertainty ranges around the calibrated coefficients were estimated based on professional judgment and used in the risk and uncertainty analysis to provide a range of water surface elevations for a given flow rate.

Estimating a flow rate based on a measured water surface elevation inherently involves error and uncertainty based on the accuracy of measured water surface elevations and flow rates, and the fit of the rating curve to those measured points. The risk associated with this error is that flow is either under- or over-estimated based on a measured stage. This can be particularly important if the goal of the Project is to design for a particular recurrence interval. For example, if the goal was to design for the 1-percent annual chance event, the flow will first be estimated, and then the water surface elevation will be estimated from the flow and rating curve. Without considering the uncertainty and potential errors in the rating curve, the water surface elevation could be under-estimated and overtopping flows could occur more frequently than expected. However, the design approach for the MREFPP is based on the 2011 flood of record; note that the river flow was measured during the 2011 event. There are still uncertainties in measured values and in the relationship between water surface elevation and flow rate. Additionally, incorrectly estimating the design flow rate can impact the estimated water surface elevation under a proposed, altered condition. Therefore, the uncertainty in the rating curves was accounted for in the hydraulic uncertainty in the risk and uncertainty analysis (defined as natural variability).

3.6.3 Operational Uncertainty

Upstream reservoir operations in the Souris River watershed are governed by Annex A of the 1989 International Agreement between the Government of Canada and the Government of the United States of America (reference [25]). Even with the guidance in Annex A, there is uncertainty in how interpretation and implementation can moderately alter the intended operation. The future operation is even more uncertain because operation plans may change and decision making could still be a factor. There are an infinite number of ways the upstream reservoirs could be managed and operated in the future. Because this uncertainty is difficult to quantify, it was not directly accounted for in this analysis. Instead, the analysis assumes the reservoir operations described in Annex A are currently followed as close as possible, and that this plan will not change in the future. If there comes a time when these plans are changed, further analysis will need to be completed to understand the impact of those changes on the Project and the frequency of downstream flooding. The risk associated with not accounting for this uncertainty is that the frequency of future regulated flood flows in the Mouse River are poorly characterized. The flood risk reduction features of this Project could be tested or overtopped more or less frequently than expected as will flooding potential of the essential agricultural lands within the basin.

Significant deviations from the planned operation of the upstream reservoirs and dams are possible and introduce future unknowns. These include extreme events such as gate failures, dam breaches, or other catastrophic events that result in a highly unexpected release or storage of water. These future unknowns are present but are believed to be inappropriate for this study and the design of the Project. The risks and impacts of these potential deviations from expected operations will be better handled in an Emergency Action Plan for each dam. These unknowns were therefore not accounted for in this work.

3.6.4 Resiliency

Project resiliency is integrated into the design in several ways. The Project definition establishes a levee system height that is significantly above the 1-percent AEP design flood elevation that is typical of flood risk reduction projects. The three feet of additional system height above the Project design flood is one to two feet greater than the calculated hydraulic uncertainty, which allows for the creation of superior sections of levee so an overtopping event will happen at the least damaging location. The geotechnical design of the levee is such that the levee could be elevated in the future if needed to increase the non-exceedance probability of the system should changes in watershed hydrology increase the chances of the system experiencing floods larger than the Project design flood.

One way the Project design incorporates some of the potential uncertainty around several of the items describe in this section is by using risk and uncertainty methods and adding superiority above the selected design flow rate. A review of the proposed structural flood risk reduction elements of Phase WC-1 and other levee systems proposed as part of the PER indicates the levee systems will have at least 4 feet of addition system height up to about 20,000 and 25,000 cfs, depending on the system, which is 2 to 2.5 times FEMA's updated preliminary 1-percent AEP discharge estimate and 4 to 5 times FEMA's effective 1-percent AEP published discharge. This suggests the proposed system have the ability to achieve FEMA accreditation in the future if reevaluation of the hydrology and basis of design is needed as more information becomes available relative to climatic conditions. Also, it suggests that the systems will generally overtop near the downstream end.

Generally, the levees were not raised an additional foot around bridges, ends of levees and floodwalls, and constrictions because the levees are already 8 to 10 feet higher than the 1-percent annual chance event water level. If the hydrology of the Mouse River watershed changes in the future from any of a number of potential factors, the design height of Project features, specifically areas around bridges, ends of levees and floodwalls, and constrictions, may need to be revisited to maintain compliance with FEMA requirements.

4.0 Interior Drainage Analysis

The interior drainage systems tributary to the Tierrecita Vallejo gatewells and associated piping through the levee consist of overland flow conveyances (ditches and streets), culverts, and detention ponds. The interior drainage is conveyed through the levee by gatewells and ultimately discharges to the Mouse River. Stormwater typically discharges by gravity without being affected by normal river flows. However, during periods of high river flows, the flow conveyed to the river will be affected by water surface elevations in the river—making it important to understand how the interior drainage system will function during periods of high water levels. An inadequate interior drainage system may contribute to flooding during large storm events. Therefore, flooding that may occur from the drainage impeded by the river must be analyzed. As a minimum requirement of 44 CFR §65.10(b) (6) (reference [26]), the interior drainage analysis must be based on a coincident analysis of exterior (river) and interior stages that includes the capacity of gravity and blocked gravity conveyance features.

An interior drainage analysis was performed to verify the adequacy of the conveyance features, detention volume, gatewells and pump station capacity, and to determine expected flood levels for areas landside of the line of protection. A coincident frequency analysis was completed following the methodology described in USACE EM 1110-2-1413 (*Hydrologic Analysis of Interior Areas Engineering and Design*, reference [27]). The interior drainage assessment was completed to verify the 1-percent annual chance coincident peak inundation levels behind the levee meet the minimum requirements of 44 CFR §65.10 and the Minot *Storm Water Design Standards Manual* (reference [28]).

The pre-project drainage area upstream of the Tierrecita Vallejo levee alignment was studied as part of the MI-2 and MI-3 interior drainage analysis. The current conditions interior drainage analysis of Tierrecita Vallejo is affected by elements of other construction projects including MI-2, MI-3, MI-2C (West Peterson Coulee Outlet), and the reconstruction of the U.S. Highway 83 Bypass. The following section provides a summary of the interior drainage methodology and proposed modifications to the interior drainage system. Additional detailed discussion is in Appendix D.

All elevations are presented in NAVD88 unless otherwise noted. The conversion from NGVD29 to NAVD88 for the Project area is NGVD29 + 1.24 feet = NAVD88.

4.1 Tierrecita Vallejo Watershed

4.1.1 Pre-Project Conditions

As part of the MI-2 and MI-3 project design process, the watershed area contributing runoff to the Tierrecita Vallejo levee was examined. The study showed that runoff from approximately 792 acres drains to the Tierrecita Vallejo cut-off meander before discharging into the Mouse River. The watershed is primarily located west of the U.S. Highway 83 Bypass and extends north as far as Twenty Seventh Avenue North. The historic drainage area extents have been modified by the construction of the NW Regional Retention facility, the U.S. Highway 83 Bypass, and the Gravel Products gravel mining pit. The existing watershed area is primarily agricultural and low density residential (2-acre lots) north of Fourth Avenue NW. The northern portion of the watershed drains south along the U.S. Highway 83 Bypass right-of–way, ultimately discharging to the historic Mouse River channel west of U.S. Highway 83 adjacent to the Tierrecita Vallejo subdivision. The watershed is within Minot's extraterritorial jurisdiction and, according to the Minot Comprehensive Plan, areas that are currently undeveloped are planned for development. These areas include low density residential, commercial, and office business park land uses. Although much of the area north of Fourth Avenue NW remains agricultural, the area is rapidly developing.

The NW Regional Retention facility is a major watershed feature that was constructed by the Ward County Water Resource District northwest of the intersection of Fourth Avenue NW (Ward County Road 15) and Harmony Street NW. This 22-acre facility is designed to detain 45-acre feet of stormwater during the 1-percent annual chance storm event (reference [29]).

Within the MI-2 and MI-3 Basis of Design Report (reference [18]), the following recommendations were made with regard to the Tierrecita Vallejo Watershed area:

- Construct an outlet for the NW Regional Retention facility basin that will discharge to the west along the County Highway 15 to the Mouse River.
- Increase the culvert capacity at Harmony Street NW upstream of the NW Regional Retention facility.
- Add 6000 gpm pump station capacity in the stormwater detention area adjacent to the Tierrecita Vallejo development.

Phase MI-2C (West Peterson Coulee Outlet) adds the recommended NW Regional Retention facility outlet and the increases the Harmony Street NW culvert capacity. Phase MI-2C has been designed, funded and is scheduled to be constructed during the 2020 construction season. The project features are considered fully operational for the purpose of the analysis and design of the Tierrecita Vallejo interior drainage system.

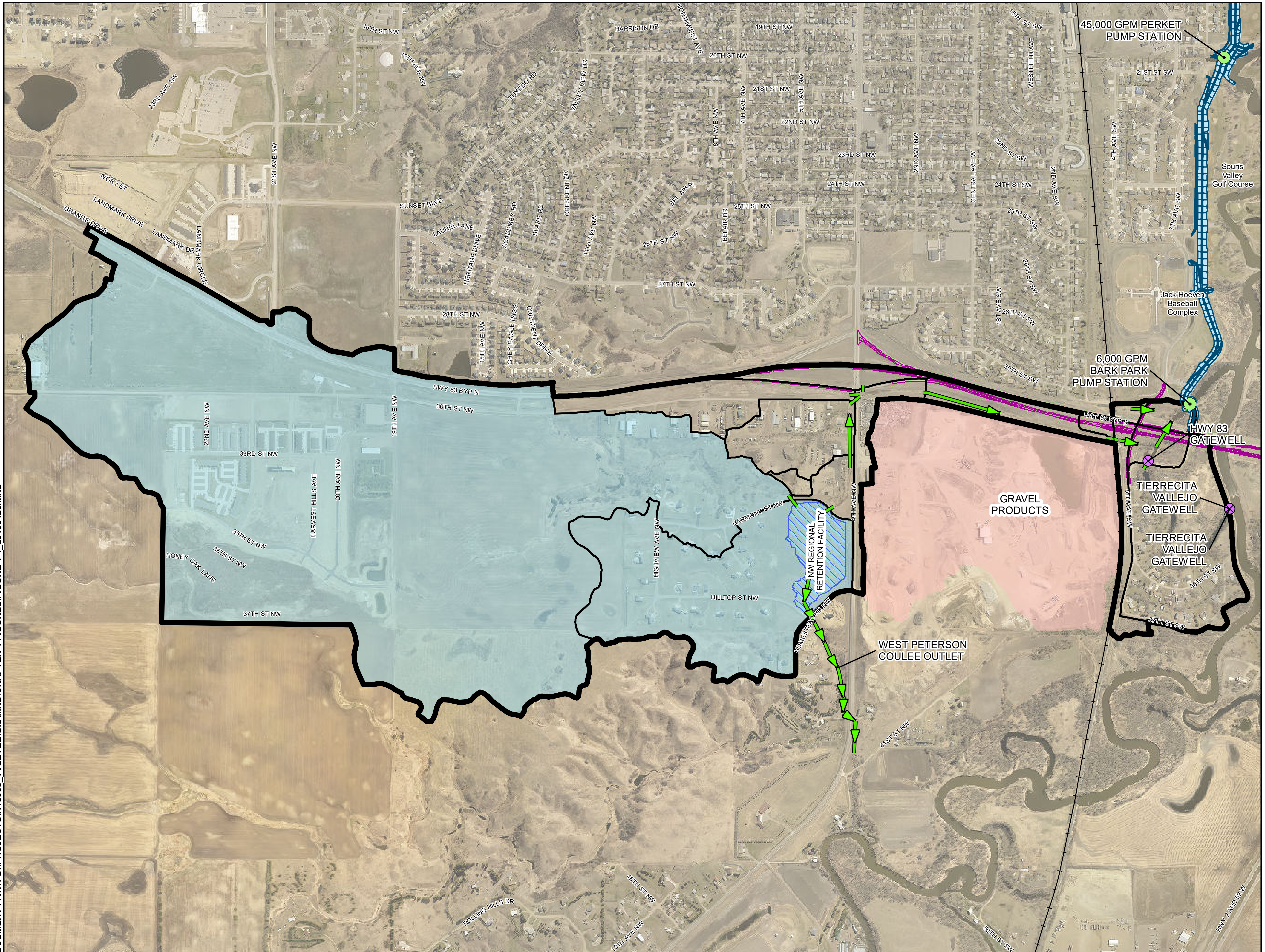
The Bark Park pump station is currently under construction as part of MI-2 and MI-3 levee construction effort. This 6,000-gpm pump station will act to remove water impounded in the cutoff meander. For the purpose of the analysis and design of the Tierrecita Vallejo Interior drainage system, it is assumed that this pump station will be in place and operational.

The reconstruction of the U.S. Highway 83 Bypass is construction activity adjacent to Tierrecita Vallejo that affects the interior drainage analysis. Southbound lanes and embankment located to the west of the existing roadway were added as part of this reconstruction. In addition, the existing U.S. Highway 83/Cam Real/Park access intersection was abandoned and a new intersection providing access to Fifth Avenue SW and Park North was constructed including embankments and drainage features. The effect of the U.S. Highway 83 bypass reconstruction was to reduce the available stormwater storage volume adjacent to Tierrecita Vallejo.

An additional feature adjacent to the Tierrecita Vallejo subdivision and watershed area is the Gravel Products facility. This is a sand and gravel mining facility with a large water retention area necessary for the mining processes. Water is pumped from the Mouse River to this retention area. Currently, the Gravel Products property retains all its stormwater and does not discharge off-site. For the purpose of the Tierrecita Vallejo interior drainage analysis, it is assumed that the Gravel Products property will retain all stormwater runoff generated on the site.

This assumption is also valid in the future even though this area is currently zoned low density residential. In accordance with the Minot Storm Water Design Standards Manual (reference [42]), a development of a property must not increase the peak discharge from a site, when comparing pre-project to post-project conditions. Because the site is currently landlocked and does not discharge stormwater, future development must also meet that standard. Alternatively, part of future development, the developer of the property could infrastructure needed as not to increase flooding of the downstream (Tierrecita Vallejo) properties. This could be done by completing a coincident peak analysis in accordance with of 44 CFR §65.10 and submitting a Letter of Map Revision to revise the Base Flood Elevations the stormwater ponded areas.

Figure 4-1 shows the Tierrecita Vallejo watershed and the location of these project features.



- Pre-Project Watershed
- Pre-Project Subwatershed
- West Peterson Subwatersheds
- Pre-Project Culverts
- Storm Water Detention
- Pump Station
- Gatewell Structure
- US 83 Bypass Reconstruction
- Levee

0 0.1 0.2 0.4
Miles

BARR

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701.837.8737 · www.ackerman-estvold.com
Minot, ND | Williston, ND

Figure 4-1

PRE-PROJECT CONDITIONS
INTERIOR DRAINAGE
TIERRECITA VALLEJO
WATERSHED

Basis of Design Report
Mouse River Enhanced Flood
Protection Project -
Phase WC-1
Minot, North Dakota

DOCUMENT PATH: G:\PROJECTS\R18068_TVLEVEE\GIS\MXD\CHAPTER 4 FIGURES\FIGURE 4-1_20190426.MXD

4.2 Design Considerations

Interior drainage design for the levee system was driven by the considerations outlined below:

- Following the procedures presented USACE EM 1110-2-1413 (*Hydrologic Analysis of Interior Areas Engineering and Design*, reference [27]), for determination of coincident frequency of interior and exterior (i.e., river) flood events
- Meeting the minimum requirements of 44 CFR §65.10(b) (6) (reference [26]), the interior drainage analysis which allows for a coincident analysis of exterior (river) and interior stages that accounts for the capacity of gravity and blocked gravity conveyance features
- Accommodating criteria set forth in the Minot *Storm Design Standards Manual* (reference [28]) and on the Minot Standard Details (reference [30]).
- Incorporating results of the river hydraulic analysis (Section 3.0), levee design features (Section 6.0), and geotechnical analysis (Section 2.0)

4.3 Coincidental Frequency Analysis

A coincidental frequency analysis is a probabilistic method that can be used to perform a flood analysis of interior areas next to the levee system. This means that the probabilities of the river being at a given flood stage and a storm event happening over the interior drainage area are combined to determine the probability of that joint condition occurring. The procedure is directly applicable to areas where the occurrence of the interior and exterior (river) flooding are independent, meaning that the event that causes the flooding behind the levee system is different than the event that causes the flooding in the Mouse River. The assumption of independence is valid due to the significant difference between the drainage areas of the Mouse River watershed and the contributing interior drainage area. In general the analysis includes four steps:

1. Develop a discharge-duration function for the exterior area (Mouse River) based on historical gage data. Split the duration curve into several blocks based on hydraulic points of concern and obtain the average stage (elevation) for each block.
2. Simulate a series of hypothetical storm events over the interior drainage area for the average stage (elevation) of each block developed in Step 1. For each block elevation (external flooding condition) develop a stage-frequency curve for each interior locations of interest.
3. Develop a weighted (coincident) probability function using the total probability theorem for each interior location of interest.
4. Repeat steps 2 and 3 for each alternative analyzed.

4.3.1 Stage Duration Relationship

The stage duration relationships required for the coincident peak analysis were determined as part of the MI-2 and MI-3 interior drainage analysis. The initial step in the coincident frequency analysis uses the historic river data to develop a stage – duration function for the Mouse River. USGS gage 05117500 (Souris River above Minot, North Dakota) data were used to develop a flow–duration function. The USGS daily flow data is available from 1903 to the present. Because of reservoir operations in the contributing watershed, the period of record from March 1, 1997 to November 3, 2015 was used to develop the flow-duration curve. Although an additional three years of river flow data is now available, for consistency with the MI-1 and MI-2 analysis, the additional data was not included for the Tierrecita Vallejo (WC-1) interior drainage analysis. In addition, the flow-duration curve does not consider the winter months in this time period (December 1 through March 1), because the river stage is low during the winter months and it is unlikely that precipitation events that would cause flooding will occur during the winter. The flow-duration curve for the USGS Souris River above Minot, North Dakota gage and for this period is shown on Figure 4-2.

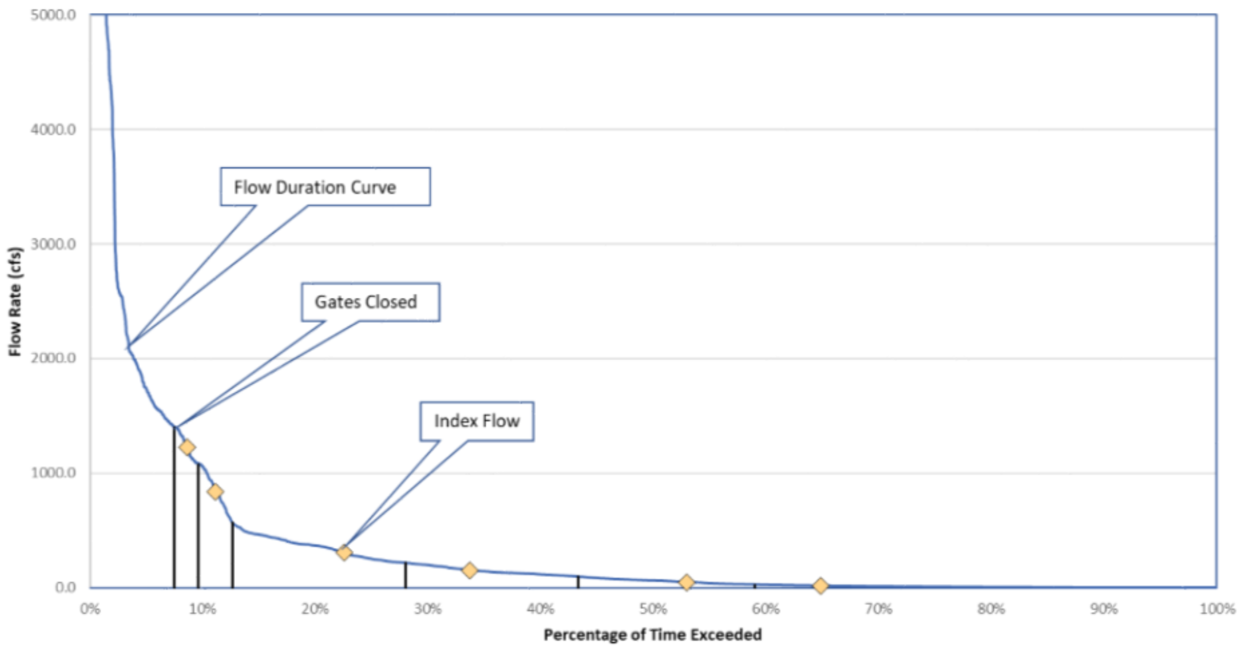


Figure 4-2 Flow-Duration Relationship for the Mouse River at USGS Gage 05117500 (Souris River above Minot, North Dakota) using March 1997 – November 2015 Data

The discharge-duration curve was divided into seven blocks, each characterized by an index flow (discharge of mean probability of the block). The rarest block (0 to 7.5-percent time exceeded) represents the portion of time the gates are closed at the Bark Park Pump Station outfall and the Tierrecita Vallejo Gatewell outfall. The index flow rates that were determined as part of the MI-2 and MI-3 interior drainage analysis as well as two additional index flow rates were used for the Tierrecita Vallejo interior drainage analysis.

The stage of each Mouse River index flow was determined at the Tierrecita Vallejo Gatewell and Bark Park Pump Station outfall locations using stage-discharge information based on the Mouse River HEC-RAS model developed for the Project. This HEC-RAS model is discussed in Section3.0 of this BDR and was developed assuming full MREFPP implementation. The MREFPP HEC-RAS model was truncated to include only the reach from Sixteenth Street to Tierrecita Vallejo and to run as a steady state model so that rating curves could be developed for the Bark Park Pump Station outfall and the Tierrecita Vallejo Gatewell outfall.

The stage-discharge rating curve and the discharge-duration relationship were used to create the stage-duration relationship needed for the coincident peak analysis. Table 4-1 lists this information for the seven index flows at the Bark Park Pump Station Outfall and the Tierrecita Vallejo Gatewell inlet.

Table 4-1 Mouse River Index Flows and Stages

Index	Index Flow (CFS)	Stage at Bark Park Outfall (feet)	Stage at Gatewell 1 Outfall (Feet)	Percentage of Occurrence
1	17	1547.29	1550.18	41.0
2	49	1547.53	1550.36	15.7
3	155	1548.02	1550.77	15.3
4	307	1548.61	1551.20	15.3
5	840	1550.33	1552.31	3.1
6	1230	1551.40	1552.94	2.1
Gates Closed	> 1400	1551.80 at 1400 cfs	1553.2 at 1400 cfs	7.5

Each index elevation and associated probability is used in conjunction with modeling of hypothetical storms over the interior areas to determine the joint probability of an occurrence of various combinations of rainfall / river stages. A XP Solutions Stormwater Management Model (XPSWMM) model was developed to model the hypothetical storms with the index elevations used as discharge boundary conditions. The development of the XPSWMM model is discussed in the next section. The coincident frequency analysis is applied to design conditions with assumption that the MI-2, MI-3, MI-2C (West Peterson Coulee Outlet), Tierrecita Vallejo levees and storm sewer, and U.S. Highway 83 reconstruction are complete. The application of the coincident frequency analysis is discussed in Section 4.5.2.5.

4.4 Interior Drainage Assessment Methodology

The XPSWMM model software uses rainfall and watershed information to generate runoff that is routed simultaneously through pipe and overland flow networks. Simultaneous routing means that flow in the entire system is simultaneously modeled for each time increment, moving from one increment to the next. Other models calculate runoff for the entire duration of the storm by subwatershed, moving from one subwatershed to the next. Simultaneous routing allows the model to account for flow in pipes, flow detained in ponding areas, and effects of operating pumps.

XPSWMM, Version 2014, was used to model the MI-2 and MI-3 interior drainage system. XPSWMM, Version 2018.1, however was used for the Tierrecita Vallejo interior drainage analysis. Data inputs include (1) synthetic rainfall events, (2) subwatershed hydrologic parameters, and (3) hydraulic parameters of the conveyance systems (i.e., storm sewer pipes, culverts, ditches, gatewells and pump station parameters).

Subwatersheds and curve numbers developed as part of the MI-2 and MI-3 interior drainage analysis were used as the basis for the Tierrecita Vallejo analysis. Subwatersheds were refined based on construction that has been completed or planned with regard to the U.S. Highway 83 bypass reconstruction and MI-2 and MI-3.

4.4.1 Rainfall

Atlas 14 precipitation frequency estimates were used for this analysis. Consistent with MI-2 and MI-3, Atlas 14 rainfall depths were applied to the methodology described in the USACE Training Document 15 (TD15) (reference [31]), to develop the synthetic rainfall events for the interior drainage analysis. The methodology incorporates precipitation totals from shorter-duration storm events that have an equal return period to create a synthetic hyetograph of a longer duration event. Twenty-four hour hyetographs for several return periods were developed for the interior drainage analysis. Additional discussion regarding the rainfall depth and distribution is in Appendix D.

4.4.2 Hydrologic Analysis

The Natural Resources Conservation Service (NRCS) curve number methodology described in the Technical Release 55 Manual, *Urban Hydrology for Small Watersheds* (TR-55, reference [32]), was used to simulate the conversion of rainfall to stormwater runoff. The primary hydrologic input parameters are subwatershed area, curve number, and time of concentration; each are described below. The use of NRCS curve number methodology is consistent with the Minot's *Storm Water Management Plan* (reference [33]).

4.4.2.1 Subwatershed Area

Subwatersheds contributing runoff to the study areas were delineated using topographic information derived from the 2014 Project LiDAR, 2016 Minot LiDAR, 2010 Ward County LiDAR and Tierrecita Vallejo project design surface. The topographic data and available aerial photography were used to delineate drainage areas to each culvert or inlet connected to the storm sewer network. Subwatersheds used for the analysis of pre-project conditions are shown on Figure 4-1. Subwatersheds used for the analysis of project conditions are shown Section 4.5 of this report. Additional discussion regarding the methodology used to delineate subwatersheds is in Appendix D.

4.4.2.2 Curve Number

Curve numbers for each subwatershed were determined using the MI-2 and MI-3 project future conditions curve number raster using GIS zonal statistics methodology. Existing and future land-use within the study area was primarily determined using the Minot Comprehensive Plan (reference [34]). The *2011 National Land Cover Database* (NLCD) (reference [35]) was used for areas not included the Minot Comprehensive Plan (reference [34]). Existing land-use data were used for the model validation that was conducted as part of the MI-2 and MI-3 interior drainage analysis. Future land-use data were used to evaluate proposed modifications to the interior drainage system.

Hydrologic soil data were gathered from the Web Soil Survey (WSS) produced by the National Cooperative Soil Survey Geographic (SSURGO) database (reference [36]).

Each land cover and hydrologic soil group combination was assigned a curve number (CN) based on typical values published in the NRCS TR-55 document (reference [32]). Areas of open water were estimated based on the National Wetland Inventory. Open water classifications were compared to the 2014 Ward County aerial imagery and 2015 Google imagery and adjusted, as necessary, to match the aerial imagery.

Curve numbers used for the analysis are summarized in Table 4-2. Additional discussion on the methodology to calculate curve numbers for the analysis is in Appendix D.

Table 4-2 Composite Curve Numbers

Land-Use Classification	Comprehensive Plan Land Use	Impervious	Hydrologic Soil Group			
			A	B	C	D
Low-density residential	Very low-density residential Low-density residential	32%	58	73	82	86
Medium-density residential	Medium-density residential Manufactured home park	36%	60	74	83	87
High-density residential	High-density residential	46%	66	78	85	88
Commercial	Commercial Neighborhood commercial Downtown mixed-use General mixed-use Office business park Hospital	82%	87	91	94	95
Industrial	Industrial	77%	84	89	92	94
Parks and open space	Parks and open spaces Public/Semi-public Golf course Cemetery Rural/Agricultural	10%	45	65	76	82
Right-of-way—urban	Right-of-way	71%	81	87	91	93
Right-of-way—rural	Right-of-way	34%	59	74	82	86
Open water	Open water	100%	100	100	100	100

4.4.2.3 Time of Concentration

Consistent with the MI-2 and MI-3 methodology, the time of concentration for each subwatershed was calculated using the NRCS watershed lag method equation contained in Part 630, Chapter 15 of the NRCS *National Engineering Handbook* (reference [37]). Modifications for urbanization were based on the FHWA Hydraulic Engineering Circular 19: *Hydrology* (HEC-19, reference [38]).

Following the guidance in TR-55 (reference [32]), a minimum time of concentration of 6 minutes was used for each watershed. In watersheds where time of concentration was less than six minutes, it was increased to the 6-minute minimum. Additional discussion on calculating time of concentration for each subwatershed is in Appendix D.

4.4.3 Hydraulic Analysis

4.4.3.1 Stormwater Conveyance

Existing culvert information (length, invert, material) was developed based on survey data, as-built information, and project plans obtained from the SRJB, Minot, NDDOT and the USACE. Field surveys focused on locations in close proximity to the proposed levee, at feature tie-in locations, where as-built information was either not available or there was uncertainty (e.g., conflicting elevations between plans or datum differences). Data included pipe sizes, materials, lengths and invert elevations and inlet configurations and types. Pipe roughness and manhole losses were consistent with the Minot *Storm Water Design Standards Manual* (reference [28]) and FHWA HEC-22 (*Urban Drainage Design Manual*, reference [39]).

4.4.4 Model Validation

The existing conditions model was validated as part of the MI-2 and MI-3 interior drainage effort for June 4, 2014, and June 28, 2014, rainfall events. No additional validation was conducted for the subwatersheds that drain to the Tierrecita Vallejo subdivision.

4.5 Interior Drainage System Proposed Modifications

Proposed modifications to the interior drainage system are required to prevent impacts to existing infrastructure within the Tierrecita Vallejo subdivision. The following sections describe the design considerations and proposed drainage system modifications in the watersheds tributary to the Tierrecita Vallejo subdivision.

4.5.1 Design Considerations

Proposed interior drainage modifications are based on design guidance from the SRJB and Minot to minimize flooding and reduce the need for FEMA flood insurance for commercial or residential structures landside of levee systems. Proposed modifications to stormwater management infrastructure, such as storm sewers, catch basins, and detention ponds, were developed based on criteria set forth in the Minot *Storm Water Design Standards Manual* (reference [28]). The shallow backyard swales associated with drainage along the levees were graded to convey stormwater to storm sewer inlets.

4.5.1.1 Interior Drainage Criteria

According to 44 CFR §65.10 (reference [26]), the interior drainage analysis must be based on coincident analysis of exterior (river) and interior stages accounting for the capacity of both gravity and blocked gravity conveyance features. Additionally, 44 CFR §65.10 states if the average depth of flooding is greater than 1 foot, the extent of flooded areas should be identified and a base flood elevation assigned on the Flood Insurance Rate Maps (FIRMs). To meet the requirements of 44 CFR §65.10, a coincident frequency analysis was conducted for the proposed interior drainage system, including conveyance systems and pump stations. Ideally, these features will reduce interior flooding so that no structures are affected, thereby removing FEMA's flood insurance requirements.

4.5.1.2 Stormwater Management Infrastructure Criteria

Stormwater management infrastructure such as culverts, storm sewers, catch basins, and detention basins necessary to convey stormwater to interior drainage facilities are designed based on criteria set forth in the Minot *Storm Water Design Standards Manual* (reference [28]) and on Minot Standard Details. Design storm criteria associated with stormwater infrastructure is listed in Table 4-3.

Inlets placed in grass areas will be Neenah R-2560-E5 or equal. Proposed storm sewer inlet capacities were modeled consistent with FHWA HEC-22 (reference [39]) and manufacturers' information (reference [40]).

Table 4-3 Stormwater Management Infrastructure Design Storms

Structure	Criteria	Design Storm
Storm sewer	Residential Pipes not surcharged Velocity 3 fps to 20 fps	2-year
Storm sewer	Commercial Pipes not surcharged Velocity 3 fps-20 fps	5-year
Culvert	Headwater below crown	10-year
	Headwater below 1 foot above crown	50-year
	Headwater below 1.5 diameters	100-year
Ditch	Within banks	50-year
	Within easement	100-year
Detention Basin	Regulate at Critical duration	10-year
	1-foot freeboard to lowest finished floor	100-year

Note: Based on Section 3.2.1, 4.8 and 8.2.5 of the Minot Storm Water Design Standards Manual (reference [28])

4.5.2 Tierrecita Vallejo Watershed Drainage System Improvements

Drainage system improvements that have been or will be conducted in the Tierrecita Vallejo watershed independent of the Tierrecita Vallejo levee construction are shown on Figure 4-2 and include:

- NW Regional Retention facility outlet (MI-2C)

- Harmony Street NW culverts (MI-2C)
- U.S. Highway 83 reconstruction
- Bark Park pump station (MI-2)

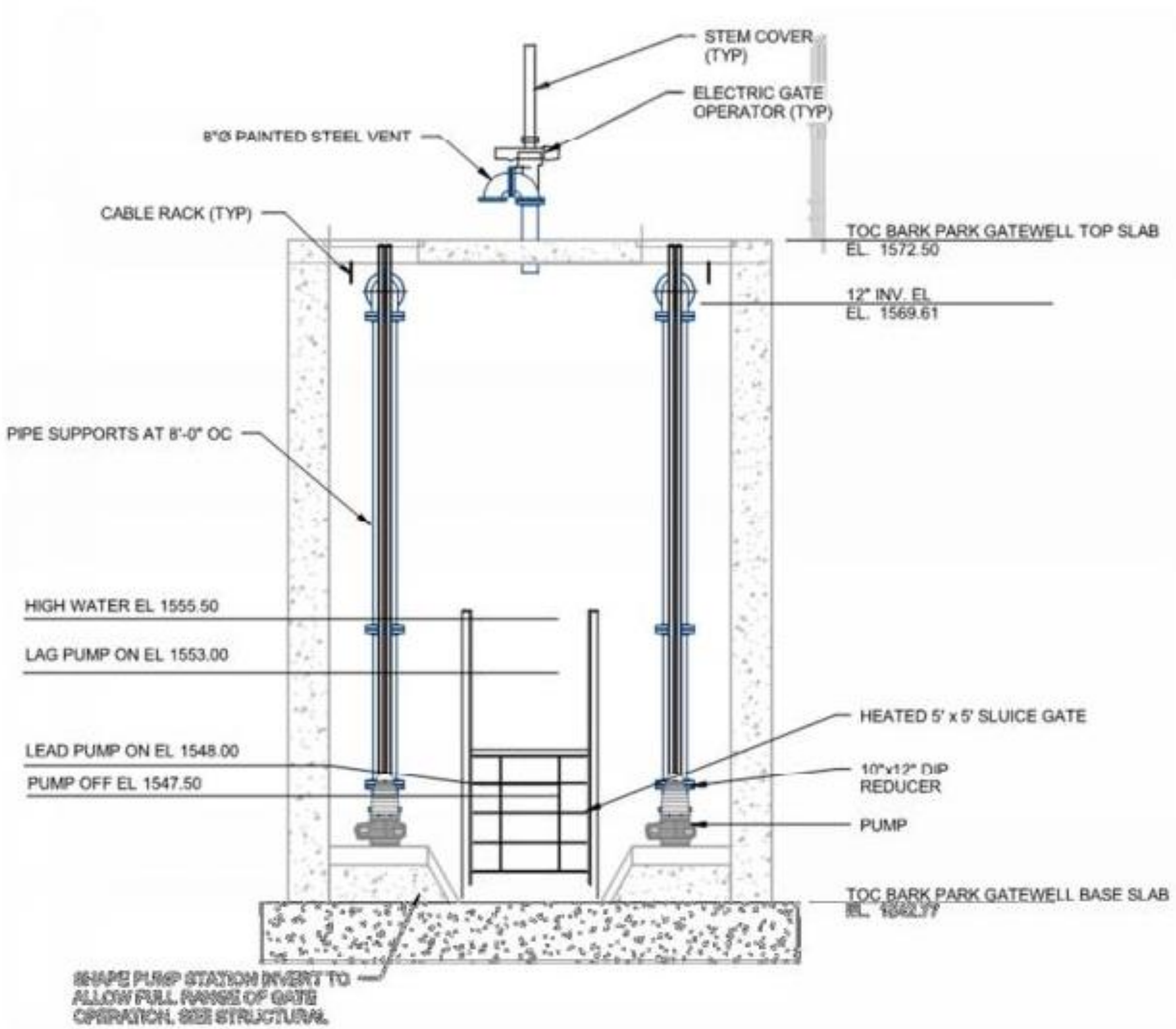
The sizing of the Bark Park Pump Station and planning of the West Peterson Coulee Outlet (MI-2C) were the result of the preliminary alternatives evaluation that was conducted as part of the MI-2 and MI-3 Basis of Design report (reference [18]). The capacity at the Bark Park Pump Station was based on the minimum project pump station size. Due to stormwater storage volume within the conveyance system, the discharge rate for the pump station was not considered in the preliminary alternatives analysis when evaluating the 1-percent annual chance flood elevation for the Tierrecita Vallejo interior drainage storage areas.

A more rigorous evaluation of the interior drainage is required, however, because some volume available for stormwater storage volume that was originally considered during development of the PER and the MI-2 and MI-3 Basis of Design Report evaluations has been filled. The Bark Park Pump Station operation parameters that were developed as part of the MI-2 and MI-3 Basis of Design and are listed in Table 4-4.

Table 4-4 Bark Park Pump Station Operation Information

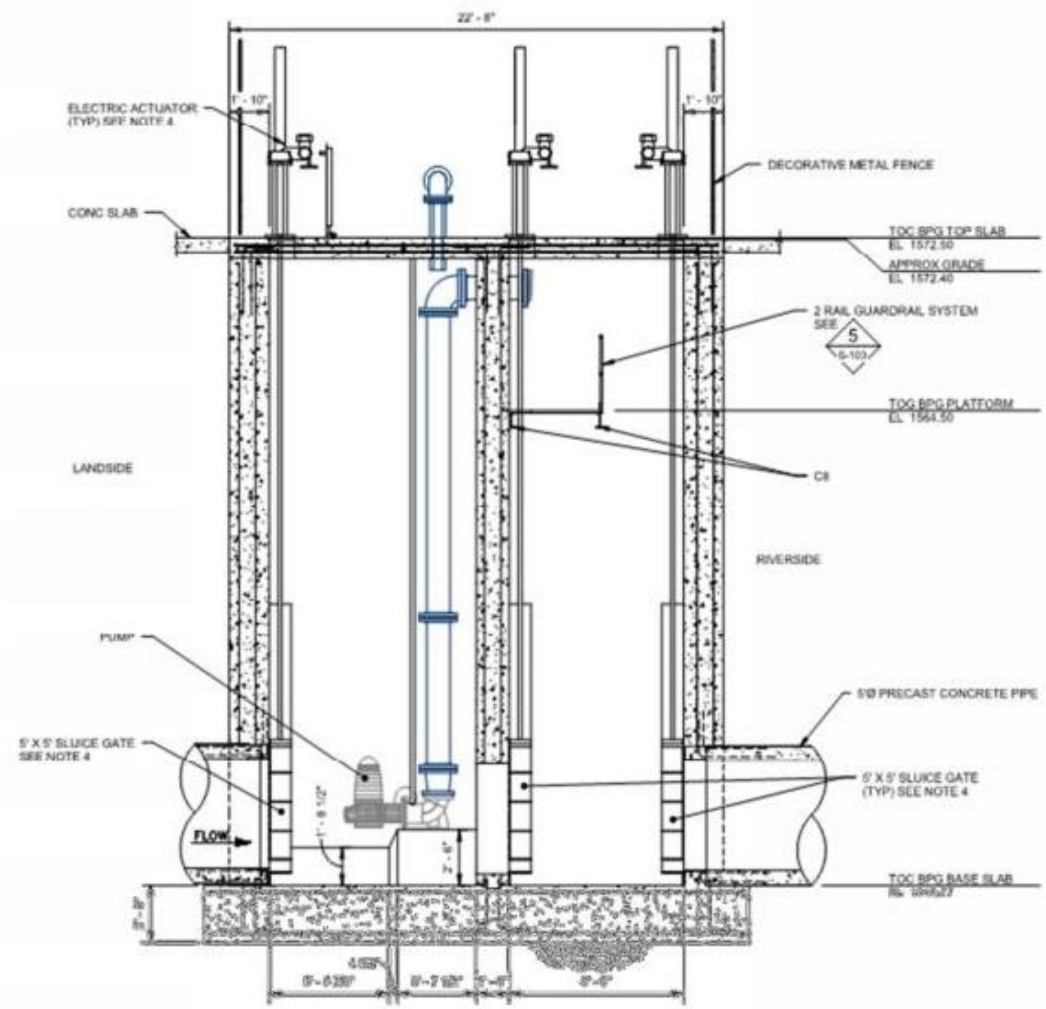
Bark Park Pump Station	
Pump Configuration	6,000 gpm (Duplex)
Gates Closed Elev.	1553.2
Lead Pump On Elev.	1548.0
Lag Pump On Elev.	1553.0
Initial Damage Elev.	1557.7
Pumps Off Elev.	1547.5

Cross section drawings for the Bark Park Pump Station based on the MI-2 and MI-3 construction drawings are shown on Figure 4-3 and Figure 4-4.



Source: From Construction Drawing P-407

Figure 4-3 Bark Park Gatewell / Pump Station



Source: From Construction Drawing S-446

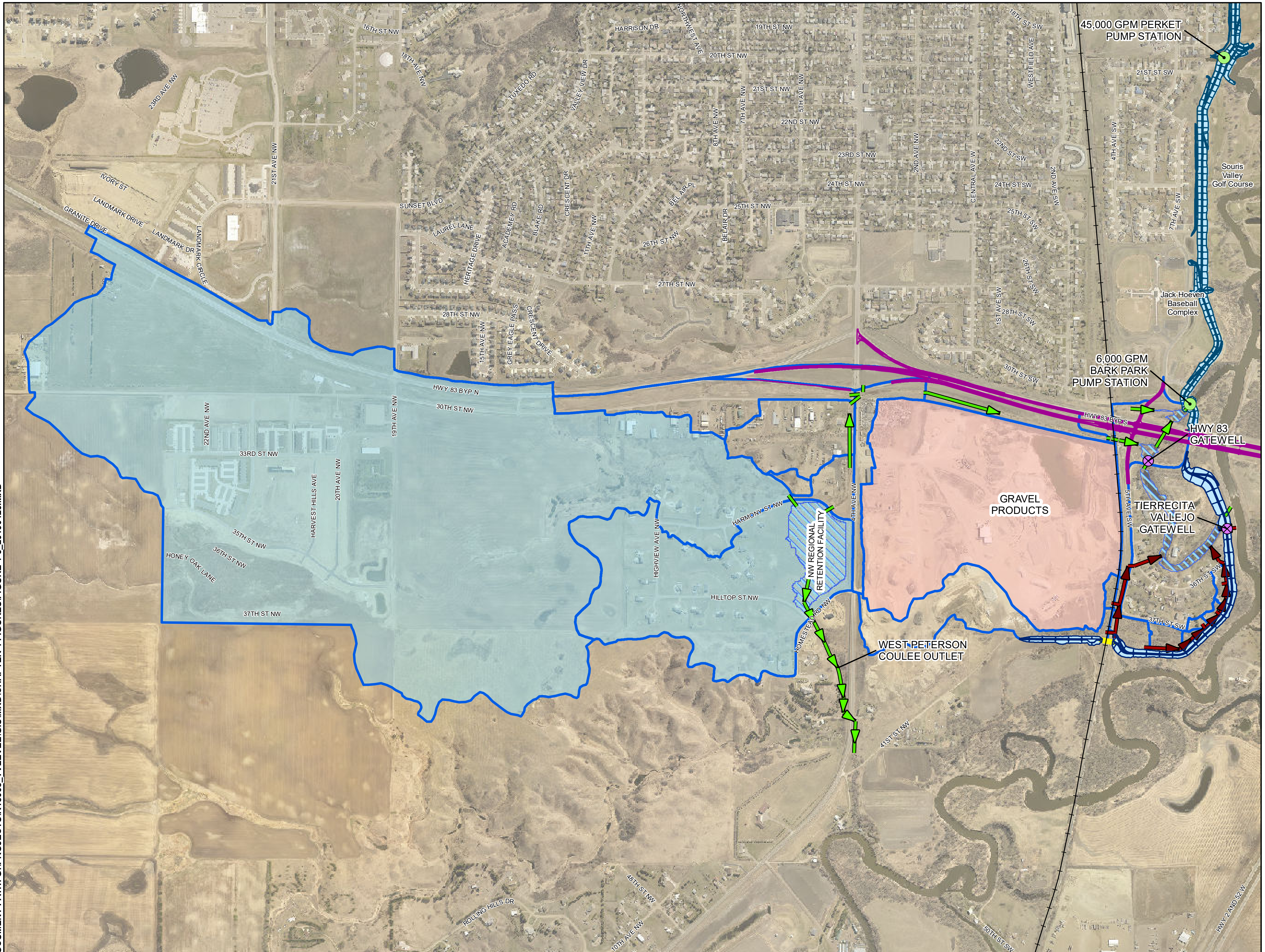
Figure 4-4 Bark Park Gatewell / Pump Station

4.5.2.1 Summary of Tierrecita Vallejo Drainage Improvements

Phase WC-1 drainage improvements are:

- Construct 18-inch and 30-inch storm sewer trunk lines and catchbasins landside of the levee intended to capture surface water and convey this away from the landside toe of the levee
- Grade landside of the Tierrecita Vallejo levee to convey stormwater away from the levee toe and to storm drain inlets
- Reconstruction of Tierrecita Vellejo Gatewell from the Mouse River to the Tierrecita Vallejo cutoff meander
- Modify U.S. Highway 83 Gatewell to include an overflow weir at Elevation 1550

The Tierrecita Vallejo watershed and drainage improvements are shown in Figure 4-5 and the gatewell modifications are shown on Figure 4-6.



Project Subwatersheds

Gravel Products Subwatershed

West Peterson Subwatersheds

Railroad Closure

Pre-Project Culverts

Project Storm Sewer

Storm Water Detention

Pump Station

Gatewell Structure

US 83 Bypass Reconstruction

Levee

0 0.1 0.2 0.4

Miles

BARR

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Figure 4-5

PROJECT CONDITIONS
INTERIOR DRAINAGE
TIERRECITA VALLEJO
WATERSHED

Basis of Design Report
Mouse River Enhanced Flood
Protection Project -
Phase WC-1
Minot, North Dakota

DOCUMENT PATH: G:\PROJECTS\R18068_TVLEVEE\GIS\MXD\CHAPTER 4 FIGURES\FIGURE 4-5_20190426.MXD

Place holder

Figure 4-6 Tierrecita Vallejo Gatewell and U.S. Highway 83 Gatewell Modifications

Awaiting design.

4.5.2.2 Storm Sewer and Grading

Drainage swales, storm sewer and inlets will be constructed landside of the levee to drain surface water away from the toe of the levee. In addition, storm sewer will be added to capture stormwater north of the CP Railroad that is trapped by the levee embankment. Stormwater conveyance features will be:

- The Cam Abierto roadway will be sloped away from the levee to convey sheet flow away from the toe of the levee to the road ditch.
- 2,050-LF of 18-inch storm sewer will be constructed along the levee toe in the south western portion of Tierrecita Vallejo. This storm sewer will collect stormwater from low areas adjacent to the levee and in the Cam Abierto road ditches. Five catchbasins will collect stormwater adjacent to the levee and from the road ditch. The slope of the storm sewer will be approximately 0.2-percent. The storm sewer outlet will discharge to the cutoff meander at invert 1550.
- 1,470-LF of 30-inch storm sewer pipe will be constructed along Fifth Avenue SW alignment in the north western portion of Tierrecita Vallejo. The purpose of this storm sewer is to convey stormwater captured in the CP Railroad ditch. The slope of the storm sewer will be approximately 0.5-percent. The storm sewer outlet will discharge to the cutoff meander at invert 1550.

4.5.2.3 Stormwater Detention

The cutoff meander that extends from Tierrecita Vallejo Gatewell to the Bark Park Pump Station will be used for stormwater detention. U.S. Highway 83 Gatewell will be modified with a weir at elevation 1550. The removal of the gate will prevent mis-operation of the facility and insure availability of flood storage in the cutoff meander. As a result, the portion of the cut off meander within the Tierrecita Vallejo subdivision will not be drawn down further than 1550, providing levee stability and aesthetic benefits for the residents. The portion of the cut off meander between U.S. Highway 83 Gatewell and the Bark Park Pump Station will be controlled by the Mouse River stage when the gates are open. During sunny day conditions with the gates open, a portion of the Mouse River discharge, will be diverted through the cut off meander.

During an internal runoff event with the gates open, stormwater will drain through the cutoff meander and flow to the Mouse River by gravity. During an internal runoff event with the gates closed, stormwater will be stored within the cutoff meander and be slowly expelled by the Bark Park Pump Station. Approximately 6.4 acre-feet of stormwater will be pumped to the Mouse River when the Bark Park Pump Station is activated in preparation of runoff events. At 6,000 gpm, this will take approximately 6 hours. During a runoff event, stormwater will back-up through U.S. Highway 83 Gatewell and be stored in the portion of the cutoff meander within the Tierrecita Vallejo subdivision. Once water levels in the meander have been pumped down, approximately 37 acre-feet of stormwater storage is available before the initial damage elevation of 1557.7 is reached.

4.5.2.4 Storm Sewer Evaluation

The proposed storm sewer performance was evaluated for conformance with the Minot *Storm Water Design Standards Manual.* The storm sewer was evaluated to ensure the trunk line is not surcharged during the design storm (5-year storm) and will obtain self-cleaning velocities (3 fps) during the design event. In addition, the catch basin grates were evaluated to ensure that excessive ponding does not occur above the storm sewer grate during the design event. In addition, the storm sewer was evaluated to check that during the 1% AEP event, additional risk to property will not result from the installation of this storm sewer.

The XPSWMM model that was developed as part of the MI-2 ad MI-3 project was modified to incorporate the changes to the watershed previously discussed and the proposed storm sewer. Subwatersheds were delineated to each storm sewer inlet. Ponding volumes at the storm sewer grates were developed based on the project terrain and proposed grading. Storm sewers were evaluated assuming low river flow allowing for free outflow from the storm sewer pipes into the cutoff meander. Table 4-5 lists the ponding depth at storm sewer inlets. Table 4-6 lists the depth/diameter ratio and velocity that will occur in the during the 5-year event.

Table 4-5 Ponding Depth at Storm Sewer Inlets

Structure	Inlet Type	Design Storm Inlet flow (cfs)	Design Storm Inlet Ponding Depth (feet)	100-year Storm Inlet flow (cfs)	100-year ponding depth (feet)	Comments
STMHCB 50	R-2560-E5	0.2	< 0.1	1.3	3.4	
FES (65)	FES	18.3	1.4	36.0	6.4	In north CP Railroad Ditch
STMHCB 75	R-2560-E5	0.1	0.6	7.2	0.7	
STMH 80	R-2560-E5	0.0	0.5	0.0	1.3	In south CP Railroad Ditch
STMHCB 110	R-2560-E5	5.2	0.1	5.9	2.6	
STMHCB 140	R-2560-E5	0.1	< 0.1	1.5	0.7	
STCB 155	R-2560-E5	0.1	< 0.1	1.2	0.1	
STCB 165	R-2560-E5	0.1	< 0.1	1.7	0.1	
FES (205)	FES	0.1	< 0.1	3.1	1.1	

Table 4-6 Compliance with Storm Sewer Criteria (20%-annual chance design event)

Upstream Structure	Downstream Structure	Diameter (inch)	Depth / diameter	Velocity (fps)	Comments
STMHCB 50	STMH 60	15	0.353	1.9	
FES (65)	STMH 60	30	0.577	10.5	
STMH 60	STMH 70	30	0.637	5.8	
STMHCH 75	STMH 70	18	0.342	2.5	
STMH 70	STMH 80	30	0.657	5.5	
STMH 80	STMH 90	30	0.702	5.3	
STMH 90	STMH 100	30	0.702	5.0	
STMH 100	FES	30	0.673	5.3	
STMHCB 110	STMH 130	18	0.955	3.0	
STMH 130	STMH 140	18	0.874	3.1	
STMH 140	STMH 150	18	0.869	3.1	
STCB 155	STMH 150	18	0.803	0.8	Inlet pipe for small area
STMH 150	STMH 160	18	0.873	3.0	
STCB 165	STMH 160	18	0.806	1.0	Inlet pipe for small area
STMH 160	STMH 180	18	0.878	3.0	
STMH 180	STMH 200	18	0.905	2.9	
FES (205)	STMH 200	18	0.055	2.8	
STMH 200	STMH 210	18	0.905	2.8	
STMH 210	STMH 220	18	0.875	2.8	
STMH 220	FES	18	0.800	3.1	

4.5.2.5 Tierrecita Vallejo Inundation Area

The preliminary alternatives evaluation was conducted as part of the MI-2 and MI-3 Basis of Design (reference [18]) that considered flood elevation landside of the Tierrecita Levee and the Bark Park Pump Station. These alternatives assumed the required improvements will be based on a 1-percent annual chance event occurring concurrently with the river stage high enough so that the floodgates are closed. These alternatives, however, also assumed the large volume of stormwater storage proposed in the Preliminary Engineering Report would be available.

As part of this study, the interior drainage features and ponding landside of the levee was evaluated based on the minimum requirements of 44 CFR §65.10 using coincident frequency analysis techniques. This evaluation assumes the Bark Park Pump Station will remove stored river water and seepage waters to the operating depth before a landside rainfall event occurs and will evacuate stored runoff over time, but is not considered to be effective during a rainfall event. In addition, it is assumed that stormwater storage will be available on NDSWC property, Minot Parks property, and within the Tierrecita Vallejo cutoff meander only and these properties will not be subject to additional fill or encroachment.

The coincident analysis was conducted to determine the extent of interior flooding if the Tierrecita Vallejo levee and storm sewer, MI-2C (West Peterson storm sewer), U.S. Highway 83 reconstruction, and MI-2 and MI-3 (Phases 2 and 3) are all constructed as planned. In addition, it is assumed that the Gravel Products property is landlocked and non-contributing. This analysis did the following:

- Modified existing XPSWMM model to remove ponding volumes at the storm sewer inlets. These roadway ditch and shallow backyard ponding areas were removed for the coincident peak analysis because there is no guarantee that private property owners will not fill these areas in the future.

- Ran the modified XPSWMM model for the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year rainfall events for the gates closed scenario and each of the six "index stage" tailwater values shown in Table 4-1.
- Developed elevation-weighted probability relationship for each location of interest based on the probability of each "index stage" occurring, and the probability of a rainfall event occurring. The 1-percent annual chance coincident probability flood elevation at each point of interest can be interpolated from its elevation-weighted probability relationship.

The coincident frequency analysis was used to determine the 1-percent annual chance flood elevations (Base Flood Elevation) for the Tierrecita Vallejo detention areas. The extent of the 1-percent annual chance inundation was delineated based on flood elevations determined using coincident analysis methods and are shown on Figure 4-7.

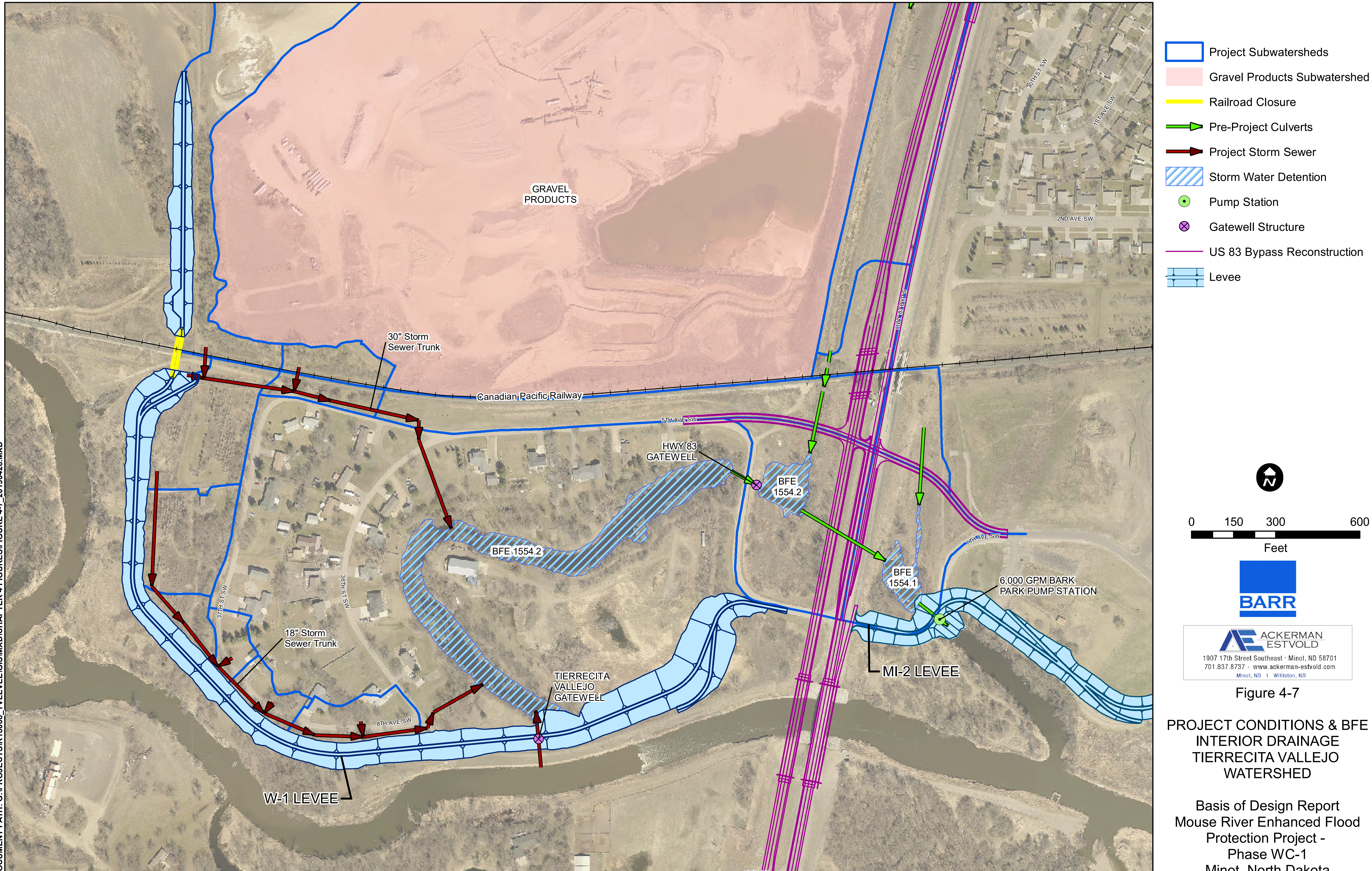

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Figure 4-7

PROJECT CONDITIONS & BFE INTERIOR DRAINAGE TIERRECITA VALLEJO WATERSHED

Basis of Design Report
Mouse River Enhanced Flood Protection Project - Phase WC-1
Minot, North Dakota

DOCUMENT PATH: G:\PROJECTS\R18068_TVLEVEE\GIS\MXD\CHAPTER 4 FIGURES\FIGURE 4-7_20190426.MXD

5.0 Environmental Evaluations

The alteration and modification of the existing levee system requires approval by the USACE. Section 14 of the Rivers and Harbors Act of 1899 (33 United States Code [USC] 408, hereinafter referred to as "Section 408") authorizes the Secretary of the Army to permit alterations and modifications to existing USACE projects in certain circumstances. The Secretary of the Army has delegated this approval authority to the Chief of Engineers of the USACE. The types of alterations and modifications under Section 408 that require approval by the Chief of Engineers include degradations, raisings, and realignments of levee systems. Nonfederal proposals to alter or modify an existing USACE project such as the MREFPP must be evaluated as new construction of federal projects. The potential impacts of these changes, including system impacts, must be evaluated in accordance with USACE regulations and policy, including the regulatory requirements of the National Environmental Policy Act (NEPA).

Environmental surveys and inspections were conducted in Phase WC-1 to collect data for environmental review and permitting, document existing conditions at the site, and assist in design and engineering. These surveys and inspections include wetland delineations, ordinary high water mark (OHWM) determination, biological studies, cultural resources investigations, and a review of potential hazardous, toxic, and radioactive waste (HTRW) sites in or near Phase WC-1. A pre-demolition inspection of any remaining structures to be removed from the project footprint will be completed prior to demolition. These surveys and inspections are briefly described in the following sections.

5.1 Environmental Review

An environmental review of the proposed Project will be conducted to comply with NEPA regulations (33 CFR Part 230). A programmatic EIS was approved by Record of Decision (ROD) for the MREFPP in December 2017 and covers general impacts associated with construction of the full MREFPP from Burlington through Minot. This programmatic EIS was prepared in accordance with the guidelines specified in the Section 408 Submittal Package Guide as part of CECW-PB Memorandum titled *Clarification Guidance on the Policy and Procedural Guidance for the Approval of Modifications and Alterations of Corps of Engineers Projects* (reference [41]) The EIS evaluated resources listed in Section 122 of the Rivers and Harbors Act of 1970 and analyzes Project alternatives and the direct, indirect, and cumulative impacts on natural and socioeconomic resources. The draft EIS was released for public comment on November 4, 2016, and the comment period concluded on December 22, 2016. The final EIS was released for public review on July 14, 2017, with the comment period concluding on August 14, 2017. The final EIS was completed in July 2017, with the ROD issued on December 19, 2017.

The USACE determined that project-specific impacts associated with future phases of project development shall be addressed under NEPA documents tiered to the EIS. The environmental review for this Project will consist of an environmental assessment that has an analysis of impacts unique to Phase WC-1 of the proposed Project.

5.2 Wetland Delineations

Potential wetland areas within the construction limits of Phase WC-1 were identified based on field wetland delineations completed in September 2018 in accordance with the procedures specified in the *Corps of Engineers Wetland Delineation Manual* (reference [42]) and the *Regional Supplement to the Corps of Engineers Wetland Delineation Manual: Great Plains Region (Version 2.0)* (reference [43]). Wetland boundaries were determined by completing USACE Wetland Determination Data forms for paired sample points and by observing vegetation and hydrology in the study areas. The sample points and wetland boundaries were documented using site photography and GIS positioning in conjunction with GPS point locations taken with a Trimble Geo 7x instrument. A summary of the wetland delineation results is provided in Appendix O-1.

5.3 Ordinary High Water Mark Determination

Other waters within the Project area include the Mouse River and its associated fluvial features (e.g., oxbows). As part of state and federal regulations, the OHWM is used to determine the jurisdictional boundaries of these waterbodies. The OHWM of the Mouse River was determined at several transects throughout designated Project segments in accordance with the State Water Engineer OHWM guidance document (reference [44]). Identifying the OHWM along the river channel consists of determining the elevation at which the vegetation changes from a predominantly wetland community to an upland community as well as identifying the presence of high water indicators such as drift lines and water marks (stains) on the banks, rocks, or concrete headwalls. Several transects were selected in each Project segment as well as areas planned for overbank excavations. The locations of transects, photographs, and the OHWM data points were georeferenced with a Trimble GPS unit. A summary of the OHWM determination results are provided as part of the wetland delineation report in Appendix O-1.

5.4 Wetland and Ordinary High Water Mark Impacts

Based on the wetland delineation completed in September 2018, an estimate of wetland impacts within the construction limits was determined. As shown in Table 5-1, it is estimated that 0.51 acres of wetlands will be permanently impacted and 0.09 acres temporarily impacted by construction of Phase WC-1; temporary wetland impacts are not anticipated. Wetland impact areas are shown on Figure 5-1.

Table 5-1 Wetland Impact Estimates

Wetland Impact Area	Permanent Impact Area (acres)	Temporary Impact Area (acres)
Wetland #1	0.06	0.00
Wetland #2	0.03	0.00
Wetland #3	0.03	0.00
Wetland #4	0.03	0.00
Wetland #5	0.00	0.02
Wetland #6	0.35	0.07
Wetland #7	.01	0.00
Total	**0.51**	**0.09**

Construction activities will be conducted below the OHWM of the Mouse River at several locations throughout Phase WC-1. These river impact areas are summarized in Table 5-2. For Phase WC-1, a total of approximately 0.61 acres of the Mouse River will be permanently affected below the OHWM; temporary impacts below the OHWM are not anticipated. Impacts are primarily associated with installation of erosion control measures. OHWM impact areas are shown on Figure 5-1.

Table 5-2 OHWM Impact Estimates

River Impact Area	Permanent Impact Area (Acres)	Temporary Impact Area (Acres)
OHWM #1	0.07	0.00
OHWM #2	0.46	0.00
OHWM #3	0.01	0.00
OHWM #4	0.01	0.00
OHWM #5	0.05	0.00
OHWM #6	0.01	0.00
Total	**0.61**	**0.00**

Additional areas that may be impacted by the project but were not in the September 2018 wetland delineation will evaluated in spring 2019 as site conditions become suitable. The findings will be in the 90% BDR.

Construction Limits
Ordinary High Water Mark
Wetland Boundaries
Approximate Permanent Impacts Below OHWM
Temporary Wetland Impacts
Permanent Wetland Impacts

0 150 300 600
Feet

BARR

ACKERMAN ESTVOLD

Figure 5-1

WETLAND AND OHWM IMPACTS

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

The estimated wetland and OHWM impacts stated in Table 5-1 and Table 5-2 and shown in Figure 5-1 have been evaluated related to avoidance and minimization. Following is a summary documenting the justification for each wetland and OHWM impact area.

- Wetland Impacts #1 and #2 – The proposed levee alignment crosses wetlands at two locations. Levee slopes have been graded to 3H:1V to minimize impacts and the levee alignment adjusted to the extent practicable to also minimize impacts.
- Wetland Impacts #3 and #4 – Impacts will occur due to the placement of riprap as erosion control for the storm sewer outlets at these locations.
- Wetland Impact #5 – Impacts will occur in this location due to removal of accumulated debris at the storm sewer inlet.
- Wetland Impact #6 – This area will be impacted due to construction of the levee alignment. Levee slopes have been graded to 3H:1V to minimize impacts.
- Wetland Impact #7 – This area will be impacted due to installation of the gatewell flared end.
- OHWM Impacts #1, #2, #3, #4, and #5 – Impacts will occur along the Mouse River channel due to overbank excavation necessary to provide increased flood conveyance for the design flood event.
- OHWM Impact #6 – This area will be impacted due to installation of the gatewell flared end.

As stated in the EIS, wetland and OHWM impacts are likely to be offset wither through on-site, in-kind permittee-responsible mitigation or by purchasing mitigation credits from an approved mitigation bank or in-lieu fee program. Specifics of the mitigation plan will be coordinated as part of Phase WC-1 permitting.

5.5 Biological Inventory

A biological evaluation was conducted for Phase WC-1 on October 24, 2018. This evaluation included a visual inspection for raptor nests (primarily bald eagles) in the Project area, as well as confirmation of areas of moderate tree cover or forest. No raptor nests were observed in the Project area. A summary of the raptor survey results are provided in Appendix O-2.

5.6 Cultural Resources Investigation

The cultural resources evaluation for the majority of Phase WC-1 was completed as part of the evaluation for Construction Stage 1.5 and documented in the programmatic EIS. As noted in the programmatic EIS, the results of the Class I Cultural Resources inventory indicated that there are no known archaeological sites or historic structures located within the impact areas of Phase WC-1. The portion of Phase WC-1 that extends north of the Canadian Pacific Railroad was not evaluated for cultural resources previously. A cultural resources investigation of this extended levee area is pending, once ground conditions become clear, and findings will be in the 100% BDR.

5.7 Hazardous, Toxic, and Radioactive Waste Assessment

An HTRW assessment was conducted in May 2019 in general conformance with ER 1156-2-132 (*HTRW Guidance for Civil Works Projects*, reference [45]). The purpose of the HTRW assessment was to identify issues and problems associated with waste in Phase WC-1 of the Project. The HTRW focused on areas surrounding the new levee alignment. The assessment included a review of regulatory reports, historic aerial photographs, fire insurance maps, reverse city directories, and topographic maps; interviews with city staff; and a field inspection of the Project area to identify land-use practices and potential sources of contamination (Appendix O-3).

The following environmental risks were identified as having the potential to affect Phase WC-1 of the Project:

- Hazardous building materials may have been used during construction of buildings and should be abated prior to demolition of any remaining buildings.
- Based on the age of the residences in the Project area, storage tanks may be encountered during demolition and will need to be removed and properly disposed.
- Hazardous materials may be present within a metal and pallet debris pile on private property observed south of the Mouse River oxbow. Although no barrels, signs of stressed vegetation, or staining were observed, removal of the debris should be managed appropriately.

Based on the lack of regulatory sites within and adjacent to the proposed levee alignment and the lack of drums, storage tanks, or other potential sources of hazardous materials or petroleum products in or near Phase WC-1, it was determined that no further HTRW investigations were needed. However, it is recommended that a contingency plan for unanticipated releases be in place during construction to specify procedures for management and disposal of hazardous materials that may inadvertently be encountered (Appendix G of Appendix O-3).

5.8 Pre-Demolition Inspection

Inspections of any homes or structures remaining in the construction areas will be performed prior to demolition activities. Inspections will involve documenting asbestos and hazardous materials. Regulated waste within buildings will be documented in accordance with North Dakota requirements (NDDH Title 33 Article 20 – Solid Waste; NDDH Title 33 Article 24 – Hazardous Waste). A report will be prepared to document hazardous materials identified during on-site inspections and specify procedures for proper management and disposal of the materials.

6.0 Civil Design

6.1 Civil Design Features

Civil design generally focused on Phase WC-1 elements related to alignment and definition of feature geometry, vertical profiles, utility design, and corridor requirements. USACE standards and guidelines were used for the design development. Specific elements include the following:

- Erosion control
- Demolition and corridor preparation
- Horizontal and vertical levee alignments
- Utility penetrations and alignments within the proposed USACE right-of-way
- Levee ramps for access, service, or crossings
- Alignment of floodwalls and railroad closure structure
- Drainage control, including seepage collection, interceptor ditches, culverts, and gatewells
- Slope erosion protection for levees, structures, and river bank areas
- Overbank excavation for increased channel capacity
- Borrow site, earthwork balance, and disposal options
- Municipal infrastructure modifications including watermain, storm sewer, and street
- Traffic control during construction
- Franchise utilities including electric, gas, cable, telephone and other private services
- Correction of USACE inspection items from the most recent routine inspection
- Site restoration and landscaping

6.2 Design Considerations

Civil design for the Phase WC-1 levee system was driven by location, elevation, and alignment considerations outlined below:

- Project Design Guidelines (Appendix N)
- Reviewing and implementing the PER (reference [2]) dated February 29, 2012
- Incorporating river hydraulic analysis and interior drainage features

- Considering geotechnical subsurface investigation and modeling results
- Limiting, to the extent possible, property acquisitions beyond those proposed in the PER
- Integrating deficiencies identified in the USACE 2017 Routine Inspection Report (reference [46]) for Tierrecita Vallejo Levee Systems (left bank) along Phase WC-1 corridor
- Minimizing environmental, social, and economic impacts of the project

6.3 Temporary Erosion Control

Erosion control measures will need to be installed by the contractor prior to the start of construction activities. As part of the construction documents, a Stormwater Pollution Prevention Plan (SWPPP) will be developed to comply with North Dakota Pollutant Discharge Elimination System (NDPDES) Permit requirements. In accordance with local, state, and federal requirements, the SWPPP will outline the design, implementation, management, and maintenance of best management practices (BMPs) to reduce the amount of sediment and other pollutants in stormwater discharges associated with land-disturbing activities. The SWPPP is in the 60% design submittal.

Temporary BMPs used to control erosion and sedimentation during construction may include one or more of the following:

- Rock construction entrance
- Sediment pond
- Silt fence
- Erosion-control blanket
- Inlet siltation protection
- Concrete washout
- Floating silt curtain
- Rock filter dike
- Temporary and Permanent Vegetation
- Dewatering controls

BMPs shown in the construction drawings are intended to serve as a baseline. During the course of the work, the contractor will inspect and monitor the BMPs to ensure they are functioning correctly and providing adequate functionality for construction phasing and scheduling. The contractor will customize the BMPs as work proceeds and, if needed, furnish and install additional BMPs to accomplish the requirements of the SWPPP. Modifications to recommended BMPs should be documented in the SWPPP by the contractor and included on any required contractor submittals and/or work plans.

6.4 Demolition and Corridor Preparation

Demolition and removal of existing structures, streets, and municipal utilities are required where the levee realignment passes through existing developed residential areas, Canadian Pacific right-of-way, and public right-of-way. The following tasks are necessary to prepare the corridor for construction of the new levee.

6.4.1 Exploration Trench

Prior to levee construction, a minimum 6-foot-deep exploration trench or minimum 10-foot-deep trench within demolished structure footprints will be excavated to verify that the corridor is clear of unknown utility penetrations or unsuitable subgrade materials. This excavation will be done in accordance with Section 7-2 of EM 1110-2-1913 (reference [15]) with the location at the levee centerline alignment. Exploration trench Type 1 through 2 geometry is shown in Figure 6-1 with locations defined by station in the construction drawings in Appendix K.

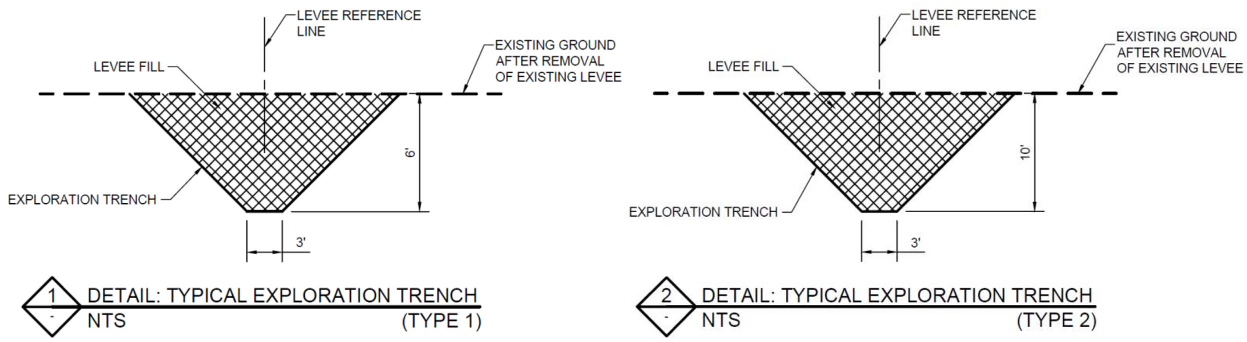


Figure 6-1 Exploration Trench Geometry Type 1 through 2

Exploration trenches shall extend 3 feet below bottom of building or structure foundations discovered during exploration trenching operations. Exploration trenching may require water management and dewatering. Backfill will only be placed after careful inspection of the excavated trench to ensure that seepage channels, utilities, or undesirable materials are not present. Exploration trench geometry details and notes are shown on the construction drawings in Appendix K.

The exploration trench to verify that the corridor is clear of utility penetrations or undesirable materials for the railroad closure structure will be sequenced with the structure construction. Excavation for the trench will extend approximately 3 feet below the bottom of the closure and floodwall footings.

6.4.2 Structure Demolition

Houses, sheds, and garages on properties where buyouts have been accepted have been removed by Ward County; demolition included removal of foundations and known subsurface utilities. Where needed, removal of individual sanitary sewer and water service to the mainline where applicable will be completed as part of the MREFPP.

For properties and structures acquired as part of Phase WC-1, demolition procedures are in the construction documents. Prior to demolition activities, pre-demolition inspections of existing structures will be completed to identify hazardous materials. Structures will be completely removed, including basements, foundations, drain tiles, and bedding aggregate(s). Appurtenant items will also be removed, including sidewalks and private utilities (water, sanitary sewer, and all other underground utilities). The excavations will be backfilled with low permeability material and compacted in lifts in accordance with requirements set forth by the construction documents. Excavations may require water management and dewatering.

6.4.3 Vegetation Removal

The minimum width of the vegetation-free zone shall be the width of the levee, floodwall, or embankment dam, including all critical appurtenant structures, plus 15 feet on each side, measured from the outer edge of the outermost critical structure, per guidance in Section 2-2 of engineer technical letter (ETL) 1110-2-583, *Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures* (reference [47]). In accordance with Section 5-3 of ETL 1110-2-583 (reference [47]), noncompliant vegetation will be removed to reduce risks to the levee's integrity.

Areas where trees and shrubs infringe on the vegetation-free zone of the current levee system were identified during previous USACE routine and periodic inspections. This unwanted vegetation will be removed during construction, as shown in Figure 6-2. This includes removing trees and shrubs above grade and removing roots in the required root-free zone below ground. Roots greater than ½ inch in diameter must be removed in the vegetation-free zone. Areas where vegetation must be removed are shown on the demolition drawings. Excavations will be backfilled in individually compacted lifts, in accordance with the levee embankment construction specifications and backfill material will be taken from the identified borrow location.

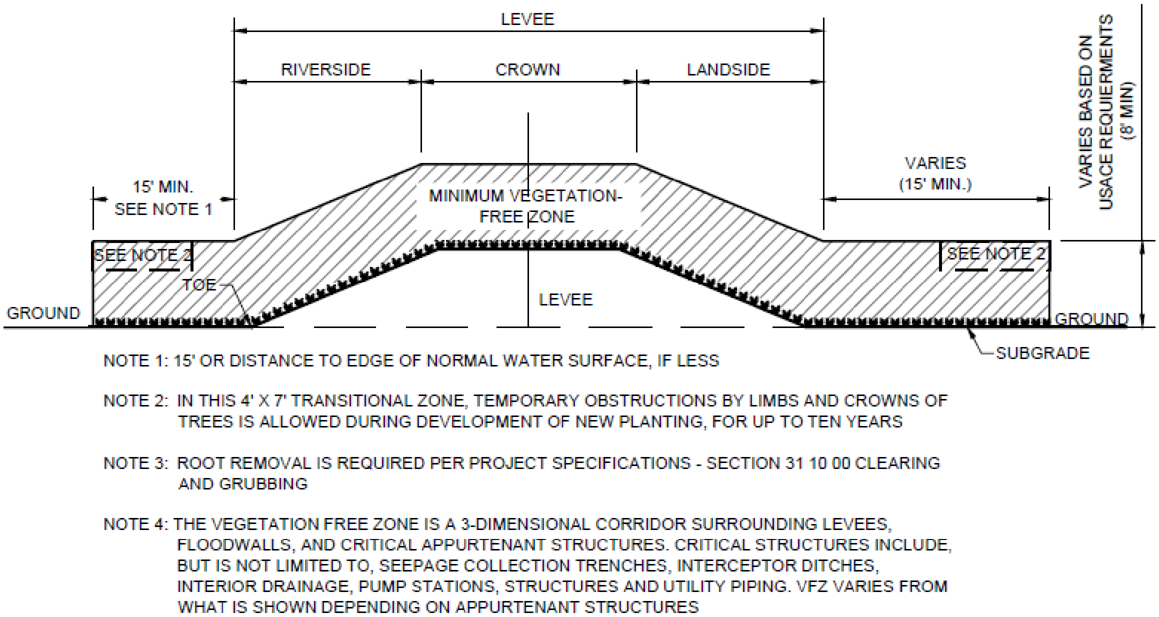


Figure 6-2 Vegetation-Free Zone

Generally, vegetation-free zone requirements will be followed.

6.4.4 Existing Utility Levee Penetrations

Where the new levee alignment will be constructed, an exploration trench of 6-foot depth typical and 10-foot depth within existing roadways will be excavated to verify that the corridor is clear of unknown utility penetrations (Section 7-2 of EM 1110-2-1913, reference [15]). With exception of the Northwest Area Water Supply (NAWS) watermain, locations where the proposed levee corridor crosses existing utilities, complete removal of pipe, conduit, structures, and granular foundation material will be performed. The excavations will be filled with low permeability fill and compacted in lifts in accordance with the construction specifications. These excavations will require water management and dewatering as needed.

At the existing NAWS water line crossing (Station 55+60F), the existing water line is within a steel casing pipe and will remain. The inspection and modifications are discussed in Section 6.6 – Levee Penetrations.

6.4.5 Existing Levee Removal

The design includes full of the existing levee during Phase WC-1. Removal is deemed necessary due to the anticipated extent of root penetration, levee soils that may not meet levee fill material requirements and impacts due to exploration trenches. The construction drawings define the extent of existing levee removal in profile and cross sections and the proposed levee base elevation coincides with the removal extent. Depending on existing soil properties, the embankment material removed may be used as borrow material for new levees.

To the greatest extent possible, levee removal will be sequenced to maintain the existing level of flood risk management to Tierrecita Vallejo Development during construction. The Contractor shall sequence demolition of the levee and exploration trench operations so as to minimize areas with no flood risk management features to a total of 1,000 feet at any given time during project construction. The Contractor will need to be able to reconstruct the removed portion within 24 hours if flooding is forecasted.

6.4.6 Wetland and Unsuitable Soils Excavation

Construction of the proposed levee alignment is anticipated to require excavation of wetland soils from 18+30F to 21+25F and 45+50F to 47+70F. Wetland soils consisting of organic material, peat, or topsoil will be removed entirely from areas within the levee footprint. This material will be replaced with acceptable, compacted levee fill.

Unsuitable soils will be corrected in the same manner outlined in Section 7-2 of EM 1110-2-1913 (reference [15]) and defined in the drawings and technical specifications. A detail defining over-excavation is shown in the construction drawings.

6.4.7 Street and Utility Demolition

Streets and utilities identified for removal are in the demolition plans. Roadway pavements and base aggregates will be removed down to subsoils of low permeability. Public and private utilities will be removed.

6.5 Levee Alignment and Section

6.5.1 Levee Horizontal Alignment

A permanent levee for Phase WC-1 will be constructed along horizontal alignments denoted as Levee Alignment F. The alignment is defined by stationing with values increasing from upstream to downstream. The horizontal alignment locations represent the centerline of the levee crest. The minimum radius for the levee centerline alignment is 100 feet.

Levee Alignment A is on the north (left) side of the river and extends from high ground approximately from 1,000-ft north of the Canadian Pacific Railroad (the upstream boundary) to the U.S. Highway 83 Bypass (the downstream boundary).

The following sections identify elements affecting the horizontal alignment of the flood risk management features for Phase WC-1.

6.5.1.1 Real Estate Acquisitions

Minimizing the need for additional property acquisition was a significant consideration in determining levee horizontal alignments. Property acquisitions were primarily defined by alignments developed as part of the PER and modified based on existing field conditions. In addition to the properties Ward County acquired, the SRJB will continue to acquire properties needed to construct Phase WC-1.

6.5.1.2 U.S. Highway 83 Bypass

The horizontal alignment for Phase WC-1 connects to the existing embankment at the U.S. Highway 83 Bypass, perpendicular to the highway alignment, north of the existing southbound bridge crossing the Mouse River. The location where the levee connects to the existing roadway embankment follows the old Fifth Street alignment and is in line with the upstream end of the MI-Phase 2 levee alignment. As shown on Construction Drawing C-314 in Appendix K, the existing embankment for the old Fifth Street intersection will be removed and the proposed levee will tie directly to the existing west U.S. Highway 83 Bypass roadway embankment. Tie-in to the highway embankment will follow the procedure described in Appendix E-1. Embankment material will be confirmed by acquiring NDDOT embankment construction documentation for construction completed for the southbound embankment in 2018.

6.5.1.3 Geotechnical Considerations

Geotechnical field investigations were completed by Barr, including soil borings, CPT testing, and soil sampling. The subsurface investigation data is in Appendix B. In determining the overall flood risk management alignment, the Barr team identified critical cross-sections within the Phase WC-1 corridor to analyze slope stability and seepage. Following is a summary of geotechnical requirements which have an impact on the levee alignment.

- Setback Analysis—Geotechnical analysis generally requires a setback of 25 feet from the edge of the riverbank to the levee toe for slope stability on the riverside of the levee. For the landside, a 30-foot zone from the levee toe has been established for construction of seepage, interior drainage-control measures, and vegetation free zone.
- Overbank Excavation Side Slope—Overbank excavation of the existing river channel is shown to improve river flow conveyance. Geotechnical review of the overbank excavation slopes identified a localized bank stability concern where the slope intersects an existing subsurface sand layer. The levee alignment was adjusted to provide a 3.5H:1V levee side slope where overbank excavation occurs (Figure 6-3).

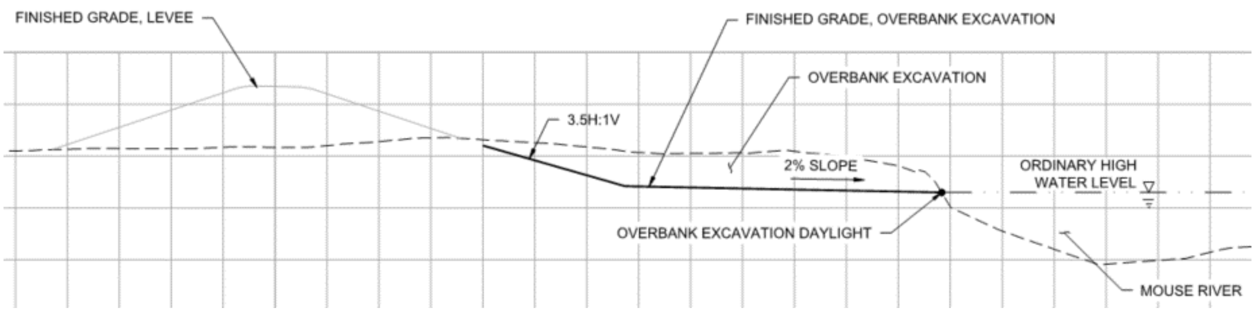


Figure 6-3 Overbank Excavation Side Slope

6.5.2 Levee Vertical Alignment

The vertical alignment/elevation for the top of the levee (defined at the levee reference line) was determined from hydraulic modeling based on the 2011 design event, as discussed in Section River Hydrology and Hydraulic Analysis . The design provides additional levee height above the design flood elevation to account for risk and uncertainty, superiority overbuild and settlement overbuild. The top-of-levee grade along the stationed alignment has a defined percent slope for staking and constructability and the top-of-levee cross slope is shown draining toward the river at a 2-percent grade. For reference, see typical levee sections as shown in Figure 6-4.

To account for potential levee settlement, settlement overbuild is in the overall levee construction height. Overbuild for the entire levee alignment is 12-inches. Additional discussion regarding settlement is in Section 2.8.

6.5.3 Levee Cross Section

The new levees (including tie-back levees) are designed with a 10-foot top width and 2-foot 6H:1V shoulders. The top-of-levee cross slope for the levee is shown as 2-percent grade from landside to riverside to maintain positive drainage of the levee crest.

Proposed levee side slopes are shown as 3H:1V due to maintenance requirements. As stated in EM 1110-2-1913 (reference [15]), a 3H:1V slope is typically the steepest slope that can be conveniently mowed and walked on during inspections.

The entire proposed levee alignment will be finished with 6 inches of gravel surfacing. Gravel surfacing is proposed within the levee superiority overbuild as requested by the Project sponsor. The aggregate surfacing will be 10 feet wide with a 2-foot-wide 6H:1V turf shoulder on the landside and a 2-foot-wide 6H:1V gravel shoulder on the riverside. The gravel shoulder will allow the aggregate section to drain to the riverside. A typical section of levee with gravel surfacing is shown in Figure 6-4.

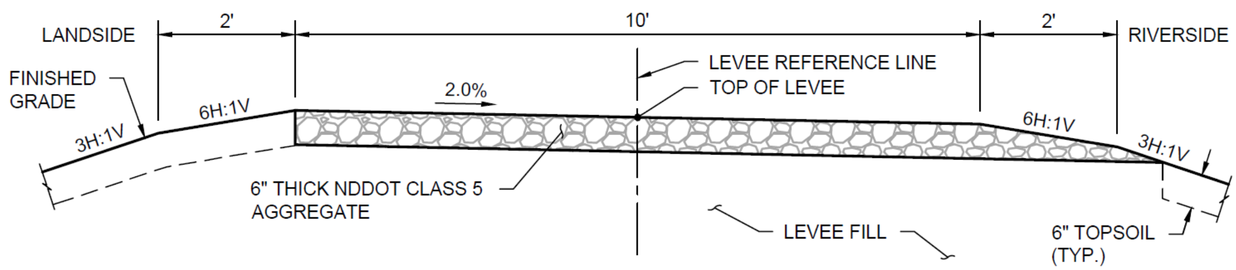


Figure 6-4 Typical Levee Section with Gravel Surface

6.5.4 Levee Construction and Material

The levee will be constructed following guidance outlined in EM 1110-2-1913 (reference [15]). Clearing and grubbing will be done within the levee corridor, including the removal of all vegetation, roots, stumps, etc. Existing topsoil will be stripped from native ground or from the existing levee. The existing levee, along with unsuitable foundation material, will be removed. If the existing levee material meets the specifications for proposed levee material, it will be temporarily stockpiled for reuse. After topsoil stripping and removal of the existing levee material, the levee subgrade will be scarified to prevent surface compaction planes. An exploration trench will be excavated to expose or intercept undesirable underground features.

The levee will be constructed with low permeability material, placed in specified lifts, and compacted according to the technical specifications. Material will be acquired mainly from designated borrow areas. Levee fill material will meet specified gradations and be clearly described in the technical specifications.

Topsoil will be installed on all slopes to the thickness specified in the construction drawings and will meet material requirements outlined in the technical specifications. Topsoil will be reused from stockpiles created during stripping operations. If necessary, additional topsoil will be acquired from a borrow pit identified by the contractor prior to construction. Topsoil thickness will be 6 inches based on typical requirements by Minot.

6.6 Levee Penetrations

The proposed levee will cross a proposed 36-inch storm sewer and the existing 24-inch NAWS line. The proposed utility will be designed and the existing utility will be evaluated for conformance with the guidance outlined in EM 1110-2-1913 (reference [15]) and, *Chapter 13, Special Features* based on the recent Agency Technical Review (ATR) Draft 01 AUG 2014 (reference [48]). In addition, the levee utility crossings will be completed/evaluated in accordance with FEMA Publication Number 484, *Technical Manual: Conduits through Embankment Dams* (reference [49]). Guidance on techniques for the design of levee penetrations was provided in Chapter 8 of EM 1110-2-1913 (reference [15]). The utility penetration locations within the levee footprint are shown in Table 6-1 with approximate stationing.

Table 6-1 Proposed Utility Penetration Locations

Levee Station	Utility Name	Utility Size	Pipe Material	New or Existing
46+20F	Storm Sewer Gravity Main from Gatewell to cutoff meander	48-inch	Concrete	New
55+60F	NAWS Watermain	24-inch	PVC	Existing

Utility penetrations through the levees have been minimized to reduce the risk of seepage, pipe line leakage, or other negative impacts. All penetrations that will be located beneath the proposed levees are designed to meet USACE standards. The existing utility is and the proposed utility crossing will be located within a steel casing and constructed to minimize the risk of pipe leaking or rupture, trench settlement, and other failures. The pipes penetrating the levee will be provided with positive closures. Gravity lines will be provided with service gates and pressurized systems will be equipped with valves on both the riverside and landside of the levee.

The existing NAWS line is a water transmission line with minimal lateral junctions. Existing junction structures will be utilized as dry side closures and a gate valve will be placed on the wet side of the proposed levee. On the north, the existing junction structure is 140 feet north of the proposed levee alignment. To the south, the existing structure is 200 feet south of the eastbound U.S. Highway 2/52 lane and at an elevation above the project level of protection.

At the existing NAWS water line crossing (Station 55+60F), the existing water line is within a steel casing pipe. The existing steel casing pipe will be exposed, and the bedding material inspected. If granular material is discovered, the bedding material will be removed and replaced with Controlled Low Strength Material (CLSM). The existing annular void space within the casing was previously filled with sand. This space will be jetted out and filled with a benitoite slurry mix and the ends of the casings will be capped in concrete. The excavation will be backfilled with levee fill material. The existing grade at the proposed crossing is at the design top of levee location above the existing NAWS line and settlement is expected to be negligible.

6.7 Levee Access Ramps

Levee access ramps for service roads, are in the design. A total of 17 individual levee ramps are to be constructed as part of the project. Figure 6-5 shows an example of an access ramp.

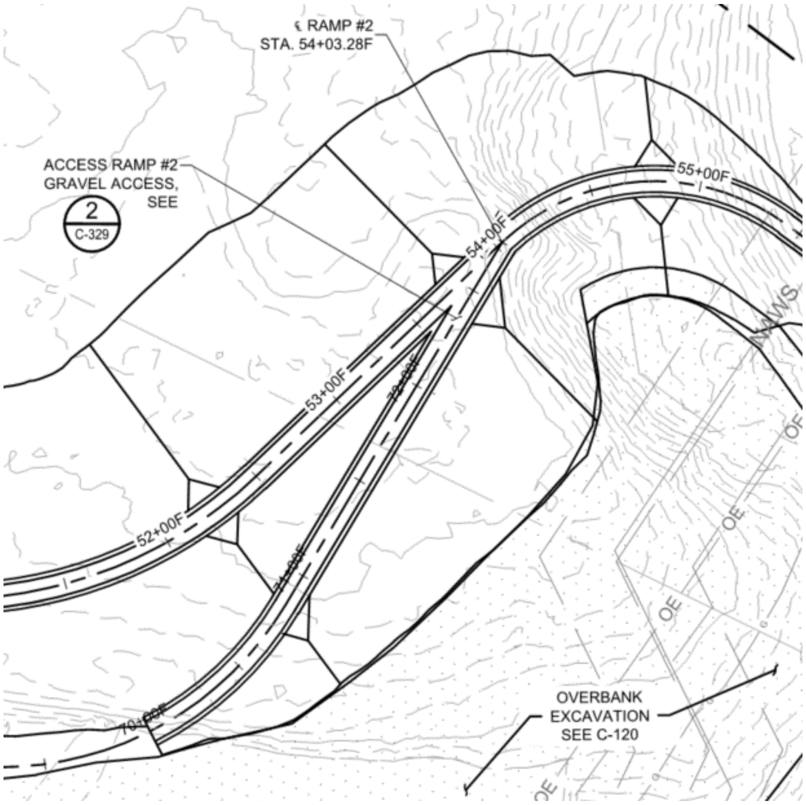


Figure 6-5 Example Access Levee Ramp #2

Ramps were designed according to EM 1110-2-1913 (reference [15]), Paragraph 8–10. Ramps are designated for maintenance access and will have gravel surfacing. Ramp slopes are dependent on the intended use and range from 5 percent to 10 percent.

Side slopes on all ramps will be no steeper than 3H:1V and will be constructed of common fill material because ramps are located outside of the proposed levee prism. Table 6-2 summarizes ramp location, intended use, and specified surfacing material.

Table 6-2 Levee Ramp Summary

Levee Station	Ramp #	Intended Use	Surfacing
28+00F	1	Maintenance	Gravel
54+00F	2	Maintenance	Gravel

6.8 Interceptor Ditches

In general, the proposed levee is on natural high ground with the existing grade along the base of the levee sloping away. However, in some locations interceptor ditches are required along the landside of the levee to intercept and route surface runoff away from the levee to storm sewer systems. Interceptor geometry is based on interior drainage modeling and the capacity required to convey anticipated runoff.

In most cases, a "V" notch ditch with 4H:1V side slopes and a minimum depth of 1 foot (Figure 6-6) is used.

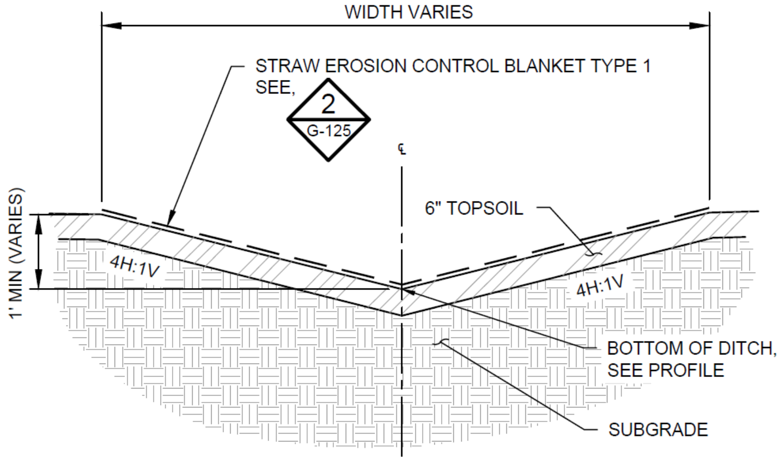


Figure 6-6 Interceptor Ditch Section

Table 6-3 includes a summary of interceptor ditch locations with reference to proposed levee stationing. Construction drawings, provided in Appendix K, show proposed grading where interceptor ditches are required.

Table 6-3 Interceptor Ditch Alignment Summary

Location	Length (ft)	Drawing Reference
Levee Station 10+00F – 19+50F	950	C-361
31+00F – 32+50F	150	C-362

6.9 Storm Sewer and Gatewell Levee Penetrations

Storm sewer proposed for the project is shown as reinforced concrete pipe (RCP). RCP used in levees is required to meet pipe class standards provided in ASTM C76. The pipe classes provided in ASTM C76 correspond to the D-load (load per foot of diameter) required to produce a 0.01-inch crack in the pipe. Pipe loading calculations were computed according to Section 3-7 of EM 1110-2-2902 (reference [50]) which is based on Section 17.4 of American Association of State Highway and Transportation Officials (AASHTO) (reference [51]). The D-load was determined to be 1,170 lbs which corresponds to an ASTM Class III RCP. Pipe loading and pipe classification calculations are provided in Appendix E-6. Other design factors including RCP pipe class, pipe bedding, and pipe foundation requirements were determined according to ASTM C-76 (reference [52]), American Association of State Highway and Transportation Officials (AASHTO) (reference [51]), Chapter 8 of EM 1110-2-1913 (reference [15]) and EM 1110-2-2902 (reference [50]). The size of storm sewer was determined according to hydrologic and hydraulic modeling.

The technical specifications indicate that pipe trenching of 20 feet or greater will be designed by a professional engineer registered in North Dakota and must consider both drained and undrained soil conditions and the influence of levee and roadway embankments near the excavations. Soil parameters used during design are provided in the technical specifications to be referenced by the trenching design engineer.

Pipe bedding and foundation design consists of three typical sections which correspond to requirements provided in EM 1110-2-2902 (reference [50]) and EM 1110-2-1913. First-class pipe laying methods for trench installations will be implemented according to EM 1110-2-2902. Low permeability backfill will be placed around the pipe and will be compacted to 95% Standard Proctor within the extents of the existing levee. EM 1110-1913 (reference [15]) and EM 1110-2-2902 (reference [50]) also requires the landside third of the sewer pipe be surrounded with an 18-inch granular drainage layer to address water and soil piping around the outside of conduits. Lastly, installation of a concrete cradle under the first length of pipe at the upstream and downstream ends of gatewells will be installed in accordance with EM 1110-2-2902 (reference [50]). Concrete cradle and pipe bedding design calculations are provided in Appendix E-6.

6.9.1 Gatewells

Gatewells are currently shown at the following locations.

- Tierrecita Vallejo Gatewell - Located near the upstream of the existing dam structure, the gatewell will pass a 24-inch pipe to convey stormwater to a remnant oxbow at the proposed levee crossing.
- Dam by-pass Gatewell—the existing by-pass gatewell is at the existing dam structure and is utilized to by-pass flow for maintenance purposes. The existing structure will remain in place and does not penetrate the levee.
- U.S. Highway 83 Gatewell - is an existing gatewell located at the east end of the cutoff meander that allows water to be held back within the cutoff meander or released downstream to the Bark Park pump station constructed as a part of MI-2. The existing structure will be modified to replace the existing sluice gate with a stoplog weir structure. The modification will allow for a design water surface to be maintained in the dead loop and allow flow through the cutoff meander with the ability to increase the flow rate in flood conditions.

6.10 Slope Erosion Protection

Slope erosion protection will be required along portions of the Mouse River channel and levee through the project limits to reinforce the slopes and minimize erosion and scour potential from flood flows.

6.10.1 Erosion Protection Design

Design for protection against slope erosion resulting from high velocities, shear stresses, and scour during flood events was completed in accordance with the Project Design Guidelines (Appendix N), EM 1110-2-1601 (reference [53]), and FHWA's HEC-11, *Design of Riprap Revetment* (reference [39]). The main focus of the erosion protection design was to protect the levee system. In general, erosion protection measures were considered between the river and the centerline of the proposed levee. Riprap and TRM are designed as the primary erosion protection measures along the levee where additional erosion protection beyond turf grass is needed. Select locations along the river bank utilized natural stabilization techniques rather than riprap to provide enhanced aquatic habitat.

The erosion protection design included an evaluation of river geometry, soil conditions, and scour potential from the 2011 flood event (27,400 cfs). A general section of the erosion protection design is shown in Figure 6-7 and overall the construction drawings (Appendix K). Additional erosion protection design details and computations are provided in Appendix E-2.

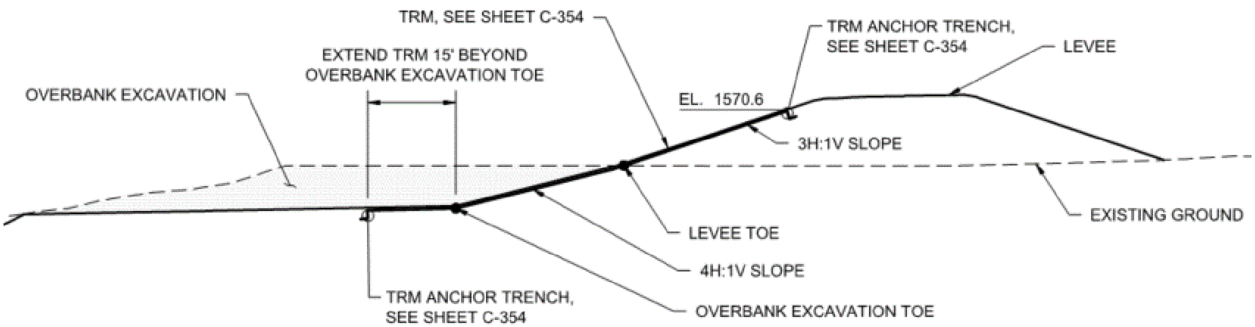


Figure 6-7 Typical Slope Erosion Protection Section

Calculated average river velocities through the construction limits range from less than 3 feet per second (fps) to approximately 7 fps. Higher velocities and greater shear stresses are expected in the river channel and at the U.S. Highway 83 bypass bridge than adjacent to the proposed levee. Therefore, erosion protection design was completed for two locations including the levee and the river bank. Erosion protection designs for both locations are described in Section 6.10.1.1 and Section 6.10.1.2.

6.10.1.1 Levee Erosion Protection

An evaluation comparing both calculated shear stresses and velocities to resistive forces of turf grass was completed to determine locations where erosion protection is needed along the levee. The shear stresses and velocities from the design flood was considered in the evaluation. The evaluation indicated turf grass will provide adequate resistive strength along the majority of the levee alignment thus additional erosion protection is not needed in these areas. Levee regions that do require additional erosion protection were identified based on proximity of the levee to the river, river geometry, hydraulic conditions, and location of critical infrastructure. These critical regions include the levee flanks immediately upstream of the U.S. Highway 83 bypass bridge and the road closure structure. Levee protection is also provided at the oxbow crossing based on geotechnical recommendation. A plan of levee erosion protection is provided in the construction drawings (Appendix K).

Vertical extents of erosion protection were based on the 1% AEP flood water surface elevation for the oxbow crossing and the design flood water surface elevation for the remaining areas. An additional 1.0 to 1.5 feet were added to the water surface elevations listed above to account for hydraulic uncertainty associated with the hydraulic model. HEC-11 guidance and engineering judgement were used to determine the longitudinal extents of erosion protection along the levee. A summary of levee erosion protection is provided in Table 6-4.

Table 6-4 Levee Erosion Protection Summary

Location	Erosion Protection Type	Riprap Thickness	Top of Riprap	Top of TRM
Oxbow Crossing	R20 Riprap	12 inches	1555.5	-
Upstream of U.S. Highway 83 Bypass Bridge	TRM	-	-	1570.6
Railroad Crossing	TRM	-	-	1571.5

- Oxbow Crossing—Erosion protection is needed along the face of the levee adjacent to the oxbow crossing due to the potential failure of oxbow slopes undermining the foundation of the levee from periods of standing water and wave action.
- Upstream of U.S. Highway 83 bypass bridge—The evaluation comparing calculated shear forces to resistive forces of turf grass indicate that a well vegetated levee slope and toe are adequate to protect against erosion from flood flows. However, TRM was deemed appropriate because of relatively higher sideslope velocities at the U.S. Highway 83 bypass bridge and the change of river geometry associated with the bridge in this area. TRM will offer additional erosion protection while vegetation is established. TRM, when fully vegetated, will also provide appealing aesthetics in a very public portion of the project. TRM was selected instead of riprap because the main channel, where the highest velocities will be observed, is set back from the levee at distances ranging from 100 to 500 feet in this location.
- Railroad Crossing—TRM protection is prescribed for both the proposed levee at the railroad crossing floodwall transition. The floodwall-levee transition can create conditions which may lead to erosion and scouring of the floodwall to levee transitions over time.

Riprap erosion protection includes granular bedding above the finished ground of the levee slope and riprap placement above the granular bedding. Typical levee erosion protection sections are shown in the construction drawings (Appendix K). Specified riprap type, bedding material, and bedding thickness are designed in accordance with standard gradations provided in USACE St. Paul District's *Standard Riprap Design* document (reference [54]). Buried riprap consists of 1 foot of vegetated topsoil above riprap and bedding.

TRM erosion protection was designed according to Appendix E-2 and is shown in the construction drawings (Appendix K).

6.10.1.2 Bank Erosion Protection

Bank erosion protection was determined to not be necessary in within the project limits. Channel velocities remain similar to existing conditions and calculations determined that significant scour was not expected. Some higher velocities were potentially noted along the U.S. Highway 83 bypass bridge; however, the NDDOT addressed this area as part of a bridge reconstruction project and is out of the scope of this phase.

6.10.2 Protection at Structures

Protection of the riverside and landside of the levee at critical transitions including at the railroad closure structure is in the erosion protection design. At the railroad closure structure, high performance TRM will extend from the riverside of the levee around the bulb of the levee to the landside of the levee to protect the levee-structure transition.

6.11 Overbank Excavation

Overbank excavations remove material adjacent to the existing river channel to increase the cross-sectional area and capacity of river flow during flood events while minimizing the increase in upstream water surface levels. Overbank excavation will occur upstream of U.S. Highway 83 Bypass on the left side of the river (Figure 6-8).

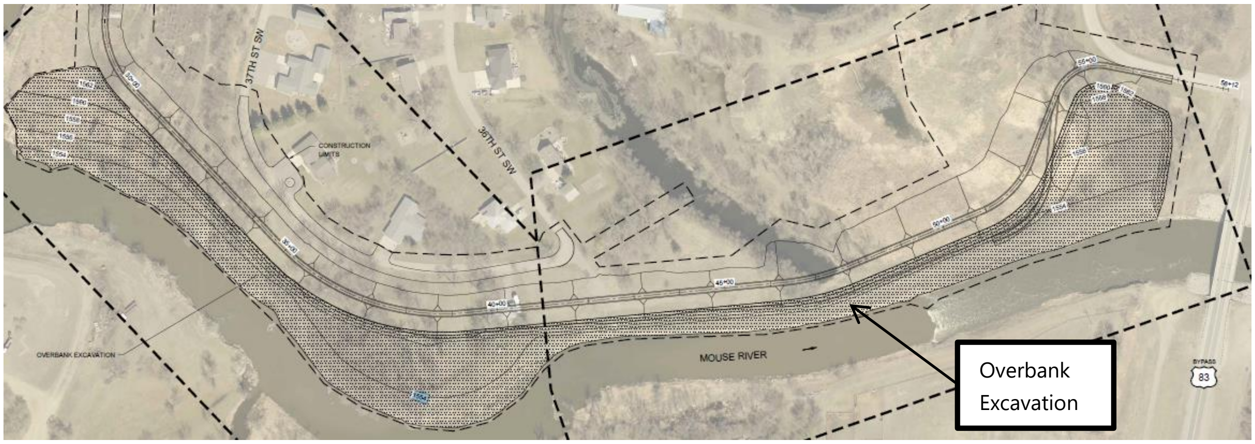


Figure 6-8 Overbank Excavation

The location of the ordinary high-water level (OHWL) was considered when designing the overbank excavation area. To minimize impacts to the existing river and wetlands, the bottom elevation of the excavated section is shown at the water surface elevation equivalent to a 2-year flood event (approximately 1,150 cfs river flow). The bottom-of-overbank excavation elevation varies from 1553.28 feet at the upstream end to 1552.26 feet at the downstream end near the U.S. Highway 83 bridge. The OHWL field determination varies in elevation; therefore, the OWHL has some level of impact from this design. A typical cross-section of the overbank excavation is shown in Figure 6-9. The typical cross-section of the overbank excavation includes a slope toward the river graded at 2-percent grade, a 3.5H:1V transition slope, and 3H:1V levee side slope.

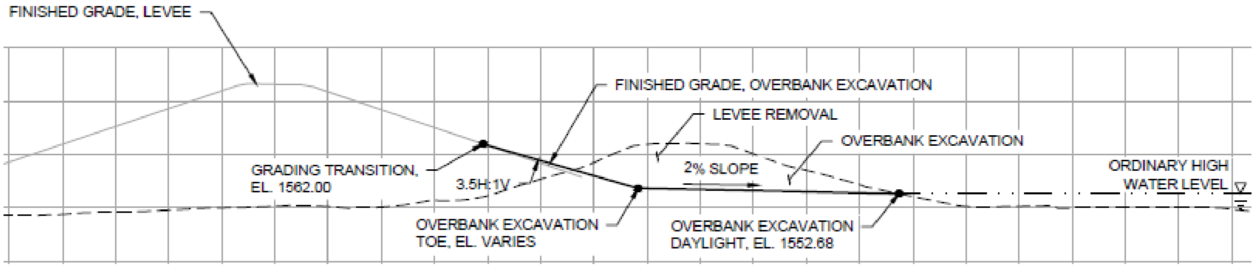


Figure 6-9 Typical Overbank Excavation Section

The 3.5H:1V transition slope was used because geotechnical review of the overbank excavation slopes identified a localized bank stability issue where the slope intersects an existing sand layer.

Upstream and downstream transitions from normal river channel to overbank excavation include gradual horizontal transitions with 3.5H:1V vertical changes.

The estimated volume of overbank excavation material is 36,500 bank cubic yards. A portion of the excavated overbank material will be placed adjacent to the landside and riverside of the levee or in the designated disposal locations as described in Section 6.14.

6.12 Borrow Area Selection and Design

Borrow areas and requirements for levee materials are identified and in the project construction package. The following paragraphs describe the process for selection and design of borrow sources for levee material.

6.12.1 PER Review

A desktop study identifying potential borrow sources was completed as part of the PER. The study concluded that suitable low permeability fill is in the valley walls of the Mouse River Valley corridor. Additionally, the study identified several borrow pits which have historically been used to construct levees and for flood-fighting efforts. Availability of suitable material within these pits is unknown.

A borrow source planning matrix was completed for the PER. The matrix compared the cost to obtain, deliver, and place the material in levee sections for each borrow location. As expected, the cost to utilize material adjacent to the levee was estimated to be half the cost of obtaining material from one or two offsite borrow locations.

6.12.2 Borrow Identification

The selection of borrow areas is being completed in compliance of Chapter 4 of EM 1110-2-1913 (reference [15]). The overlying objective for selecting a borrow source(s) is to identify low impermeable fill in excess of 200,000 cubic yards (the excess allowing for material shrinkage). The following sections describe the primary selection criteria used to identify, screen, and select borrow source locations. The borrow area locations will be designated for the contractor’s use. Borrow site constraints and requirements are as follows.

- Suitable Material - The borrow location must contain a sufficient quantity of impermeable soils meeting the requirements of Levee Fill consisting of lean clay, sandy clay, or silty clay soils. If the material is at an offsite borrow source, the source must be able to provide at least 100,000 cubic yards of suitable material. Additionally, the material must be readily obtainable without requiring significant construction and excavation activities. Suitable material must also be located above the water table or be obtainable at or near the material's optimum moisture content. Drying the material prior to placing and compacting it in the levee is not recommended.
- Material Proximity - The location of the borrow material must not create adverse impacts to the levee or surrounding structures and project features. Preference will be given to borrow locations near the levee. If suitable, material from existing levees will be used first. Alternate offsite locations will be identified in the construction documents based on proximity to the levee sections and haul truck accessibility.
- Property Ownership - Stakeholders who own and manage borrow areas, including Minot and the SRJB, will receive preference in selecting borrow material. Secondary preference will be given to cooperative landowners who will provide access to the material and minimal stipulations for its use. A property access and material-use agreement will be completed with the landowner prior to any borrow operations.
- Environmental Constraints - A desktop environmental study will be done prior to the selection of borrow locations. Borrow source areas within a wetland will not be considered, nor areas with cultural, archeological, or other environmental constraints.
- Restoration Requirements - Selection criteria for a borrow source includes minimal restoration requirements beyond replacing topsoil and re-establishing vegetation. Sites requiring construction and/or restoration of structures after material removal will be given low consideration.

6.12.3 Borrow Source Design

The borrow source design was completed in conformance with Chapter 4, Paragraph 4-4 of EM 1110-2 1913 (reference [15]); offsite borrow areas will be design based on owner preference and existing constraints.

6.12.4 Borrow Sources

The offsite borrow source location is assumed to be the Price borrow site currently being used as the borrow source for the Phase MI-2 & MI-3 Projects, shown in Figure 6-10.

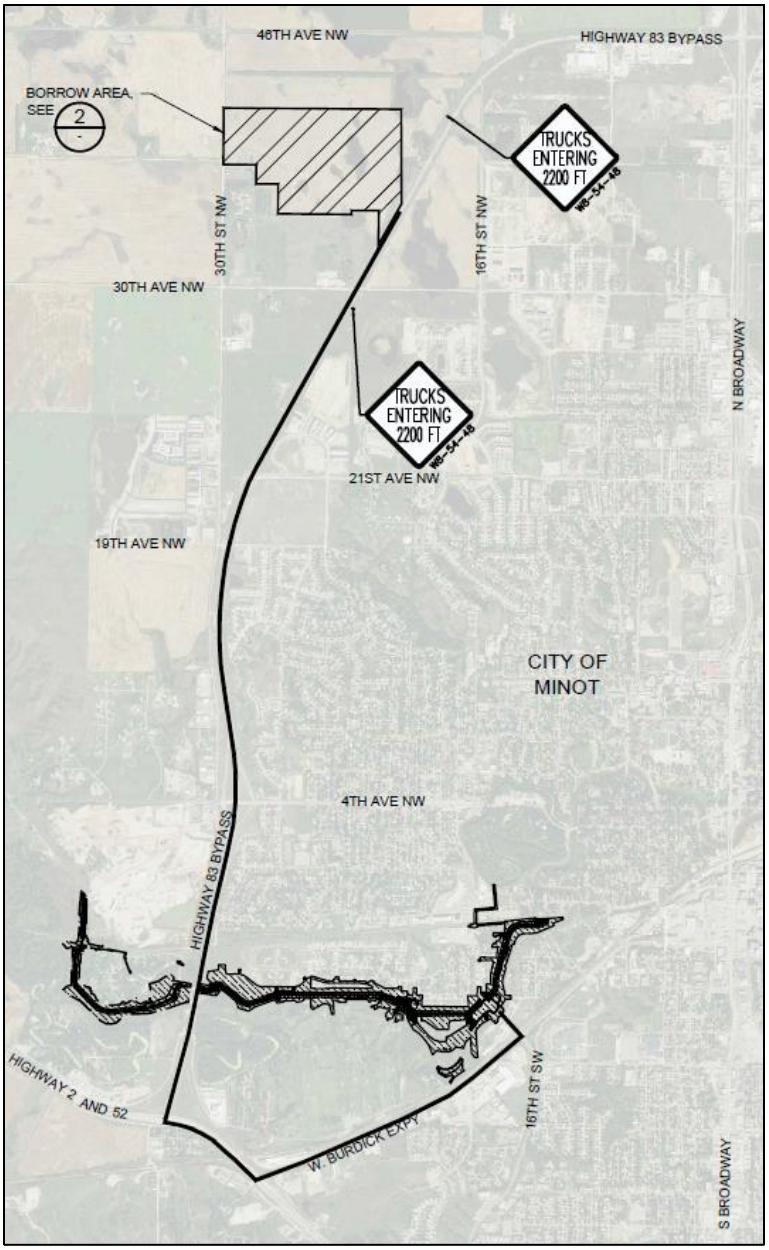


Figure 6-10 Potential Borrow Site Location

6.12.5 Borrow Source Selection

Negotiations and final agreements focused on the Price site as the preferred site are currently pending. Final determination of the borrow site will be described in the 100% design submittal after the completion of the negotiation process.

6.13 Earthwork Balance

The estimated volume of material required to construct the Phase WC-1 levees was calculated as described in the following sections. This section includes review earthwork volumes required to construct ancillary structures related to the levee and flood risk reduction Project.

6.13.1 Material Adjustment Factor

Chapter 4 of EM 1110-2-1913 (reference [15]) suggests that a shrinkage factor of at least 25 percent should be used to account for material shrinkage during placement and losses during excavation and hauling. Shrinkage and swell factors were refined based on the soil investigation completed in the preferred borrow pit. The shrinkage factor is used to calculate the volume of borrow soil required to construct the levees. The swell factor is used to calculate the loose volume of soil during transport from the borrow pit to the project site.

In-situ soil properties, when compared to standard Proctor test results, indicate a shrinkage factor of between 5–10 percent. A shrinkage factor of 10 percent was used to account for material shrinkage during placement. An additional 5-percent waste factor was also applied to the neat-line volume to account for losses during excavation, hauling, and placement. A swell factor of 40 percent, typical for clay, was applied to the borrow volume of low permeability fill in the borrow pit that will be used as levee fill. A swell factor of 25 percent was applied to material excavated from the overbank excavation areas. Summaries of levee and overbank excavation volumes are provided in Section 6.13.3.

6.13.2 Methodology for Volume Calculations

A three-dimensional surface was developed in AutoCAD Civil 3D (software) for the various phases and tasks including the existing ground, the existing ground after demolition, the exploration trench, the proposed overbank excavation, the proposed fill, and the proposed levee and ramps. A surface to surface volume calculation was completed for each item as identified in the measurement and payment section of the specifications. The volumes were checked based on average end area method with cross sections at 100-foot intervals. The following describes the volume calculations provided in Table 6-5 .

- “Fill Neat-Line Volume” quantities reflect the volume between the existing ground (or ground after levee removal) and finish ground surface and the volume required to backfill the exploration trench. The volumes calculated were also adjusted to account for 6 inches of topsoil placement on the levee slopes and for the gravel section at the crest of the levee.
- “Fill Borrow Volume” quantities reflect the volume of undisturbed fill material required to construct each portion of the levee. These calculations include a 10-percent shrinkage factor and a 5-percent waste factor and are used for borrow pit sizing calculations.
- “Fill Loose Volume” quantities reflect the loose volume of fill material required to construct each portion of the levee. These quantities include a 40-percent swell factor to reflect the hauling volume which is used for cost estimating purposes.
- ”Cut Neat-Line Volume” quantities reflect the volume between the existing ground surface and the proposed overbank excavation surface or proposed finished grade and do not include any adjustment factors.
- “Cut Loose Volume” quantities reflect the loose volume of excavation material during transport and are used for cost-estimating purposes. These quantities include a 25-percent swell factor.

6.13.3 Summary of Levee and General Earthwork Volumes

Table 6-5 shows the volume of material necessary to construct the levee, the total estimated volume of material required from borrow locations for levee construction, and the total estimated volume of material required during transport from the borrow pit to the levee. Neat-line and loose overbank excavation volumes are also provided in Table 6-5. Volumes are summarized by phase and station.

Table 6-5 Earthwork Summary

	Fill Neat-Line Volume (CY)	Fill Placed Volume (CY)	Fill Import Loose Volume (CY)	Cut Neat-Line Volume (CY)	Cut Loose Volume (CY)
Source Excavation					
Levee Removal				14,500	18,100
Exploration Trench				25,800	32,200
Overbank Excavation				36,500	45,600
Common Excavation				19,200	24,000
Levee Fill/Import					
Exploration Trench Backfill	25,800	29,700	41,600		
Levee Core	151,800	174,600	244,400		
Common Fill					
Common Fill – Wet Side	15,700	18,000			
Common Fill – Dry Side Station 48+00F to 55+00F	24,300	28,000			
Common Fill – Dry Side Misc.	8,100	9,300			
Export				**55,300(1)**	**77,400**

(1) It is anticipated that the grading between Levee Stations 48+00F and 55+00F can be adjusted to utilize excess excavated material and reduce the need for exporting waste material. This will be further evaluated for 90% design.

6.14 Disposal Options

An excess volume of soil material unsuitable for levee fill is expected to be generated from within the construction limits. Work expected to generate unsuitable material for Phase WC-1 includes the following:

- Overbank excavation
- Removal of the existing levee
- Grading activities required for drainage improvements Installation of infrastructure such as gatewells, storm drains, culverts, and water lines

- Excavation of exploration trench

Spoils generated from these activities may be used in other project features if they meet the designated material specifications. Spoils that are not suitable for use in the levee or other project features or that exceed fill requirements must be disposed in designated and approved areas.

6.14.1 Disposal Areas

Potential disposal areas include the Minot Landfill, landside of the levee from Station 10+00F to Station 19+00F and Station 48+00F to Station 59+00F, access ramps, and the borrow area(s) used to supply levee fill material. Disposal areas will be prioritized according to their proximity to the source of the waste material. The following provides a summary of each disposal location.

6.14.1.1 Landfill Sites

The Minot landfill is approximately 1.5 miles from the Phase WC-1 project site. The landfill site could potentially be the largest excess soil disposal site. Daily cover soil material is needed for landfill operations and would be accepted at no cost. Transportation costs would need to be evaluated to determine the feasibility of this option.

The Sawyer Landfill is also a potential disposal site for non-hazardous waste located approximately 25 miles southeast of the Phase WC-1 project.

6.14.1.2 Landside of Levee

Disposal of material immediately adjacent to the landside of the levee but outside of the levee prism is a potential option in two locations.

- North of the railroad (Sta. 10+00F to 19+50F). Fill is needed to correct drainage along the property line between the levee and gravel products. Although the levee is approximately 10′ tall, the crest is only approximately 3′ above the existing grade within Gravel Products and the area between the levee and Gravel Products can be filled to a point that still allows a drainage swale to be constructed. A detailed grading plan of the designed disposal site is shown in the construction drawings (Appendix K).
- Moe Property (Sta. 48+00F to Sta. 59+00F). This location is on private property, however, the property owner has indicated that he is open to filling and leveling the property. Disposed material totals approximately 24,000 cubic yards (Appendix K).

6.14.1.3 Access Ramps

Access ramps are required throughout the levee project. Disposal material could potentially be used for these levee ramps, as long as it meets the requirements outlined in the Construction Documents.

6.15 Municipal Utilities

Municipal utility modifications are required due to levee construction, interior drainage improvements, or roadway modifications. Affected utilities will include watermain, and storm sewer. Utilities crossing under the levee are designed based on USACE EM 1110-2-1913 (reference [15]).

Existing utility types and locations are defined by as-built and GIS resource information supplied by Minot and on-site data collection of visible surface items.

Water and sanitary sewer utilities have been designed in accordance with Minot *Standard Specifications and Details–2013* (reference [55]) and the Great Lakes-Upper Mississippi River Board's *Ten States Standards for Water and Wastewater Facilities* (reference [56], reference [57]). NAWS pipeline crossings and modifications are designed in accordance with NDSWC requirements.

Where practical, utilities are located within defined right-of-way corridors. In locations where this is not practical, permanent utility and construction easements will be acquired. In addition, water and storm sewer terminations and mains paralleling the levee are generally placed horizontally no closer than 15 feet from the nearest flood protection feature (toe of slope, etc.). Water and storm sewer lines will be installed via open cut trench, unless otherwise specified in the plans.

All new construction of NAWS pipelines shall be in accordance with the plan details. NAWS ductile iron pipe are poly-encased, cathodic protected (impressed current), and sand bedded from 6 inches below the pipe to the spring line. The integrity of the polyethylene encasement and cathodic protection conductors must be maintained. Casing of existing NAWS lines shall be cased in place. A representative of the NDSWC shall be present during excavation activities within the NAWS easement and during construction or modification to NAWS pipelines.

6.16 Municipal Roadway Modifications

There are five local roads located within the Phase WC-1 project limits, Fifth Avenue SW, Eighth Avenue SW, Thirty-Sixth Avenue SW and Thirty-Seventh Avenue SW, and U.S. Highway 83 Bypass Access.

Coordination is ongoing with the North Dakota Department of Transportation (NDDOT) regarding the U.S. Highway 83 Bypass. The current Fifth Avenue SW access to the U.S. Highway 83 Bypass was relocated in 2018 and will remain in place.

It is anticipated that Fifth Avenue SW will be used as a haul road to construct the proposed levee system. The current roadway is not designed to carry heavy loads and will need to be reconstructed after the levee has been constructed.

6.16.1 Eighth Ave SW

In order to convey drainage away for the proposed levee and to collect stormwater runoff, Eighth Avenue SW will be reconstructed to remove the existing crown and slope away from the proposed levee.

6.16.1.1 Sight Distance Triangles

Proposed roadway improvements are being restored in the current roadway locations and at similar grades. Sight distance triangles have not been calculated/corrected.

Detailed traffic control plans for construction impacting roadways are shown in construction drawings G-109 through G-118 (Appendix K). Work zone traffic control signing will be in accordance with the North Dakota Department of Transportation Standard Specifications for Road and Bridge Construction, 2014 Edition (reference [58]) and Standard Drawings along with the Manual on Uniform Traffic Control Devices (MUTCD), 2009 Edition (reference [59]).

6.17 Franchise Utilities

Discussions are ongoing with franchise utilities in the area that will be affected. The known franchise utilities are: Souris River Telephone (SRT), Montana-Dakota Utilities (MDU), Xcel Energy, and Midcontinent Communications.

Franchise utilities serving residential properties in the levee footprint will be removed prior to levee construction. One overhead electrical river crossing has plans to be removed, and another overhead electrical line will be moved and raised to meet the USACE levee crossing requirements. Multiple utilities are buried along the shoulder of the U.S. Highway 83 Bypass. EM 1110-2-1913 (reference [15]) will be followed to determine the proper crossing method across the proposed levee.

6.18 Landscape Design

Landscaping within the project limits has been designed to replace the trees and shrubs with permanent perennial ground cover to stabilize the levee and surrounding areas following guidance outlined in ETL 1110-2-571 Guidelines for Landscape Planting and Vegetation Management at Levee, Floodwalls, Embankment Dams, and Appurtenant Structures (reference [47]). The design considers existing conditions of the surrounding riparian corridor, neighborhood, and the existing greenway corridor planning developed by Minot as part of their Riverfront and Center neighborhood plans. Landscape includes the following:

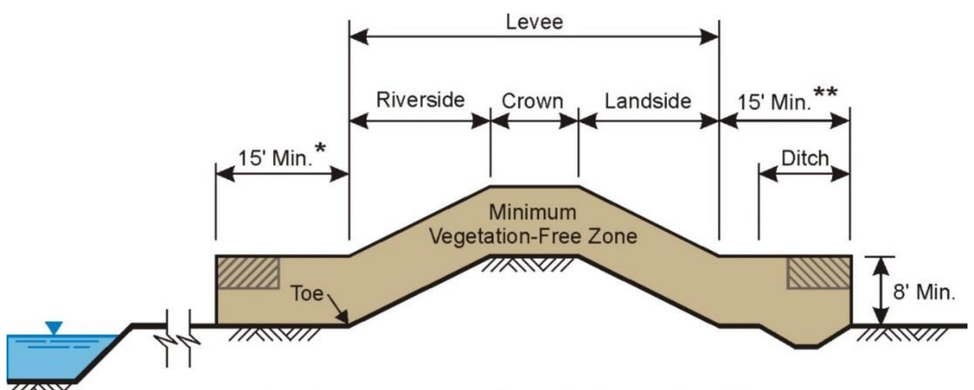
- Low maintenance ground cover turf mix in "Vegetation-Free Zones."
- Native perennial vegetation seed mixes for wetland and natural areas outside of the levee "Vegetation-Free Zones."

The landscape design process builds off of previous levee project restoration efforts. Seed mixtures for the "Vegetation-Free Zones" and natural areas were carefully developed as part of phases 2 and 3. For Phase

WC-1 the seed mixtures have been delineated to thrive in the corresponding growing conditions as well as to blend in with the surrounding landscape character.

Vegetation-Free Zone

Trees, shrubs, and native grasses will only be planted outside of the Vegetation-Free Zone as described by the USACE. The Vegetation-Free Zone of the levee is delineated in Figure 6-11:



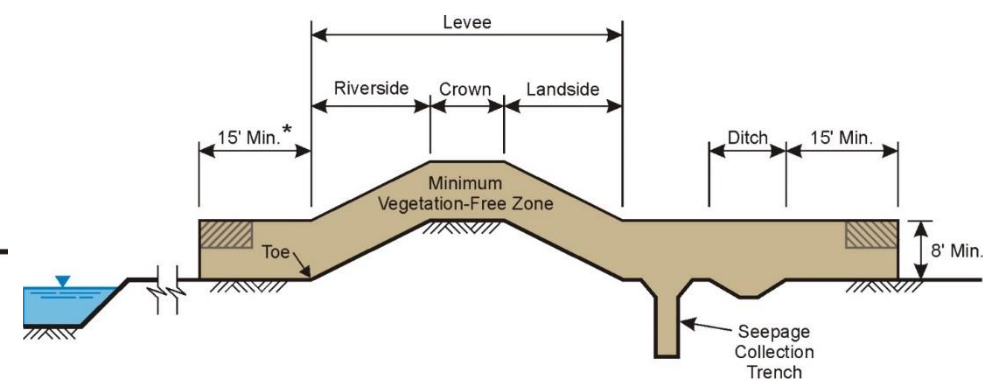
* 15' or Distance to Edge of Normal Water Surface, if Less.

** Varies Depending on Location of Ditch (15' Minimum).

In this 4' x 7' Transition Zone, Temporary Obstruction by Limbs and Crown is Allowed During Development of New Plantings, for up to 10 Years.

Normal Water Surface

Levee Section - Basic

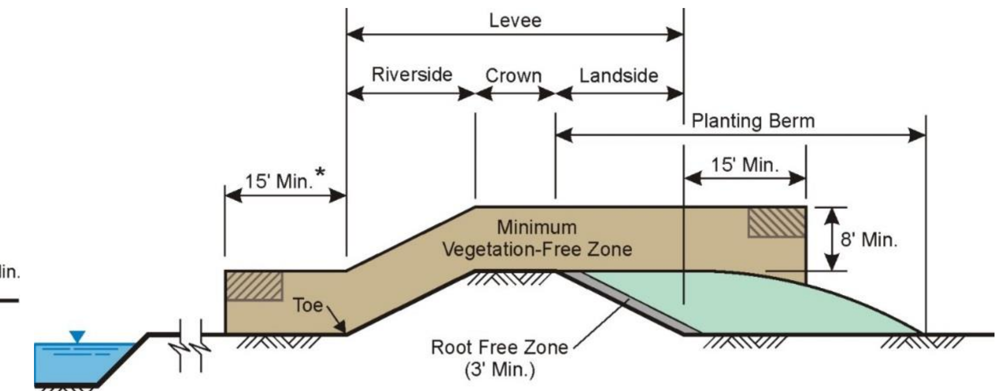


* 15' or Distance to Edge of Normal Water Surface, if Less.

In this 4' x 7' Transition Zone, Temporary Obstruction by Limbs and Crown is Allowed During Development of New Plantings, for up to 10 Years.

Normal Water Surface

Levee Section with Seepage Collection Trench and Seepage Collector Ditch

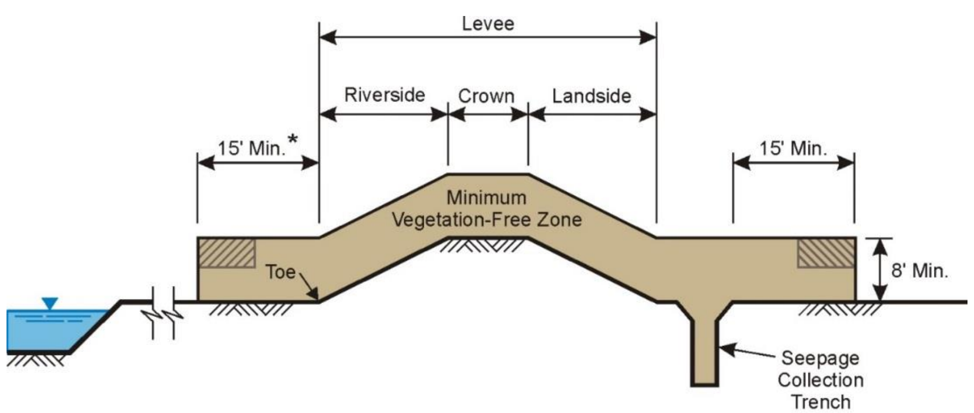


* 15' or Distance to Edge of Normal Water Surface, if Less.

In this 4' x 7' Transition Zone, Temporary Obstruction by Limbs and Crown is Allowed During Development of New Plantings, for up to 10 Years.

Normal Water Surface

Levee Section with Planting Berm - Preferred

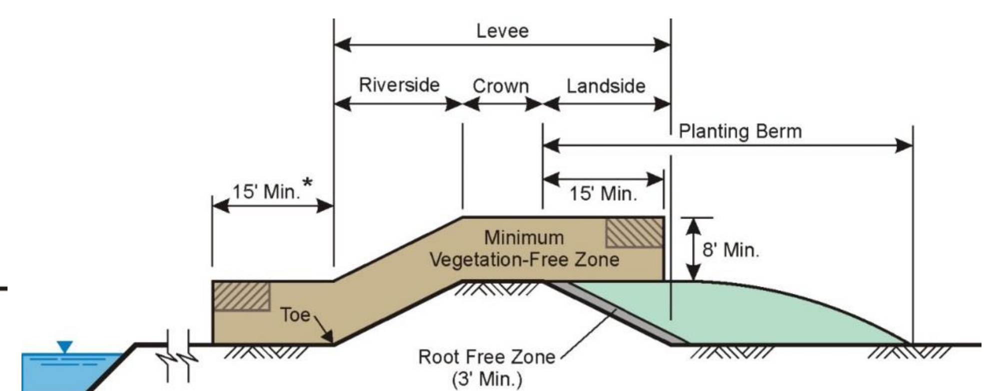


* 15' or Distance to Edge of Normal Water Surface, if Less.

In this 4' x 7' Transition Zone, Temporary Obstruction by Limbs and Crown is Allowed During Development of New Plantings, for up to 10 Years.

Normal Water Surface

Levee Section with Pervious Seepage Collection Trench

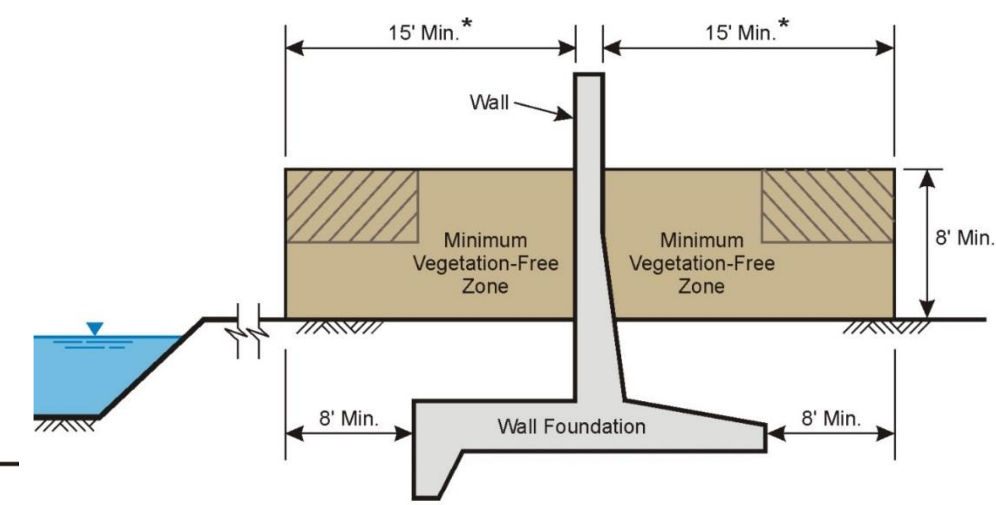


* 15' or Distance to Edge of Normal Water Surface, if Less.

In this 4' x 7' Transition Zone, Temporary Obstruction by Limbs and Crown is Allowed During Development of New Plantings, for up to 10 Years.

Normal Water Surface

Levee Section with Planting Berm - Minimum



* 15' or Distance to Edge of Normal Water Surface, if Less.

In this 4' x 7' Transition Zone, Temporary Obstruction by Limbs and Crown is Allowed During Development of New Plantings, for up to 10 Years.

Normal Water Surface

Inverted T Type Floodwall

Figure 6-11 Project Defined Vegetation Free Zone

6.19 Restoration

Areas that are disturbed as a result of construction activities will be revegetated to (1) prevent erosion and sedimentation, (2) stabilize the levee and associated appurtenances, and (3) restore a natural and aesthetic appearance. Vegetation will be installed using a variety of methods in different locations, as specified. Methods and general locations for restoration measures include the following:

- Topsoil and sod—Minot right-of-way, critical infrastructure green space
- Topsoil, hydraulic soil stabilizer, and seeding—open space areas, levee slopes, and all general disturbed areas
- Topsoil with erosion control blanket and seeding—critical slope areas typically 3H:1V or steeper, concentrated stormwater flow areas, slopes immediately adjacent to wetlands or other environmentally sensitive areas
- Topsoil with TRM and seeding—slopes around critical infrastructure such as closures, headwalls, area receiving concentrated stormwater flow, channel protection
- Site restoration on newly constructed or rehabilitated levee and in the vegetation-free zone shall consist of permitted perennial grasses which are mowable to 3 inches per guidance in Section 4 of ETL 1110-2-571 (reference [47]). Seeding rates will be based on the seed mixture selected. The levee slopes will be temporarily protected from erosion with hydromulch with tackifier spread at the specified rates. Where necessary, measures will be used to prevent erosion and control sedimentation until vegetation cover is achieved. These measures include silt fence, floating silt fence, temporary sedimentation basins, hydromulch, temporary seeding, and 100% biodegradable temporary erosion protection blankets. Such measures will be installed and maintained in accordance with the Stormwater Pollution Prevention Plan (SWPPP).

The contractor will be required to provide a planting schedule, including start and completion dates. Four seed types have been specified:

1. Levee turf seed mix: This mix is dominated by fescues that will form a dense, resilient cover. It does not require weekly mowing and forms a thatch that inhibits woody plant establishment.
2. High performance turf seed mix: This mix of Kentucky bluegrass, perennial rye and fescue will be planted in areas outside the levee No Vegetation Zone for park and residential lawns that require regular mowing.
3. Prairie mix: This mix of native grasses will be planted on city property outside the No Vegetation Zone in areas that are not to be actively used by people.
4. Wet meadow mix: This mix of native grasses will be planted on city property outside the No Vegetation Zone in areas where seasonal water will intentionally inundate.

5. Economy construction mix: This simple mix of four native pasture grasses will be used on the soil borrow site.

Performance requirements and monitoring for restoration/vegetation establishment are in the construction documents.

6.20 USACE Inspection Items

The USACE performed an inspection of the existing levee system in September 2017 and developed a *2017 Routine Inspection Report* (reference [60]), dated June 2018. Numerous items have been identified as minimally acceptable or unacceptable and include the following:

- Unwanted vegetation growth
- Gatewell structure corrections
- Riprap corrections
- Encroachments
- Erosion/bank caving
- Corrections to culverts or discharge piping

Appendix E-3 includes maps and descriptions from the inspection report, outlining the deficiencies and required work items. The design team georeferenced the inspection deficiency points using the USACE 2017 Periodic Inspection figure (Figure 6-12). Table 6-6 identifies and describes each deficiency with the anticipated correction. The construction drawings (Appendix K) show the location of each deficiency and includes a table summarizing each deficiency and the planned correction.

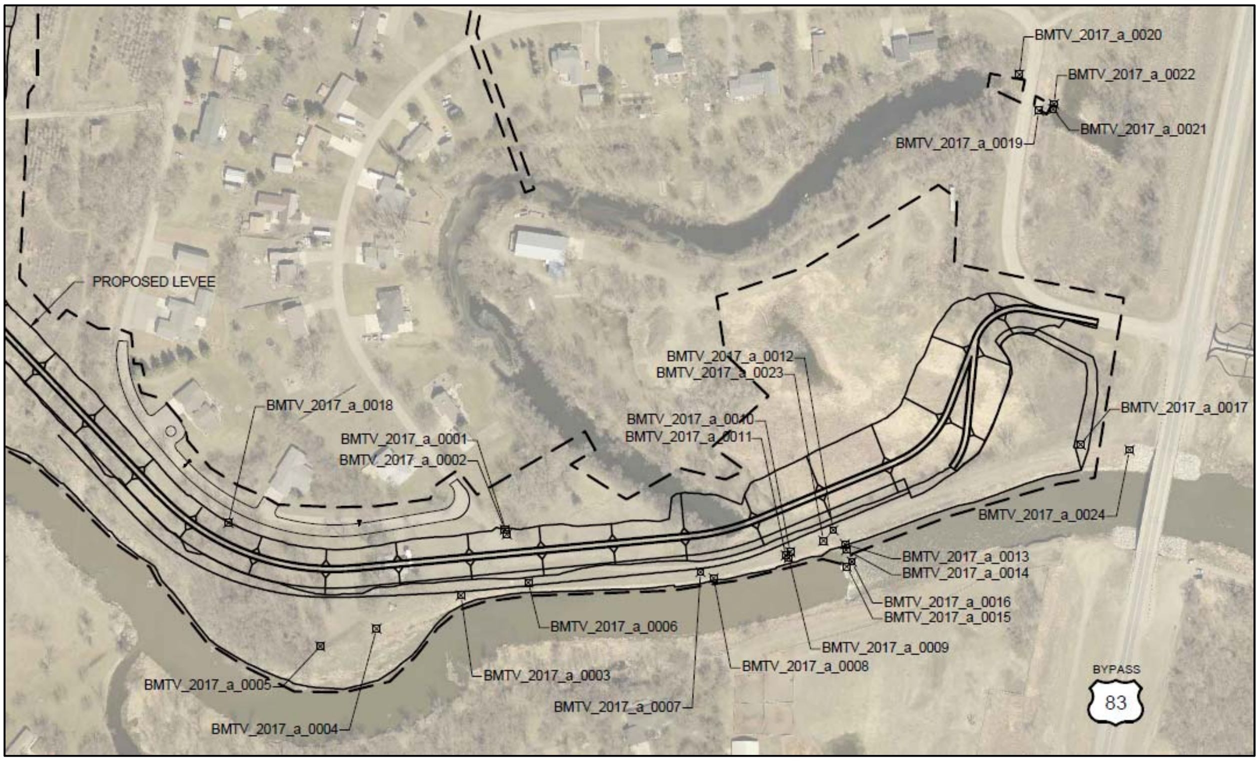


Figure 6-12 USACE Inspection Items

Table 6-6 USACE Inspection Items

USACE Inspection Deficiency ID	Remark Summary (2017 Rating)	Correction
BMTV_2017_a_0001	Houston Engineering Inc. has inspected via video recording the project's interior drainage system culverts as of 2017. per sponsor, no issues were located.	None required; rated as acceptable in 2017 report. next video inspection to take place in 2022.
BMTV_2017_a_0002	Culvert opening is being obstructed more than 10 percent by sedimentation	Culvert to be removed according to demolition plan.
BMTV_2017_a_0003	Riverside riprap not visible, may have been displaced or covered with silt. grassy vegetation covers area.	Existing slope protection to be modified according to new flood protection system and resulting hydraulics.
BMTV_2017_a_0004	Miscellaneous residential encroachments including fencing, playground. on the landside slope and toe. a light pole is no longer present	Removal of existing structures and encroachments within project footprint defined on drawings.
BMTV_2017_a_0005	The federal project ties into a discontinuous emergency embankment at the upstream extent of the project. 2017 note: overgrown vegetation prevented further inspection	Levee alignment and profile will be reconstructed according to resulting hydraulics per federal design standards. Vegetation will be removed.
BMTV_2017_a_0006	6 ft diameter pile of concrete dumped on levee crown and riverside toe	Concrete debris removal incidental to levee removal.

USACE Inspection Deficiency ID	Remark Summary (2017 Rating)	Correction
BMTV_2017_a_0007	Trees (> 2 inches in diameter) and long vegetation located on landside levee slope	Removal of undesired vegetation completed during clearing and grubbing within project footprint.
BMTV_2017_a_0008	Rutting less than 6" deep from vehicle traffic on levee crown	Existing levee to be removed and replaced with new levee. Vehicle chain barriers to be installed at levee crown access points.
BMTV_2017_a_0009	Burlington to Minot - Tierrecita Vallejo system does not have a pump station but utilizes portable pumps stored at the north county garage. 2017 note: pumps were unavailable for inspection, although sponsor noted no issues.	None required; rated as acceptable in 2017 report. The Bark Park Pump Station (currently under construction on Phase MI-2, by others) will eventually be used to control water levels in the oxbow.
BMTV_2017_a_0010	The position indicator cover for gatewell no. 1 (oxbow inlet) is weathered and prevents viewing the indicator	None required; existing gatewell to be removed and replaced with new internal drainage system.
BMTV_2017_a_0011	The plexiglass cover on the stem cover viewing window for gatewell no. 1 (oxbow inlet) is broken	None required; existing gatewell to be removed and replaced with new internal drainage system.
BMTV_2017_a_0012	Metal fence posts on the landside levee slope remain; wiring from the fence was removed	Fence to be removed according to demolition plan.
BMTV_2017_a_0013	Local sponsor is not maintaining bypass gatewell	TBD
BMTV_2017_a_0014	The project levee embankment culverts have not been videotaped or visually inspected within the past 5 years. 2017 note: these culverts were not inspected as part of the 2017 Houston Engineering Inc. culvert inspection.	None required; existing levee embankment culverts to be removed and replaced with a new internal drainage system.
BMTV_2017_a_0015	The flood damage reduction channel control structure is bowing downstream. 2017 note: riprap below the structure has displaced, and large logs are wedged on top of the structure at the bow	TBD
BMTV_2017_a_0016	Riprap on riverside slope is overgrown with brush and saplings	Existing slope protection to be modified according to new flood protection system and resulting hydraulics.
BMTV_2017_a_0017	Trees (> 2 inches in diameter) and long vegetation located on landside levee slope	Removal of undesired vegetation completed during levee removal and clearing and grubbing within project footprint.
BMTV_2017_a_0018	Overgrown grass, large dense tree cover, and brush cover the levee crown, slopes, toes, and vegetation-free zone	Removal of undesired vegetation completed during levee removal and clearing and grubbing within project footprint.

USACE Inspection Deficiency ID	Remark Summary (2017 Rating)	Correction
BMTV_2017_a_0019	Large trees and long vegetation growth surrounding the gatewell	Removal of undesired vegetation completed during clearing and grubbing within project footprint.
BMTV_2017_a_0020	Large debris blocking inlet.	Debris to be removed according to demolition plan.
BMTV_2017_a_0021	Fencing and latches on gatewell structure is missing locks. gatewell is stuffed with leaves.	Leaves to be removed according to demolition plan. Locks to be replaced on fence.
BMTV_2017_a_0022	Significant corrosion on gatewell. plexiglass gate indicator is fogged completely and cannot be read.	Corrosion to be removed from metallic surfaces and painted with corrosion resistant paint and plexiglass indicator to be replaced.
BMTV_2017_a_0023	O&M manual for system is maintained at Ackerman-Estevold offices in Minot, ND. point placed for future inspections.	New O&M manual to be provided for new flood risk reduction system.
BMTV_2017_a_0024	Construction debris surround bridge placed on levee crown and slopes.	Deficiency corrected, bridge replacement project has been completed, and construction debris removed.

Table 6-7 Earthwork Summary

	Fill Neat-Line Volume (CY)	Fill Placed Volume (CY)	Fill Import Loose Volume (CY)	Cut Neat-Line Volume (CY)	Cut Loose Volume (CY)
Source Excavation					
Levee Removal				14,500	18,100
Exploration Trench				33,300	41,600
Overbank Excavation				18,200	22,700
Common Excavation				1,100	1,400
Levee Fill/Import					
Exploration Trench	33,300	38,300	53,600		
Levee	145,400	167,200	234,100		
Common Fill					
Overbank Excavation	30,100	34,600			
Levee Station 48+00F to 55+00F – Dry Side	25,500	29,300			
Common Fill	8,100	9,300			
Import Common Fill		6,100	8,500		

7.0 Structural Design

7.1 Introduction

This section presents the structural design basis for structural features located within Phase WC-1 specifically the Road and Rail Closure, Tierrecita Vallejo Gatewell, and existing U.S. Highway 83 Gatewell weir. Aspects specific to each project feature such as specific analysis assumptions, loading values, and resulting factors of safety are described in Sections 7.4, 7.5, and 7.6, respectively.

7.1.1 Structural Locations

Station locations for each structure related to Levee Alignment F (refer to Section 6.0), which represents the centerline of the levee crest, is presented in Table 7-1.

Table 7-1 Gatewell and Closure Structure Locations

Structure	Beginning Station	End Station
Road and Rail Closure	STA 18+86.29	STA 21+23.09
Tierrecita Vallejo Gatewell	STA 46+19	STA 46+19
U.S. Highway 83 Gatewell(1)	N/A	N/A

(1) The U.S. Highway 83 Gatewell is within the interior of the leveed area and is not located along the levee centerline. Refer to Sheet G-104 for location information.

7.2 Technical Guidance and Reference Standards

These features were designed according to the applicable USACE engineering regulations (ERs), engineering manuals (EMs), engineering technical letters (TLs), and engineering circulars (Ecs) as described in the Project Design Guidelines (Appendix N). Any aspects of the design that divert from the Project Design Guidelines (Appendix N) are noted herein.

7.2.1 Performance Objectives

Performance objectives for hydraulic structures on the project listed in Table 7-2. As described in the Project Design Guidelines (Appendix N), these performance objectives were adopted from Table 1 of ECB 2017-2 (reference [61])for critical sections following the intent of guidance in EM 1110-2-2502 (reference [62]) and EM 1110-2-2607. This table supersedes Table 3-1 of EM 1110-2-2100.

Table 7-2 Load Categories to Satisfy Performance Requirements

Load Condition Categories	Return Period	Annual Exceedance Probability (AEP)
Usual	10-Year Event	10%
Unusual	10- to 750-Year Event	10% - 0.133%
Extreme	Greater than 750 years or Top of Structure	Less than 0.133%

7.2.2 Global Stability Criteria

Global stability criteria for sliding, overturning, bearing, and floatation are evaluated in accordance with EM 1110-2-3104 (reference [63]), EM 1110-2-2502 (reference [62]), EM 1110-2-2100 (reference [64]), as applicable. The minimum factors of safety for stability of critical structures with ordinary site information (as defined in EM 1110-2-2100 (reference [64])) are listed in Table 7-3. The failure mechanisms (sliding, overturning, bearing, and floatation) are described in EM 1110-2-2100 (reference [64])).

Table 7-3 Global Stability Criteria

Condition	Usual (U)	Unusual (N)	Extreme (X)	Reference
Sliding	2	1.5	1.1	EM 1110-2-2100 (reference [64])
Overturning	100% Base in Compression	75% Base in Compression	Resultant within Base	ECB 2017-2 (reference [61]
Bearing	3.5	3.0	2.0	EM 1110-2-2100 (reference [64])
Floatation	1.3	1.2	1.1	EM 1110-2-2100 (reference [64])

7.2.3 Allowable Bearing Capacities

Allowable bearing capacities for each load category are shown in Table 7-4 and are discussed in greater detail in Section 2.9.1. The baseline allowable bearing pressure was modified for the unusual and extreme per Section 3-10 EM 1110-2-2100 (Reference [64]).

Table 7-4 Allowable bearing pressure – closure structure

USACE Load Category	Allowable Bearing Capacity(1), psf	FS Modifier(2)	Modified Allowable Bearing Capacity(1), psf
Usual	3,000	1.0	3,000
Unusual		1.15	3,450
Extreme		1.5	4,500

(1) Section 2.9
(2) EM 1110-2-2100 (Reference [64]) Section 3-10 (pg. 3-6)

7.2.4 Concrete Design Load Factors

Reinforced concrete should be designed per EM 1110-2-2104 (reference [65]). Structures are designed to accounting for usual and unusual events that are likely to occur during the service life of the structure using single load factors listed in Table 7-5. The structures are evaluated for extreme loading events that are possible, but unlikely to occur during the service life of a structure. For these cases, the load cases listed in Table 7-5 are intended to provide adequate reliability against exceeding strength limit states. The load factors in Table 7-5 are applied in the determination of the required nominal strength for all combinations of axial, moment, and shear. Shear reinforcement is designed for the excess shear, the difference between the factored ultimate shear force and the shear strength provided by the concrete per Chapter 5 of EM 1110-2-2104 (reference [65]).

Table 7-5 Applicable Concrete Design Load Factors

Load Category	Variable	Usual (U)	Unusual (N)	Extreme (X)
		γ_U	γ_N	γ_X
Dead	D	2.2[4]	1.6[4]	1.2[1], 0.9[2]
Vertical Earth	EV	2.2[4]	1.6[4]	1.35[1], 1.0[2]
Lateral Earth	EH	2.2[4]	1.6[4]	1.35[3], 0.9[3]
Hydrostatic	Hs	2.2[4]	1.6[4]	1.3
Soil Surcharge	ES	2.2[4]	1.6[4]	1.3
Wind	W	NA	1.6[4]	NA

(1) Applied when loads add to the predominant load effect.
(2) Applied when loads subtract from the predominant load effect.
(3) Load Factors for structures using at-rest pressure for design:
Driving Pressure = 1.35; Resisting Pressure = 0.9.
(3) For members in direct tension (net tension across the entire cross section): Usual load factor = 2.8, Unusual load factor = 2.0

7.2.5 Concrete Strength Resistance Factors

Strength reduction (resistance) factors from Chapter 21 of ACI318-14 were used in the design as shown in Table 7-6.

Table 7-6 Strength Reduction Factors (ACI 318-14)

Application	Reduction Factor
Tension controlled sections	0.9
Compression controlled sections with spiral reinforcing	0.75
Other compression controlled sections	0.65
Shear and torsion	0.75
Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models)	0.65
Post-tensioned anchorage zones	0.85
Strut-and-tie models, and struts, ties, nodal zones, and bearing areas in such models	0.75
Flexure sections without axial load in pre-tensioned members where strand embedment is less than the development length	0.85

7.3 Design Considerations

7.3.1 Water Surface Elevations

The MREFPP hydraulic models were used to calculate the water surface elevations for the 10percent annual exceedance probability (AEP), 1percent AEP, design flood elevation, and minimum design grade surface elevations in support of the structural analysis. The minimum design grade corresponds with the top of levee. A discussion of the hydraulic modeling is presented in Section 3.0. Water surface elevations used to determine hydrostatic loading on structural features is in Table 7-7.

Table 7-7 Structure Flood Elevations

Structure	10% AEP	1% AEP	Design Flood Elevation (2)	Minimum Design Grade (3)
Road and Rail Closure	1556.00	1562.80	1570	1573.74
Tierrecita Vallejo Gatewell	1555.00	1562.20	1572.20	1573.19
U.S. Highway 83 Gatewell(1)	N/A	N/A	N/A	N/A

(1) Gatewell is within leveed area and therefore is not subject to flood loading
(2) Corresponds to the flood of record, Refer to Table 3-3
(3) Corresponds to the top of the levee adjacent to the structure

7.3.2 Structural Superiority

Structural superiority for the MREFFP generally involves adding height to project features to control the location of overtopping if a flood event exceeds the capacity of the system thus reducing the potential for scour on the Landside of the structures. This results in the structures being taller than adjacent levee features. Elevations for the minimum design project grade, the top of levee adjacent to structures, and the top of structures with structural superiority are in Table 7-8.

Table 7-8 Structure Elevations

Structure	Minimum Design Project Grade (feet) (1)	Top of Levee at Structure (feet)	Top of Structure Elevation (feet)
Road and Rail Closure	1573.00	1573.74	1575.74
Tierrecita Vallejo Gatewell	1572.2	1573.19	1575.86
U.S. Highway 83 Gatewell	N/A	N/A	1560.50

(1) Refer to Table 3-3

7.3.3 Soil Parameters

Soil parameters established for Phase WC-1 (Section 2.6.3) and used in the structure design are defined in Table 7-9.

Table 7-9 Structural Soil Parameters

Parameter	Value
Moist Unit Weight (pcf)	115
Friction Angle (deg)	32
Cohesion (psf)	0

7.3.4 Frost

The foundations of all structures covered under this section will be founded below the design frost depth. According to the local building code minimum frost depth is 4 feet below the ground for heated structures. The minimum frost depth for foundations is 6 feet below the ground surface to the bottom of the footing for non-heated structures in accordance with EM 1110-1-1905 (reference [16]) and Design Guidelines. Table 7-10 includes elevations for the bottom of each structure, the sill elevation, and the ground surface elevation representing that the frost design consideration has been incorporated.

Table 7-10 Foundation Elevations

Structure	Bottom of Structure Elevation, ft	Sill Elevation, ft	Ground Surface Elevation, ft
Road and Rail Closure	1554.00	1566.00	1566.00
Tierrecita Vallejo Gatewell	1539.70	1542.20	1573.19
U.S. Highway 83 Gatewell	1543.37	1550.00	N/A(1)

(1) Ground surface has not been surveyed at the location of the gatewell. The gatewell is existing and thus the frost susceptible is not within the scope of this design.

7.4 Road and Rail Closure

The rail and road closure (Figure 7-1) is a removable aluminum stoplog closure to accommodate the rail and access road and transition sections on either end of the closure structure connecting the closure section to the levee. The top elevation of the concrete wall and stoplogs are set higher than the adjacent levee for structural superiority (refer to Section 7.3.2 for additional discussion). The central portion of the closure structure includes removable stoplogs and posts that will be installed during a flooding event. Three transition floodwall monoliths are located at the ends of the closure structure with two sections transitioning to the levee north of the closure and one transitioning to the levee south of the closure.

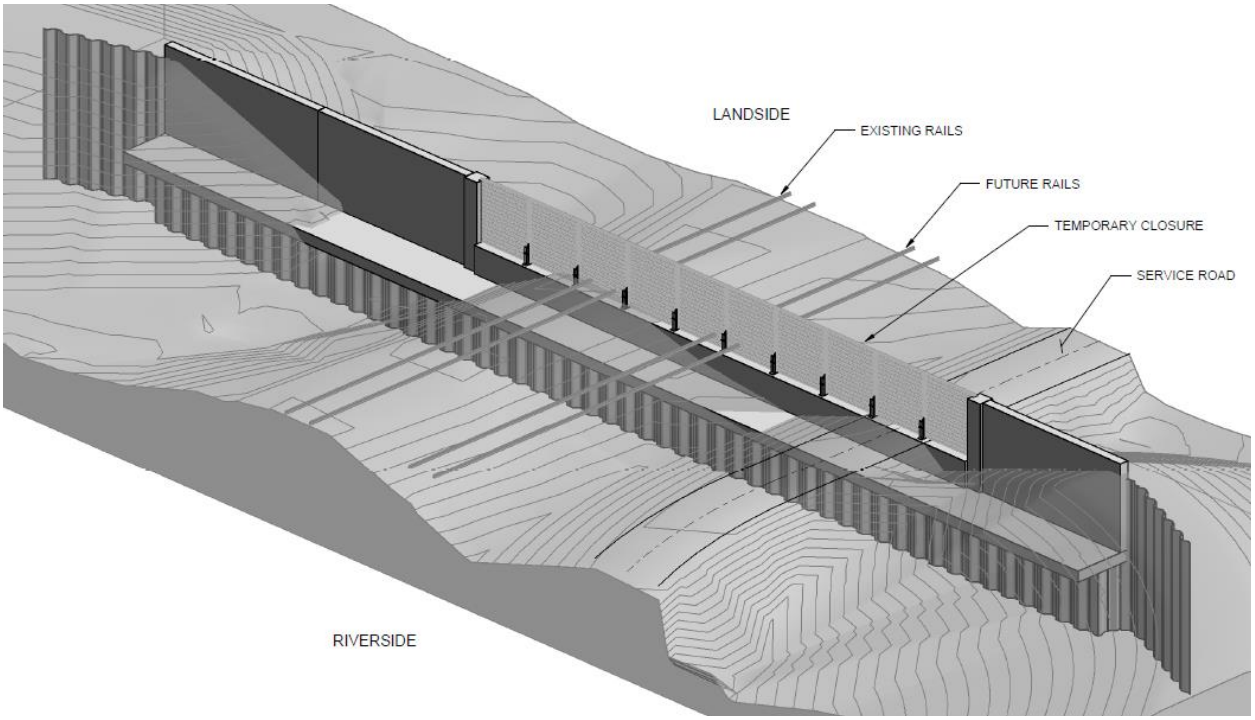


Figure 7-1 Road and Rail Closure

7.4.1 Rail and Road Closure Decisions

Decisions made during the design of the rail and road closure are discussed below.

7.4.1.1 Opening Width

The opening width was defined to allow the service road, existing rails, and future rails to pass through the openings. Because the exact location of the future rails is not known at this time, a decision was made to have the entire center portion of the closure structure open.

7.4.1.2 Removable Ballast

The MREFFP includes two types of railroad closures – (1) removable ballast and (2) embedded rails. The owner of the railroad makes the decision on the type of closure. For Phase WC-1, removal ballast will be used, which means that in the area of the railroad, the top elevation of the stem is reduced by 18 inches to allow placement of 6″ of sub-ballast topped with 12″ of ballast below the rails (Figure 7-2). In preparation for a flood, a section of the rails and ballast is removed prior to installing the stoplog closure system.

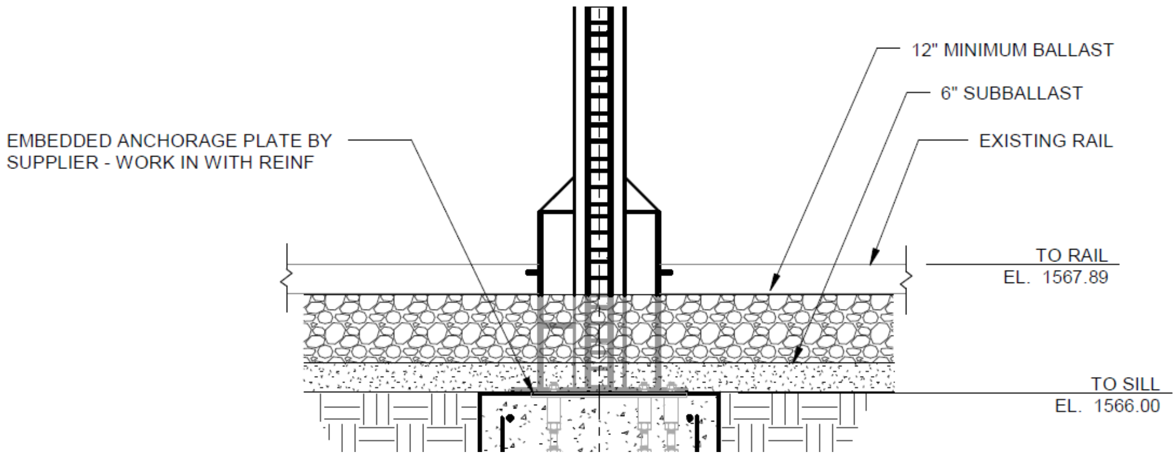


Figure 7-2 Railroad Ballast Section

7.4.1.3 Stoplog Closure

Similar to other reaches, the stoplog closure system will be performance specified. Design for the aluminum panels, connections, seals, and miscellaneous features will be submitted by the supplier based on the performance specifications. Loading criteria, fracture and fatigue are in the specifications. Closure structures are assumed to be pre-engineered, manufactured aluminum stoplog. At this time, the stoplogs and system shown in the drawings, is the same system that was installed for Phase 2 and 3. If a decision is made to install identical stoplog systems for consistency, the plans and specifications will be updated accordingly. Each stoplog will be marked with the stoplog location and Phase WC-1. .

7.4.1.4 Sheet Pile below Structure

A sheet pile cutoff was included below the structure and extends 10 feet below the structure and is embedded 6 inches into the footing. Hooks are installed through each sheet to tie the sheet pile into the concrete. Uplift pressure computations accounted for the sheet pile and assume 50percent effectiveness.

7.4.1.5 Sheet Pile and Levee Connection

To provide resiliency at the connection point between the floodwall and levee, the floodwall is extended horizontally 5 feet into the levee (in the levee profile direction) and a sheet pile is extended an additional 20 feet (beyond the floodwall). The sheet pile is extended vertically to within 1′-6″ of the levee crown elevation.

7.4.2 Design Loads and Load Cases

General load cases for design of floodwalls and closure structures are shown in Table 7-11 per the Project Design Guidelines (reference [66]) and discussed below. The maximum flood elevation considered applicable for the closure structure is the top of levee elevation. The structure has structural superiority resulting in a structure that is higher than the adjacent levee; therefore, flood loading will not reach the top of the wall without significant overtopping of the levee so this loading was not considered applicable.

Table 7-11 Applicable Load Combinations for Closure Structure

Load Case	Type
1) Construction	Unusual
2) Construction + Wind	Unusual
3) 10% AEP	Usual
4) 10% AEP + Wind	Usual
5) Design Flood Elevation	Unusual
6) Design Flood Elevation + Wind	Unusual
7) Design Flood Elevation + Ice/Debris	Unusual
8) Design Flood Elevation + Ice/Debris + Impact	Extreme
9) Minimum Grade Design	Extreme

7.4.2.1 Load Cases 1: Construction (Unusual)

The closure structure is complete with fill and a compaction loading of 250 plf is applied.

7.4.2.2 Load Cases 2: Construction + Wind (Unusual)

The closure structure is complete without fill and wind load of 50 plf is applied to the riverside.

7.4.2.3 Load Cases 3: 10percent AEP (Usual)

The closure structure is complete with fill in place. Flood loading to the 10percent AEP elevation is applied (Section 7.3.1). Because the water level at the 10percent AEP is below the bottom of footing, this load case will not control the design.

7.4.2.4 Load Cases 4: 10percent AEP + Wind (Usual)

The closure structure is complete with fill in place. Flood loading to the 10percent AEP elevation is applied (Section 7.3.1) and a wind loading of 50 psf is distributed from the lowest of either the water surface elevation or grade to the top of the wall. Because the water level at the 10percent AEP is below the bottom of footing elevation, this load case will not control the design.

7.4.2.5 Load Cases 5: Design Flood Elevation (Unusual)

The closure structure is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1).

7.4.2.6 Load Cases 6: Design Flood Elevation + Wind (Unusual)

The closure structure is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1) and a wind loading of 50 plf is distributed from the water surface elevation to the top of the wall.

7.4.2.7 Load Cases 7: Design Flood Elevation + Ice/Debris (Unusual)

The closure structure is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1) and an ice/debris loading of 0.5 kip/ft is applied at the water surface elevation.

7.4.2.8 Load Cases 8: Design Flood Elevation + Ice/Debris + Impact (Unusual)

The closure structure is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1), an ice/debris loading of 0.5 kip/ft is applied at the water surface elevation, and a 5 kip impact load is applied to the wall at eth water surface elevation. The impact load is distributed at a width equal to the stem width plus one foot plus the width gained along 45-degree angle in the vertical direction.

7.4.2.9 Load Cases 9: Minimum Grade Design (Extreme)

The closure structure is complete with fill in place. Flood loading to the minimum grade design (top of levee) is applied (Section 7.3.1).

7.4.3 Global Stability Analysis and Results Summary

The closure was analyzed as two sections: the opening section and wall section. The resulting stability factors of safety are presented in Table 7-12 and Table 7-13 for the closure and tie-in wall sections, respectively and satisfy all stability criteria required in the Project Design Guidelines (Appendix N). The land side water elevation was assumed to be at the bottom of footing. Therefore, the uplift pressure tapers from the full flood hydrostatic head on the heel side to no hydrostatic head on the toe side.

Table 7-12 Road and Rail Closure – closure section stability analysis results

Load case[1]	USACE Condition	Sliding FOS	% of Base in Compression	Floatation FOS	Heel Bearing Pressure, psf	Toe Bearing Pressure, psf
1) Construction	Unusual	N/A	100%	N/A	1,020	1,000
2) Construction + Wind	Unusual	5.8	100%	N/A	170	660
3) 10% AEP	Usual	N/A	100%	N/A	790	840
4) 10% AEP + Wind	Usual	16.6	100%	N/A	660	960
5) Design Flood Elevation	Unusual	2.4	100%	5.5	430	1,130
6) Design Flood Elevation + Wind	Unusual	2.2	100%	5.5	340	1,220
7) Design Flood Elevation + Ice/Debris	Unusual	2.1	100%	5.5	310	1,250
8) Design Flood Elevation + Ice/Debris + Impact	Extreme	1.9	100%	5.5	250	1,310
9) Minimum Grade Design	Extreme	1.2	100%	2.4	150	1,010

(1) Load cases based on Table 8-10 of the Project Design Guidelines (Appendix N)

Table 7-13 Road and Rail Closure – Tie-In Floodwall section stability analysis results

Load case[1]	USACE Condition	Sliding FOS	% of Base in Compression	Floatation FOS	Heel Bearing Pressure, psf	Toe Bearing Pressure, psf
1) Construction	Unusual	N/A	100%	N/A	1,070	1,240
2) Construction + Wind	Unusual	6.5	100%	N/A	180	740
3) 10% AEP	Usual	N/A	100%	N/A	830	1,030
4) 10% AEP + Wind	Usual	19.0	100%	N/A	700	1,160
5) Design Flood Elevation	Unusual	2.8	100%	6.3	480	1,340
6) Design Flood Elevation + Wind	Unusual	2.6	100%	6.3	390	1,430
7) Design Flood Elevation + Ice/Debris	Unusual	2.4	100%	6.3	360	1,460
8) Design Flood Elevation + Ice/Debris + Impact	Extreme	2.3	100%	6.3	300	1,530
9) Minimum Grade Design	Extreme	1.5	100%	2.7	220	1,210

(1) Load cases based on Table 8-10 of the Project Design Guidelines (Appendix N)

7.4.4 Structural Design and Analysis and Results Summary

The resulting member design results are presented in Table 7-14 and Table 7-15 for the closure and tie-in wall sections,

Table 7-14 Road and Rail Closure – Closure Section Design Capacity Values

Design Element	Calculated Maximum		Design Capacity		Utilization	
	Vu (kip)	Mu (kip-ft)	ϕVn (kip)	ϕMn (kip-ft)	Shear	Moment
Stem	6.1	24.9	45.3	131.5	0.13	0.19
Footing Heel	10.4	43.6	19.9	56.8	0.52	0.77
Footing Toe	9.1	20.8	19.9	56.8	0.46	0.37

Table 7-15 Road and Rail Closure – Tie-In Floodwall Design Capacity Values

Design Element	Calculated Maximum		Design Capacity		Utilization	
	Vu (kip)	Mu (kip-ft)	ϕVn (kip)	ϕMn (kip-ft)	Shear	Moment
Stem	6.1	24.9	16.3	46.2	0.37	0.54
Footing Heel	10.4	50.7	19.9	56.8	0.52	0.89
Footing Toe	13.1	36.8	19.9	56.8	0.66	0.65

7.4.5 Additional Design Details

7.4.5.1 Sheet Pile and Levee Connection

The sheet pile providing a connection point between the level and closure structure starts with a flat sheet embedded into the stem of the closure structure. As the sheet pile extends into the levee, one set of sheets will be connected to the flat sheet parallel with the stem followed by a sheet pile connector that will be installed to turn the sheet pile so the sheet pile follows the levee centerline. The flat sheets will be connected to the footing sheet pile using a 90 degree sheet pile connector in the footing.

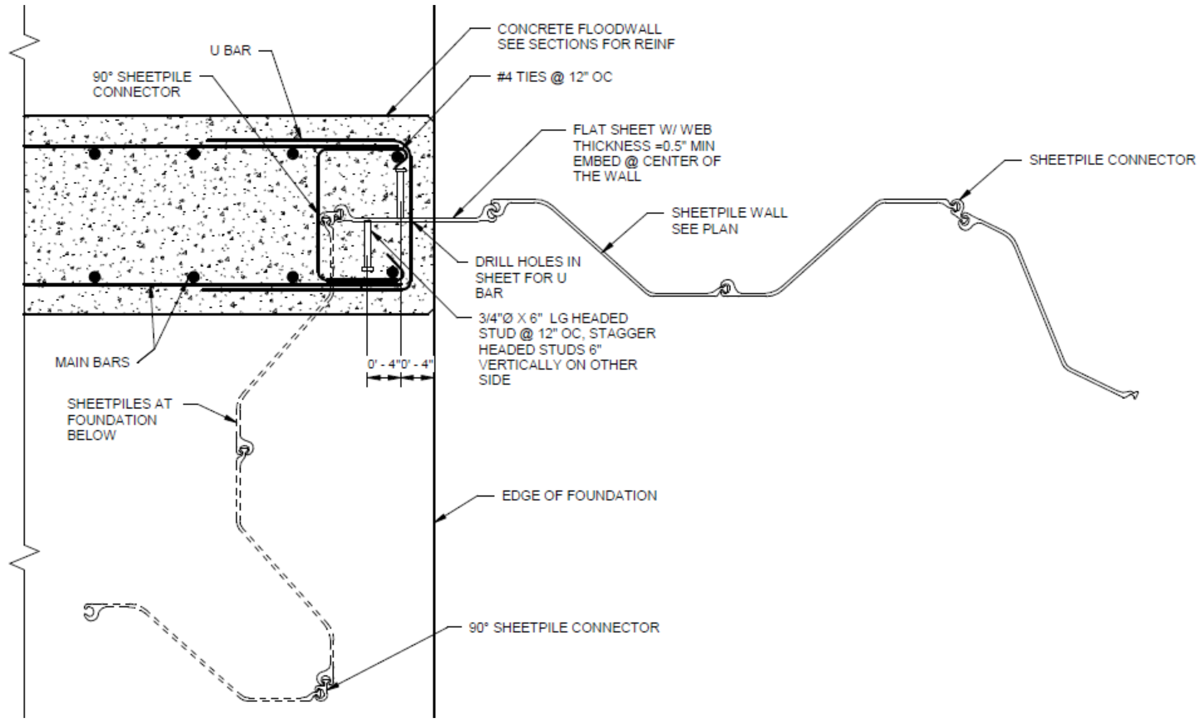


Figure 3 Sheet Pile Connection

7.4.5.2 Contraction, Expansion, Construction Joints and Waterstops

Due to the length of the closure structure, expansion joints have been added to each side of the closure section on the outside of the stoplog support piers to accommodate thermal expansion and contraction. One control joint was included on the North segment of wall to control shrinkage cracking due to its length at approximately 60 feet. Footing sand the closure sill wall do not have control joint because they are subgrade elements.

Waterstops have been included in all expansion and control joints.

7.4.6 Status

The structural design completed for the rail and road closure includes global stability, determining the wall thickness set on governing shear, and design of the primary reinforcement. Remaining tasks include optimizing the geometry, select rebar details, aesthetic detailing, and stoplog system interface design.

7.5 Tierrecita Vallejo Gatewell

The Tierrecita Vallejo Gatewell provides closure for a 36” reinforced concrete pipe (RCP) passing below the levee connecting the Mouse River with the Tierrecita Vallejo oxbow. The top of gatewell is set two feet above the levee crest at that location for structural superiority and to prevent vehicles along the levee crest from impacting the gate actuators. While only one gate is required to close shut off flow during flood events, a second gate is supplied for redundancy. Gate actuators will be manual with a socket adapter for use of a power drill.

All but the bottom segment and bottom slab are intended to be standard precast round sections. Performance specifications will require a supplier design consistent with USACE standards. The bottom segment and bottom slab have been designed and detailed as part of this design with the intent that it is fabricated by the precaster at their plant. The bottom slab was designed assuming one-way flexure spanning the inner diameter of the well.

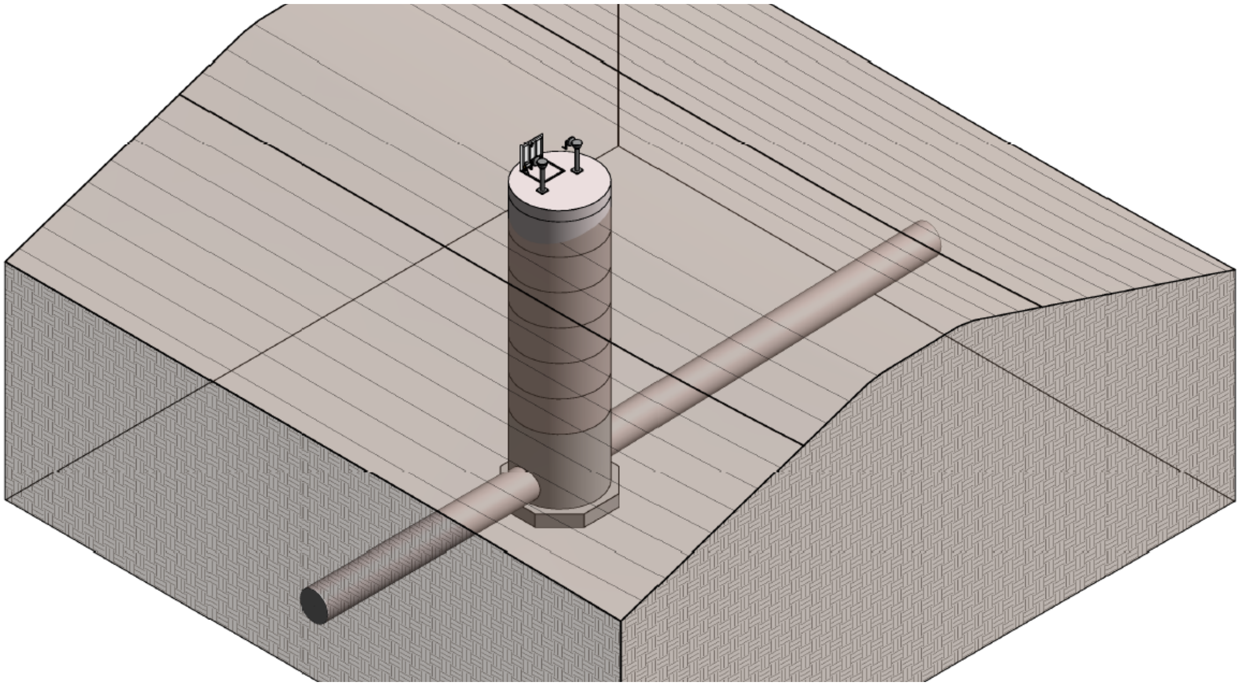


Figure 7-4 Tierrecita Vallejo Gatewell

7.5.1 Design Considerations

7.5.1.1 Precast Structure

A precast structure was selected in lieu of a cast-in place structure for the following reasons:

1) Round sections are more structurally efficient than box sections. The 34-feet deep structure would have resulted in much thicker wall sections than 9" if the cross section would have been square or rectangular resulting in significant moment.
2) The excavation for the gatewell will likely require dewatering. Assembly of precast units will be significantly quicker than cast-in-place concrete.

7.5.1.2 Gates

Stainless steel sluice gates are placed on the interior of the gatewell. The South gate is the primary gate preventing flow and the North gate serves as a redundant gate. During an event, both gates will be closed. The sluice gates are pre-engineered and will be performance specified along with the actuators and seals.

7.5.2 Design Loads and Load Cases

General load cases for design of gatewells, including the type of loading and applicability, are shown in Table 7-16 per Project Design Guidelines (reference [66]) and discussed below. The maximum flood elevation considered applicable for the closure structure is the top of levee elevation. While the structure has structural superiority resulting in a structure that is higher than the adjacent levee, flood loading will not reach the top of the wall without significant overtopping of the levee; therefore, this loading was not considered applicable.

Table 7-16 Load Combinations for Gatewell

Load Case	Type	Applicability
1) Construction	Unusual	Not Applicable
2) Construction + Wind	Unusual	Applicable
3) 10% AEP	Usual	Applicable
4) 10% AEP + Ice	Usual	Not Applicable
5) Design Flood Elevation	Unusual	Applicable
6) Design Flood Elevation + Wind	Unusual	Not Applicable
7) Design Flood Elevation + Ice/Debris	Unusual	Not Applicable
8) Design Flood Elevation + Ice/Debris + Impact	Extreme	Not Applicable
9) Minimum Grade Design	Extreme	Applicable

7.5.2.1 Load Cases 1: Construction (Unusual)

The gatewell is complete with fill and a compaction loading of 250 plf is applied. Because the gatewell is buried on all sides, stability will not be impacted by this loading. In addition, the loading will cause compression in the pre-cast gatewell. Therefore, this load case is not applicable.

7.5.2.2 Load Cases 2: Construction + Wind (Unusual)

The closure structure is complete without fill and wind load of 50 psf is applied to one side. This load case is not anticipated to control so it has not been checked for this 60percent design submittal.

7.5.2.3 Load Cases 3: 10percent AEP (Usual)

The gatewell is complete with fill in place. Flood loading to the 10percent AEP elevation is applied (Section 7.3.1) to check for floatation.

7.5.2.4 Load Cases 4: 10percent AEP + Wind (Usual)

The gatewell is complete with fill in place. Flood loading to the 10percent AEP elevation is applied (Section 7.3.1) and a wind loading of 50 psf is distributed from the water surface elevation to the top of the gatewell. Because the structure is buried, this load case is not applicable in addition to Load Case 3.

7.5.2.5 Load Cases 5: Design Flood Elevation (Unusual)

The gatewell is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1) to check for floatation.

7.5.2.6 Load Cases 6: Design Flood Elevation + Wind (Unusual)

The gatewell is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1) and a wind loading of 50 plf is distributed from the water surface elevation to the top of the wall. Because the structure is buried, this load case is not applicable in addition to Load Case 5.

7.5.2.7 Load Cases 7: Design Flood Elevation + Ice/Debris (Unusual)

The gatewell is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1) and an ice/debris loading of 0.5 kip/ft is applied at the water surface elevation. Because the water elevation is below the soil elevation, this load case is not applicable in addition to Load Case 5.

7.5.2.8 Load Cases 8: Design Flood Elevation + Ice/Debris + Impact (Unusual)

The gatewell is complete with fill in place. Flood loading to the design flood elevation is applied (Section 7.3.1), an ice/debris loading of 0.5 kip/ft is applied at the water surface elevation, and a 5 kip impact load is applied to the wall at eth water surface elevation. The impact load is distributed at a width equal to the stem width plus one foot plus the width gained along 45-degree angle in the vertical direction. Because the water elevation is below the soil elevation, this load case is not applicable in addition to Load Case 5.

7.5.2.9 Load Cases 9: Minimum Grade Design (Extreme)

The gatewell is complete with fill in place. Flood loading to the minimum grade design (top of levee) is applied (Section 7.3.1) to check for floatation.

7.5.3 Global Stability Analysis and Results Summary

The gatewell was analyzed for all applicable load cases. The gatewell was assumed to be empty and all side friction with the soil was ignored for floatation computations. All resulting stability factors of safety are presented in Table 7-17 and satisfy all stability criteria required in the Project Design Guidelines (Appendix N).

Table 7-17 Applicable Tierrecita Vallejo Gatewell Stability Analysis Results

Load case	USACE Condition	Sliding FOS	% of Base in Compression	Floatation FOS	Heel Bearing Pressure, psf	Toe Bearing Pressure, psf
2) Construction + Wind	Unusual	TBD(1)	TBD(1)	N/A	TBD(1)	TBD(1)
3) 10% AEP	Usual	N/A	N/A	2.0	N/A	N/A
5) Design Flood Elevation	Unusual	N/A	N/A	1.5	N/A	N/A
9) Minimum Grade Design	Extreme	N/A	N/A	1.4	N/A	N/A

(1) Will be computed for the 90% submittal

7.5.4 Structural Design and Analysis and Results Summary

The results of the member design for the bottom slab of the Tierrecita Vallejo Gatewell is presented in Table 7-18. The remaining sections of the gatewell will be performance specified and designed by the pre-cast manufacturer.

Table 7-18 Tierrecita Vallejo Gatewell Design Capacity Values

Design Element	Calculated Maximum		Design Capacity		Utilization	
	Vu (kip)	Mu (kip-ft)	ϕVn (kip)	ϕMn (kip-ft)	Shear	Moment
Bottom Slab	7.6	35.1	16.5	35.7	46%	43%

7.5.5 Status

The structural design completed for the gatewell includes global stability and the base slab deign. Remaining tasks include development the performance specifications for the pre-cast concrete sections and the gate pullout capacity computations.

7.6 U.S. Highway 83 Gatewell Modifications

Following completion of Phase WC-1, the U.S. Highway 83 Gatewell will be within the leveed area. The function of this gatewell will change from a closure structure to a weir. As part of this project, the slide gate will be removed and a removable steel weir will be installed. The top of weir is set to hold a water elevation of 1550.00 feet in the oxbow. It can be removed to drop the oxbow water elevation by approximately four feet.

The stoplog system will be performance specified for a design by the supplier in accordance with applicable USACE guidance.

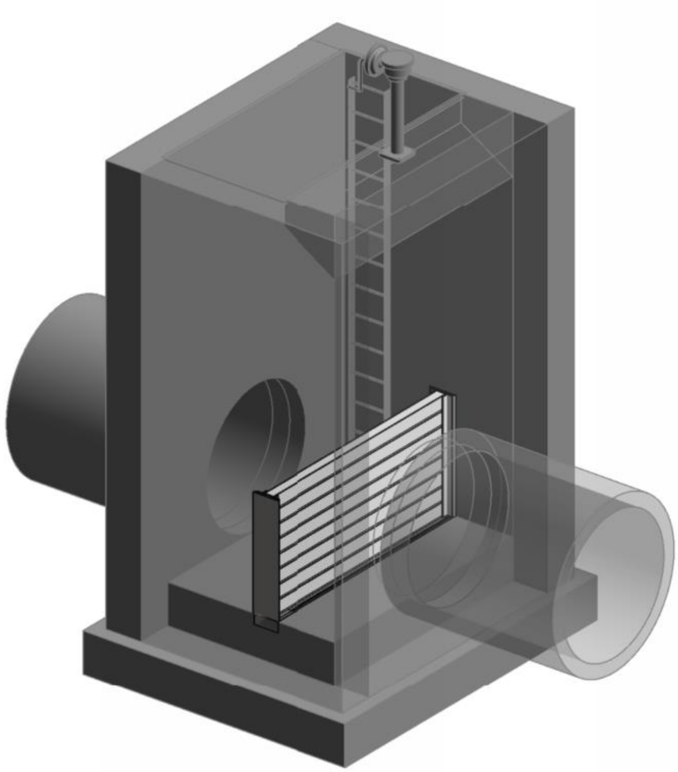


Figure 7-5 U.S. Highway 83 Gatewell

7.6.1 Design Considerations

7.6.1.1 Stoplog Weir

The stoplogs for the weir will be performance specified. Design for the aluminum panels, connections, seals, and miscellaneous features will be submitted by the supplier based on the performance specifications. Loading criteria, fracture and fatigue are in the specifications. The stop logs are assumed to be pre-engineered, manufactured aluminum stoplog will be marked with a stoplog location and Phase WC-1.

7.6.2 Design Loads

The only loading applicable to the stoplog weir is hydrostatic loading.

7.6.3 Global Stability Analysis and Results Summary

Global stability analysis of the U.S. Highway 83 Gatewell is not part of this scope because this is a retrofit to an existing structure. It is assumed that the structure is stable.

7.6.4 Structural Design and Analysis and Results Summary

Structural design of the U.S. Highway 83 Gatewell is not part of this scope because this is a retrofit to an existing structure.

7.6.5 Status

The performance specifications and drawing outlining the geometric requirements is complete and is not anticipated to require any updates for the 90% design submittal.

7.7 Materials

7.7.1 Structural Steel

All structural steel within the structural components will be per the specifications of the American Institute of Steel Construction (AISC) Manual of Steel Construction, 14th Edition (reference [67]). The minimum yield strength for structural steel is listed in Table 7-19.

Table 7-19 Structural Steel Material Properties

Structural Material	Minimum Yield Stress (ksi)	Minimum Tensile Stress (ksi)	Reference(1)
W-Shapes (ASTM A992)(2)	50	65	AISC Table 2-4
Channels (ASTM A36)(3)	36	58	AISC Table 2-4
Plates (ASTM A36)	36	58	AISC Table 2-4
Bolts (ASTM A325)(4)	N/A	105	AISC Table 2-6

(1) AISC Manual of Steel Construction, 14th Edition (reference [67])
(2) ASTM A992 – Standard Specification for Structural Steel Shapes (reference [68])
(2) ASTM A36 – Standard Specification for Carbon Structural Steel (reference [69])
(3) ASTM A325 - Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength (reference [70])

7.7.2 Reinforced Concrete

Due to high ground water levels, structures being exposed to moisture and water exposure class of concrete is set to F2, S0, P1, C1 in accordance to Chapter 4, Durability Requirements, ACI 318 (reference [71])., The minimum 28-day compressive strength for reinforced concrete in all structural components will be 4,500 pounds per square inch (psi). Concrete mix design requirements (per Chapter 4, ACI 318 (reference [71]) and ACI 350 (reference [72]) are listed in Table 7-20.

Table 7-20 Reinforced-Concrete Material Properties

Component	Designation	Reference
Exposure category and class	F2 (severe)	ACI 318, Table 4.2.1[(1)]
Maximum water-to-cement ratio	0.42	ACI 350, Table 4.2.2 [(2)]
Minimum 28-day compressive strength	4,500 psi	ACI 318, Table 4.3.1[(1)] ACI 350, Table 4.2.2[(2)]
Nominal maximum aggregate size	¾ inch	ACI 318, Table 4.4.1[(1)]
Air content	6% ± 1.5%	ACI 318, Table 4.4.1[(1)]

(1) Reference [71]
(2) Reference [72]

7.7.3 Reinforcement

All reinforcing steel will be per ASTM A615 (reference [73]) Grade 60, undeformed, uncoated. Minimum concrete clear cover is listed in Table 7-21; per EM 1110-2-2104 (*Strength Design for Reinforced-Concrete Hydraulic Structures*, reference [65]) and ACI 318 (reference [71]), this is dependent on location. Monolith length and joint spacing may dictate the requirements for more shrinkage and temperature reinforcement than the specified minimum. Table 7-22 provides minimum shrinkage and temperature reinforcement ratios for longer joint spacing of floodwalls.

Table 7-21 Minimum Concrete Clear Cover

Concrete Location	Applicable Pump Station and Gatewell Features	Minimum Clear Cover (inches)	Reference
Surfaces subject to abrasion erosion	• Top of bottom slab • Inside of exterior walls • Both sides of interior walls	6	EM 1110-2-2104, Section 2-6[(1)]
Unformed surfaces in contact with foundation	Bottom of bottom slab	4	EM 1110-2-2104, Section 2-6[(1)]
Equal to or greater than 24 inches in thickness	Exterior of exterior walls	4	EM 1110-2-2104, Section 2-6[(1)]
Greater than 12 inches and less than 24 inches in thickness	Top and bottom of top slab	3	EM 1110-2-2104, Section 2-6[(1)]
Equal to or less than 12 inches in thickness	Gatewell top slab	3	ACI 318, Section 7.7.1[(2)]

(1) *Strength Design for Reinforced-Concrete Hydraulic Structures*, reference [65]
(2) Reference [71]

Table 7-22 Minimum Temperature and Shrinkage Steel Ratios

Length between Control Joints (feet)	Minimum Temperature and Shrinkage Reinforcement Ratio (Grade 60)
Less than 30 feet	0.003
30–40 feet	0.004
Greater than 40 feet	0.005

8.0 Mechanical Design

8.1 Design Methods

Phase WC-1 does not include Mechanical Design features.

9.0 Electrical Design

9.1 Design Methods

Phase WC-1 does not include Electrical Design features.

10.0 Architectural Design

10.1 Description

Phase WC-1 does not include Architectural Design features.

11.0 Permitting and Regulatory

A number of federal, state, and local permits and/or approvals will be required prior to the start of Phase WC-1 construction. Potentially required permits/approvals are summarized below in Table 11-1.

Table 11-1 Potentially Required Permits/Approvals

Agency	Permit/Approval
Federal Permits/Approvals	
U.S. Army Corps of Engineers	Section 14 Permit (also known as Section 408 Permission)
U.S. Army Corps of Engineers	National Environmental Policy Act Compliance
U.S. Army Corps of Engineers	Section 404 Permit
U.S. Fish and Wildlife Service	Section 7 Concurrence
Federal Emergency Management Agency	Conditional Letter of Map Revision
U.S. Department of Agriculture	AD-1006 Farmland Conversion Impact Rating Form
State Permits/Approvals	
State Historical Society of North Dakota	Section 106 Concurrence
North Dakota State Water Commission	Sovereign Lands Permit
North Dakota State Water Commission	Construction Permit
North Dakota Department of Health	Section 401 Water Quality Certification
North Dakota Department of Health	Construction General Permit NDPDES
North Dakota Department of Health	Asbestos Notification of Demolition and Renovation
North Dakota Department of Health	Watermain Permits
Local Permits/Approvals	
Minot – Engineering Department	Non-building Floodplain Development Permit
Minot – Engineering Department	Utility modification approval (storm sewer and hydrant relocations)
Minot – Planning and Zoning Department	Project approval
Canadian Pacific Railway	Right of Entry Permit; Two Individual Pipe Application Permits

The NDSWC dam and levee design criteria were reviewed for applicability to the Project. While the height of levees associated with the Project and the volume of water between the levees fit within the classification of a dam, the Mouse River Enhanced Flood Protection (Project) levee system is designed to direct full Mouse River flows downstream, past the Project. As such, it does not meet North Dakota's definition of a dam. Though future Project phases may include diverting a portion of the Mouse River outside of the existing channel, temporarily impounding a portion of the river's flow, the definition of a dam would still not be met. In conversations between the Barr Engineering team and the NDSWC, the NDSWC agreed that current or future Project levees would not qualify for classification as dam systems.

12.0 Real Estate

This section describes the real estate requirements for construction and final right-of-way for modifications to Phase WC-1.

12.1 Parcel Acquisitions

The process of acquiring property needed to establish right-of-way for construction, operation, and maintenance of the flood risk management system is currently in progress. Figure 12-1 indicates the current status as of April 2019 of parcel buyouts and anticipated acquisitions.

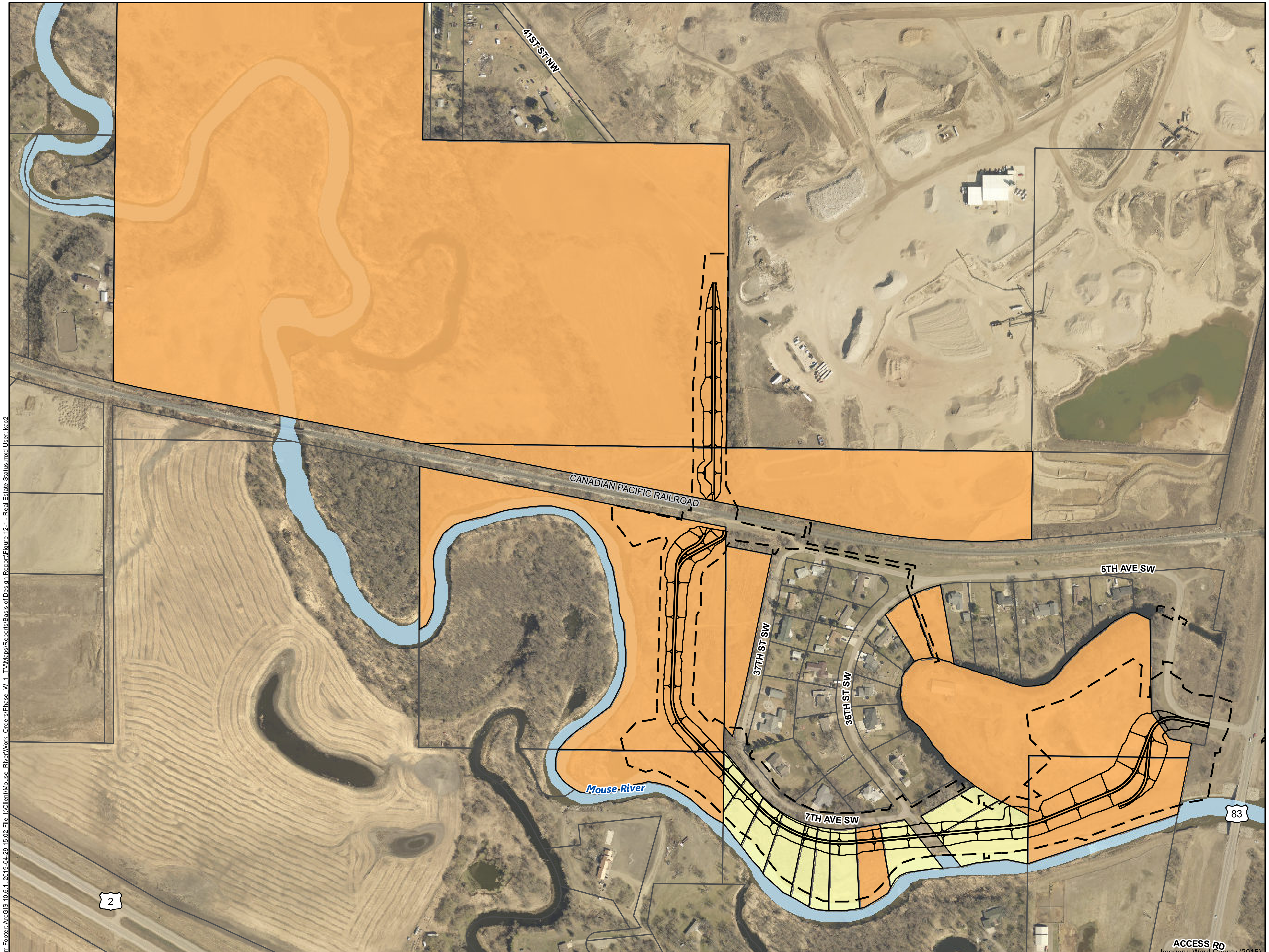
12.2 Existing Property Information

To determine legal property boundaries, property surveys will be completed through the Phase WC-1 area in June of 2019. Property corners will be recovered along the reach, and property lines and parcel boundaries will be established by North Dakota professional land surveyors in accordance with generally accepted practice and state law.

Easements for the existing federal project will be retraced by conducting deed research at the Ward County courthouse. In general, the recorded permanent easements for the existing federal project are smaller than the right-of-way indicated on the as-built plans. Temporary construction easements recorded are generally consistent with the right-of-way indicated on the as-built plans.

Property ownership data was developed using a GIS database supplied by Minot. This information shows approximate property boundaries and corresponding property owner information. Property ownership will need to be verified prior to platting any proposed easements or right-of-way.

Existing levee right-of-way information was provided by USACE in GIS format during development of the *Preliminary Engineering Report* (PER). The alignment of this right-of-way was refined by recovering survey control points from the existing federal project and retracing the boundary using the coordinates listed within the as-built plans.



Construction Limits
Levee Footprint
Purchased or Have Easement
Anticipate Full or Partial Acquisition

0 200 400 800
Feet

BARR

ACKERMAN ESTVOLD

Figure 12-1

TIERRECITA VALLEJO
REAL ESTATE STATUS

Basis of Design Report
MREFPP - Phase WC-1
Ward County, ND

The horizontal datum for the project has been established as North Dakota State Plane, North Zone, North American Datum of 1983, with U.S. survey feet as the unit of measure. This datum and unit of measure is consistent with the information currently being used by Minot, Ward County, and the USACE. This datum and unit of measure is not applicable with regard to legal property surveys; state law dictates the distances on plats to be ground versus grid (i.e., State Plane) distances. Additionally, the unit of measure for legal surveys is international feet.

Parcel and property information on the engineering drawings are shown in the project coordinate system. Parcel and property information shown on plats and legal drawings is shown in accordance with state law.

12.3 Project Right-of-Way

The proposed Project right-of-way will create a corridor with a minimum width of the levee, floodwall, and appurtenant structures, plus 15 feet on each side measured from the outer edges of the outermost critical structure. The following features are located within the proposed Project right-of-way.

- **Levee:** Real estate surrounding the levee alignment to provide access for operation and maintenance of this feature
- **River channel:** Real estate surrounding the river channel, slopes, and overbanks to provide access for operation and maintenance of these features
- **Gatewells:** Real estate surrounding gatewells to provide access for operation and maintenance of these features
- **Access road ramps:** Real estate surrounding roads which are required to access the levee for inspections or levee maintenance activities. *Access ramps solely intended to provide pedestrian access or access up and over the levee are not in the Project right of way.*
- **Interceptor / drainage ditches:** *Real estate surrounding ditches to provide for surface runoff that is not related to seepage is not in the Project right of way.*
- **Closure structure:** Real estate surrounding the roadway closure structure to provide for access and maintenance of this feature

12.4 Municipal Right-of-Way

Several city streets and utilities, along with corresponding public right-of-way, will be modified to accommodate the flood risk management system. Additional roadway right-of-way will not be required.

12.5 Permanent Utility Easements

As a part of the interior drainage modifications associated with this project, portions of storm sewer are being rerouted across private property. The acquisition of permanent utility easements is in progress in these areas to accommodate access and future maintenance and repairs.

12.6 Temporary Construction Easements

During construction, temporary construction easements are required to allow access to staging areas, borrow sites, transport of materials, and clearance for construction of structures. Temporary easements will be in effect until final acceptance of the work.

12.7 Real Estate Requirement Tabulation

The USACE Real Estate Division requires tabulation of real estate requirements for Phase WC-1 of the Project. Based on the current design configuration, the real estate requirements are presented in Table 12-1. Additional information is in the Real Estate Summary (Appendix I-1) and Real Estate Drawings (Appendix K). Further detail will be developed in the remaining design tasks. Minor revisions may be made to the alignment of features but are not expected to substantively impact the real estate requirement. The SRJB and Minot will acquire all necessary property in fee title and easements prior to construction.

Table 12-1 Real Estate Requirements for Phase WC-1

Real Estate Description	Estimated Area
Existing levee right-of-way (from USACE drawings)	7.3 acres
Existing easement in Project area to be vacated	0 acres
New permanent easement in Project area	23.9 acres
Net permanent easement in Project area	31.2 acres
Temporary construction easement in Project area	11.3 acres

13.0 Opinion of Probable Cost

This opinion of probable cost (OPC) is intended to provide information for consideration during decision-making, planning and budgeting at this 60-percent Draft submittal design stage for Phases WC-1 of the Mouse River Enhanced Flood Protection Project (Project). After preliminary alternative evaluations and alignment revisions were presented in the 2012 Preliminary Engineering Report (PER, reference [10]), further detailed design has been completed and new quantity takeoffs have been estimated for the Project. The cost estimate is of a level of detail intended to establish budget and a bid/control estimate of the Project as defined at this time.

13.1 Basis of Cost

The cost estimates and associated information in this section are intended to provide background information to understand the basis for the development of the OPC. A brief review of regional and local construction cost escalation between 2012 and 2019 indicates that construction costs for big ticket items in 2019 are similar to those used in 2012 for the PER with some increases (reference [10]). Costs are presented in 2019 dollars. Recently obtained bid tabulations for work in the region were referenced for developing unit costs.

Costs are based on analysis methodology and quantities summarized in Appendix J.

13.2 Opinion of Probable Cost Summary

The OPC is summarized in Figure 13-1 and Table 13-1.

Figure 13-1 Phases WC-1 Breakdown

Table 13-1 Point Estimate: Opinion of Probable Cost Summary

Item	OPC WC-1 Anticipated Accuracy Range, Low (-15%)	OPC Phase WC-1	OPC WC-1 Anticipated Accuracy Range, High (+25%)
Estimated Construction Cost(1)(2)(3)(4)(5)(6)	**$9.39 Million**	**$11.04 Million**	**$13.81 Million**
Lands & Easements		**Excluded**	
Planning, Engineering and Design (PED) (Assume 12%)		**Excluded**	
Construction Management (CM) (Assume 7%)		**Excluded**	
Total Opinion of Cost	**$9.39 Million**	**$11.04 Million**	**$13.81 Million**

(1) Includes 15% contingency.
(2) This feasibility-level (Class 3, 60%+ design completion per ASTM E 2516-11) cost estimate is based on detailed design alternatives, alignments, quantities, and unit prices. Costs will change with further design. Time value-of-money escalation costs are not included. The estimated accuracy range for the total Project cost, as the Project is defined, is -15% to +25%. The accuracy range is based on professional judgment considering the level of design completed, the complexity of the Project, and the uncertainties in the Project as scoped. This accuracy range is not intended to include costs for future scope changes that are not part of the Project as currently scoped or for risk contingency.
(3) Does not include temporal escalation costs, O&M costs, relocations, or betterments.
(4) Does not include acquisition of lands and easements.
(5) Does not include planning, engineering, design, or construction management.
(6) Numbers rounded to the nearest $0.01 million.

13.3 Opinion of Probable Cost Considerations

The OPC was developed based on developed designs and unit prices that are benchmarked against 2019 regional prices for similar construction scopes, and engineering judgment. This OPC is intended to correspond to a Class 3 estimate, characterized by 60-percent Draft submittal design completion (per ASTM E 2516-11, reference [74]). The OPC is based on detailed design alternatives, alignments, quantities, and unit prices. Costs will change with further design. A contingency of 10-percent for construction costs (estimated bid price) has been used based on referenced projects, published references, and the addition of project definition since previous estimates. A 5-percent contingency for construction contingency is also included to serve as budget for typical change orders. Time value-of-money escalation costs are not included. Operation and maintenance costs are not included

The Owner should consider the estimated accuracy range when allocating a budget for the Phase WC-1 construction project. The Owner must consider the Project's tolerance for the risk that actual costs exceed the allocated budget amount, and select a budget number accordingly. The OPC is a point estimate ($11.04 million) within an estimated accuracy range. The estimated accuracy range for the total project cost, as Phase WC-1 are defined, is -15-percent to +25-percent, or between $9.39 million and $13.81 million. The accuracy range is based on professional judgment considering the level of design completed, the complexity of the project, and the uncertainties in the project as scoped. This accuracy range is not intended to include costs for future scope changes that are not part of Phase WC-1 as currently scoped or risk contingency. A two-year construction duration is assumed. A detailed construction schedule is not available at this time. As design progresses, estimated costs change.

The magnitude of mechanical and electrical work on the project is not likely to require separate bids as dictated by Section 48-01.2-02 of the North Dakota Century Code. For the 60% design submittal OPC, one bid tabulation is assumed. Obtaining separate mechanical and electrical bids for the project could cause costs to be either higher or lower than a single-tabulation project bid.

The OPC is considered a construction bid estimate and has been developed on the basis of similar projects and the Barr team's experience and qualifications. The estimate represents our best judgment as experienced and qualified professionals familiar with Phase WC-1, based on Phase WC-1-related information available, current information about probable future costs, and a 60-percent Draft development of design for Phase WC-1. The OPC will change as more information becomes available and further design is completed. Given the level of project definition, uncertainty exists related to the limited design work completed to-date including, but not limited to, uncertainties associated with quantities, unit prices, and design detail. In general, it can be anticipated that as the future level of project definition increases, the uncertainty associated with these items will decrease.

Because the Barr team has no control over the eventual cost of labor, materials, equipment, or services furnished by others; the contractor's methods of determining prices; competitive bidding or market conditions; the Barr team cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from the OPC.

14.0 Drawings and Technical Specifications

Construction drawings for Phase WC-1 of the Project are in Appendix K under a separate cover. The drawings are a 60% design submittal level of completion and will be modified pending Sponsor, IEPR, and USACE comments, permitting, and agency reviews.

Technical specifications for Phase WC-1 have been developed for the 60% design submittal and are in Appendix L. These specifications have been prepared according to Construction Specifications Institute (CSI) 2004 MasterFormat guidelines, using a six-digit numbering system to organize the specifications sections. Front-end documents are based on EJCDC Document C-520 (Engineers and Joint Contract Documents Committee, Form of Agreement) and EJCDC Document C-700 (Engineers and Joint Contract Documents Committee, Standard General Conditions).

The following sources were used for technical information, guidelines, and reference specifications:

- Minot Standard Specifications and Details (2013)
- North Dakota Department of Transportation Standard Specifications for Road and Bridge Construction (2014)
- ASTM International (ASTM)
- American Water Works Association (AWWA)
- American Concrete Institute International (ACI)
- American National Standards Institute (ANSI)
- U.S. Army Corps of Engineers St. Paul District Master Specifications
- The United Facilities Guide Specifications (UFGS)
- ER 1110-1-8155 Specifications (2003)

15.0 Operation and Maintenance Manual

If necessary, an addendum to the original Operations and Maintenance (O&M) Manual will be completed as part of Phase WC-1. The manual will summarize the procedures required for operation, maintenance, repair, rehabilitation, and replacement of project features and will contain the latest approved flood risk reduction regulations, maps, drawings, tables, and references. The manual will be necessary for the project to provide ongoing benefit to Minot. The content of the manual is anticipated to include:

- Section 1.0 General Information
- Section 2.0 Ordinary Inspections, Maintenance, and Operations
- Section 3.0 Inspections, Tests, and Operations during an Impending Flood
- Section 4.0 Operations during Floods
- Section 5.0 Post-Flood: Inspections, Tests, and Operations
- Section 6.0 Post-Flood Report
- Section 7.0 Repair, Replacement, and Rehabilitation

An addendum to the O&M manual is anticipated to be in future Appendix P and will be part of the project *Construction Documentation Report,* which will be submitted to the USACE, FEMA, and Project sponsor upon completion of Phase WC-1.

16.0 Project Design Guidelines

The Project Design Guidelines represent procedures, guidelines, and formats to be used in the design of the MREFPP. It is intended to give designers general guidelines that apply consistently throughout the Project reach from Burlington through Minot with the focus on components contained in Phase WC-1. The document is not considered to be a design code, rather a living document that may be updated as design continues, and alternative or improved procedures are developed. The document will be updated as the Project moves forward to other phases.

A version 2.0 of the Project Design Guidelines is in Appendix N. This document was developed in cooperation with the Souris River Joint Water Resources Board and reviewed by the U.S. Army Corps of Engineers-St. Paul District.

17.0 QAQC

The QAQC Plan (Appendix Q) was prepared starting with the framework developed for the Preliminary Engineering Report (reference [2]) and has refinements and modifications that incorporate the lessons learned during that and subsequent efforts related to the MREFPP from the Barr/Ackerman team over the past 7 years. Important lessons learned which were used in preparation and execution of this plan involves:

- Implementing a general approach of “one doer rep” working with “one reviewer rep” to reduce the chance of conflicting directions about the path forward –with the project management team having ultimate decision rights.
- Trusting and empowering the task leads to implement the plan in a way that fits the nature of their task, and also the personalities of their team members—the QC review forms in the QAQC Plan provide a reference, but they were not considered mandatory, instead in many instances they were used as cover sheets of the actual QC reviews documented in PDFs or other electronic filing systems that captured the reviews in handwritten notes, spreadsheet calculation checks, drawing redlines, specification edits, etc. The idea behind is to have a process that is owned and managed by the staff doing the work as opposed to creating the sentiment of having another task to deal with that may not be adding value to the final product.
- Clearly defining expectations for each of the three levels of review (peer review, task lead review, and senior review).
- Engaging the senior reviewers early in the process and along the way, to increase the opportunities for getting meaningful and timely advice on the big picture (methodology, assumptions, interpretation of results, qualifications about key decisions) rather than being focused on the end products only.
- Completing the review cycle with back checks, in particular of items that were considered critical, in many cases in the form of one-to-one meetings between the doer representative and the reviewer representative.
- Giving priority to edge matching, including those necessary with HEI, not only for the sake of consistency, but primarily to produce design packages of the different Project features that correctly integrate the input from the relevant disciplines.
- Providing administrative support for complete documentation of QC reviews.

For Phase WC-1 of the Project, in addition to the internal reviews, the Barr/Ackerman team also created spreadsheets to track and address the comments received from key stakeholders, including the SRJB, Minot, the IEPR panel, USACE, and HEI. Although not in this submittal, the complete QC review documentation for this as well as for previous submittals is available in the project files, and can be made available upon request.

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