HoustonEngineering, Inc.

December 21, 2018

90% DESIGN SUBMITTAL

Mouse River Enhanced Flood Protection Project – Phase MI-5

4th Avenue NE Tieback Levee

Basis of Design Report

Prepared for the Souris River Joint Board

90% DESIGN SUBMITTAL

Mouse River Enhanced Flood Protection Project
Phase MI-5 – 4th Avenue NE Tieback Levee
Basis of Design Report

Prepared for the Souris River Joint Board

December 21, 2018
Fargo, ND

I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision, and that I am a Registered Professional Engineer under the laws of the State of North Dakota.

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XX/XX/XX

Date

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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*
APEX	*Apex Engineering Group*
ASTM	*American Society for Testing and Materials*
BARR	*BARR Engineering, Inc.*
BDR	*Basis of Design Report*
BFE	*Base Flood Elevation*
BMP	*Best Management Practice*
BRAUN	*Braun Intertec, Inc.*
CADD	*Computer-aided Drafting and Design*
cfs	*Cubic feet per second*
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*
kcf	*Kilopounds per cubic foot*
ksi	*Kilopounds per square inch*
LiDAR	*Light Detection and Ranging*
MDE	*Multiple Discrete Events*
mm/s	*millimeters per second*
MREFPP	*Mouse River Enhanced Flood Protection Project*
MWH	*MWH Global Engineering*
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 Department of Game and Fish*
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*
pcf	*Pounds per cubic foot*
PER	*Preliminary Engineer's Report*
PLC	*Programmable Logic Controller*
psi	*Pounds per square inch*
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 City of Minot storm water system to the 4th Avenue NE Pump Discharge Gatewell including all buildings, walls, pump and motor equipment, valve and meter vaults, and discharge pipes connecting to the 4th Avenue NE Pump Discharge Gatewell.*
RCP	*Reinforced Concrete Pipe*
RMSE	*Root Mean Square Error*
SCADA	*Supervisory Control and Data Acquisition*
SHPO	*State Historic Preservation Office*
SRF	*SRF Consulting Engineers*
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	*Storm Water Pollution Prevention Plan*
XP-SWMM	*XP Solutions Storm Water Management Model*
USACE	*US 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*

NOTICE TO REVIEWERS

The recently completed detailed design of the Mouse River Enhanced Flood Protection Project (MREFPP) consists of design of Phases MI-1 and MI-2/3 within the City of Minot. Each of the three phases has undergone significant USACE review as part of a comprehensive plan and 408 approval. Therefore, in an effort to provide continuity and ease for the reviewers, extra effort was taken to ensure that this Phase MI-5 report and plans are consistent in format with the reports and plans for Phases MI-1 - MI-3. As a result, much of the report has a similar layout and wherever applicable text and attachments were reprinted from the Phase MI-1 - MI-3 100% reports if the same design considerations applied to assist with ease of review. This report is supplemental to the previously published 30% and 60% Design Submittals dated June 29, 2017 and September 26, 2017.

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 MI-5, located in Minot, North Dakota. This document reflects a 90% level of design for review and comment along with submission for 408 approval and other regulatory approvals.

The MREFPP is part of an overall basin-wide effort of the SRJB to address water issues within the Mouse River Valley. In the immediate aftermath of the flood of 2011 the SRJB and the 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. The resulting product was the Preliminary Engineering Report (PER), which was adopted by the City of Minot in April 2012.

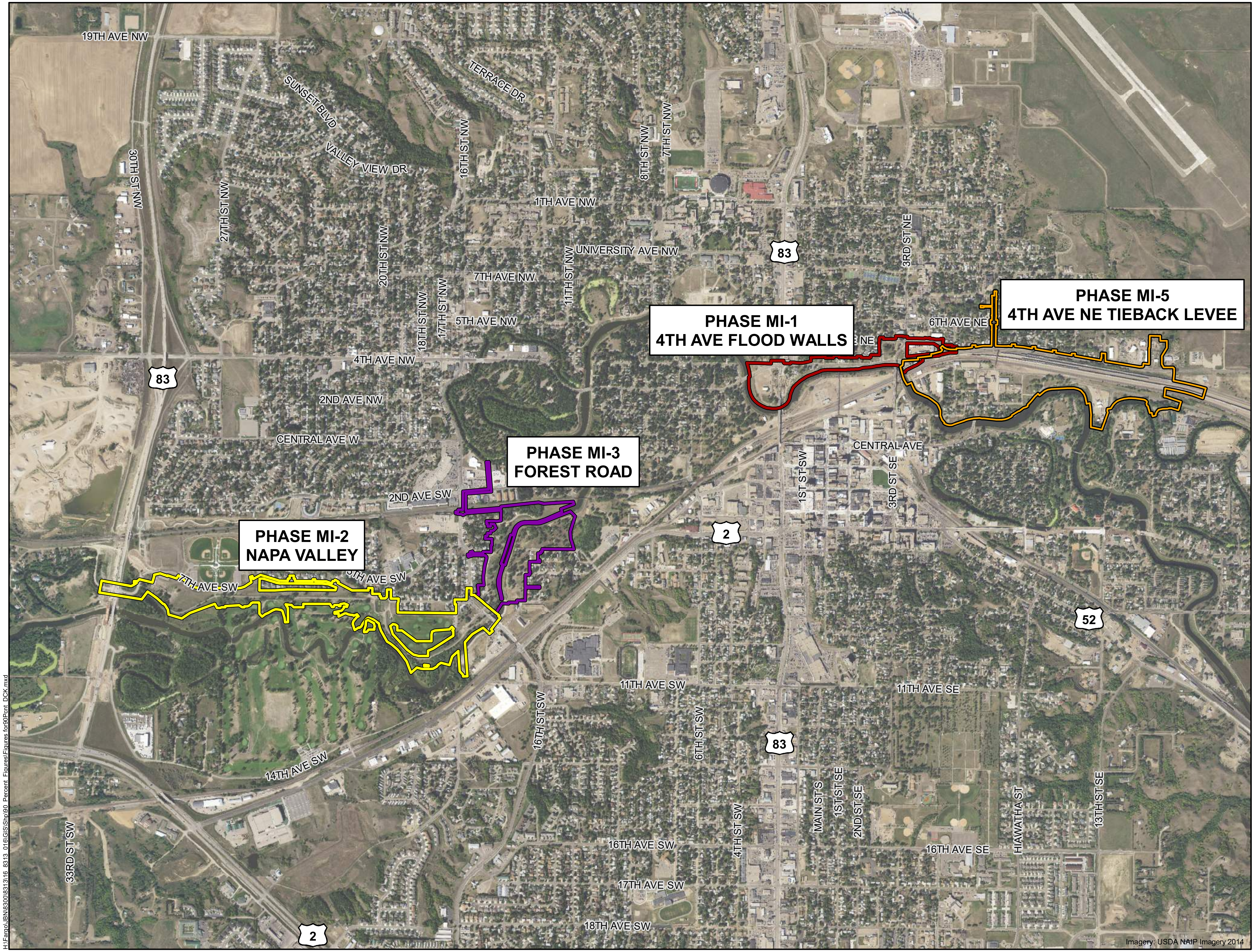
This report establishes the design basis for Phase MI-5 of the Mouse River Enhanced Flood Protection Project (MREFPP) (Project) in Minot.

Phase MI-5, also referred to as the 4th Avenue NE Tieback Levee, is located on the north side of the Mouse River and extends from the downstream end of Phase MI-1, just east of 3rd Street on the west end; to natural high ground east of 13th Street NE on the east end. Phase MI-5 of the MREFPP is being designed to work in conjunction with Phases MI-1 - MI-3. As a result, as applicable, the sections of this report were developed to be consistent with the 100% submittal developed for Phases MI-1 - MI-3.

Figure ES-1 identifies the location of Phases MI-1, MI-2/3 and MI-5. Major design features associated with Phase MI-5 of the Project are listed below.

- Approximately 5,400 feet of new levee, and 400 feet of new floodwall.
- A stoplog removable closure through the floodwall for the BNSF railroad tracks.
- A sheetpile cutoff across the BNSF railroad where the semi-permanent segment of the 4th Avenue NE Tieback Levee turns and extends north.
- Storm sewer gatewell, 4th Avenue NE Pump Station, and associated structures.
- Bank and slope stabilization at various locations within the proposed Project area.
- Storm sewer upgrades for a pipe network crossing through the line of protection at the 4th Avenue NE Pump Station within the proposed project.
- Municipal infrastructure modifications and improvements to accommodate the project, including sanitary sewer, watermain, storm sewer and street reconstruction. This includes penetrations through the line of protection that are necessary for municipal utilities.
- Franchise utility relocation.
- City greenway implementation and features, including a shared-use path system and open space.
- An interim tieback levee located south of 4th Avenue NE to maintain the existing level of flood risk management based on conditions that will exist in the interim after completion of MI-5 and until connecting future phases of the MREFPP are completed.

The USACE 2017 periodic inspection report identified sixteen deficiencies within the Phase MI-5 project reach. These work items are included in the planned system improvements within Phase MI-5.



Project Phase

- Phase MI-1
- Phase MI-2
- Phase MI-3
- Phase MI-5

0 700 1,400 2,800

Feet

Houston Engineering Inc.

Figure ES-1

PHASE MI-5
Revised for 90%
Project Location & Phasing
Basis of Design Report
Mouse River Enhanced Flood
Protection Project
Phase MI-5
Minot, North Dakota

PERTINENT DATA

Original Project Authorization

The project for local flood risk management improvements on the Souris 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 MREFPP 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 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 event the size of the 2011 flood.
- Design and construct a flood risk-reduction system for a 27,400 cfs design flood event that meets current USACE standards and FEMA requirements for accreditation.

Type of Project

This is a local flood risk management project consisting of floodwalls, levees, interior drainage facilities, a pump station, a gatewell, a removable closure structure, municipal utilities, and bank/levee erosion protection.

Hydrology and Hydraulics

Drainage Area	31,200 sq. miles
Design Flood Flow	27,400 cfs
Channel Capacity (discharge at which river banks overflow)	1,150 cfs

Principal Items of Work

Floodwall Alignment

Floodwall length	407.5 feet
Maximum Height	15 feet
Average Height	13 feet

Levee Alignment (West of Station 74+07)

Type	Compacted impervious fill
Length	Sta. 46+69 to 74+07
Design Top Elevation	27,400 cfs plus 4 feet
Side Slopes	3H:1V
Maximum Height (including freeboard)	18 feet

Average Height (including freeboard)	15 feet
Stage Uncertainty	1.5 – 1.7 feet
Superiority Overbuild	1.3 – 2.4 feet
Settlement Overbuild	1 foot
Top Width – Typical Levee	12 feet
Semi-Permanent Levee Alignment (East of Station 74+07)	
Type	Compacted impervious fill
Length	Sta. 74+07 to 85+60 and Sta. 88+36 to 95+42
Design Top Elevation	27,400 cfs plus hydraulic uncertainty for sheetpile cutoff (1558.75) and settlement (1.0 foot) for remainder of levee (1559.75)
Side Slopes	3H:1V
Maximum Height	10 feet
Average Height	8 feet
Top Width	12 feet
Sheetpile Cutoff	
Type	Sheetpile
Length	Sta. 85+17 to 88+79
Design Top Elevation	1558.75
Bottom	1530.00
Levee and Bank Erosion Protection	
Turf reinforcement mat	11,259 square yards
Riprap (Type R140)	1,586 cubic yards
Riprap (Type R270)	5,931 cubic yards
Riprap (Type R470)	575 cubic yards
Closure Structure	
BNSF Railroad Closure Structure	Sta. 45+06 – Sta. 46+69 (Stationing at Columns) Closure Length 159.5 ft

Interior Drainage Facilities

Pump Station	
Design Capacity of Station	20,000 gpm
Number of Submersible Pumps	3
Submersible Pumps Capacity	10,000 gpm/pump
Forcemain Outlet	24-inch
Sump Pump	6-inch discharge
Gatewell	
Outlet Pipe Size	8’ x 8’ box culvert
Ponding	
4th Avenue NE Detention Pond (storage volume)	11.1 acre-feet at Peak WSEL of 1545.3 (100-Yr Gravity Outfall)

USACE Inspection Work Items Corrected

	MINL_2017_a_0069-0082 and 0084-0085

Property Acquisition

Construction Temporary Easement	3.347 Acres
Fee Title	0.000 Acres
BNSF Temporary Construction Easement	15.953 Acres
Required BNSF Easements	7.155 Acres
USACE Project Permanent ROW	19.683 Acres

Project Cost Share

Federal Share	0%
Local Share	100%

1 INTRODUCTION

This report establishes the design basis for Phase MI-5 of the Mouse River Enhanced Flood Protection Project (MREFPP) (Project) in Minot. Phase MI-5, also referred to as the 4th Avenue NE Tieback Levee, is located on the north side of the river and extends from the downstream end of Phase MI-1, just east of 3rd Street NE on the west end; to natural high ground just east of 13th Street NE on the east end.

Phase MI-5 of the MREFPP is being designed to provide flood protection for the City of Minot in conjunction with other phases as a part of what is known as Construction Stage 1.5 as shown in Figure 1-1. Construction Stage 1.5 consists of several separate reaches known as Terracita Vallejo, Phase MI-2 (Napa Valley), Phase MI-3 (Forest Road), Phase MI-4 (Maple Avenue High-Flow Diversion), Phase MI-1 (4th Avenue NE) and Phase MI-5 (4th Avenue NE Tieback Levee). These phases stretch from approximately the Highway 83 Bypass in the west, through downtown Minot, to past 13th Street NE in the east. Once completed, the phases will work concurrently to provide flood protection that can be accredited by FEMA to remove the need for mandatory floodplain insurance.

Phase MI-5, similar to previously designed Phases MI-1 and MI-2/3, will impact elements of the existing federal project and will thus require Section 408 permission from the USACE prior to construction. These phases alone will not create a closed, certifiable flood risk reduction system and will be dependent on the construction of future phases before the preliminary Flood Insurance Study maps can be modified to remove homes from the 100-year regulatory floodplain. However, during the flood of 2011 significant flood fighting resources were deployed to these areas, and completion of these project segments is expected to allow Minot to focus flood fighting resources to other areas of the city should a flood occur in the interim as the MREFPP advances to future phases.

The easterly end of Phase MI-5 (east of station 74+07) is intended to provide semi-permanent flood protection by providing a tie in to high ground on the east end. This is considered semi-permanent because ultimately future phases of the MREFPP will connect in to Phase MI-5 at station 74+07 and continue full-height protection along the river on both sides. These future phases were presented in more detail in the Final Programmatic Environmental Impact Statement – MREFPP included as Appendix O1.1 of this report. As a result, beginning at station 74+07 the top of proposed protection was reduced due to the semi-permanent nature of flood protection to be provided by this segment.

1.1 PROPOSED PROJECT BACKGROUND

The MREFPP is part of an overall basin-wide effort of the SRJB to address water issues within the Mouse River valley. In the immediate aftermath of the flood of 2011, attention focused 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. Significant stakeholder involvement was solicited in obtaining the project constraints, which include, but are not limited to the following:

- Minimize property acquisitions.
- Minimize impacts to the NAWS water pipeline.
- Provide a top of protection at three feet above the 2011 level (including project induced stage increases).
- Maintain critical transportation routes during a flood similar to the 2011 flood.

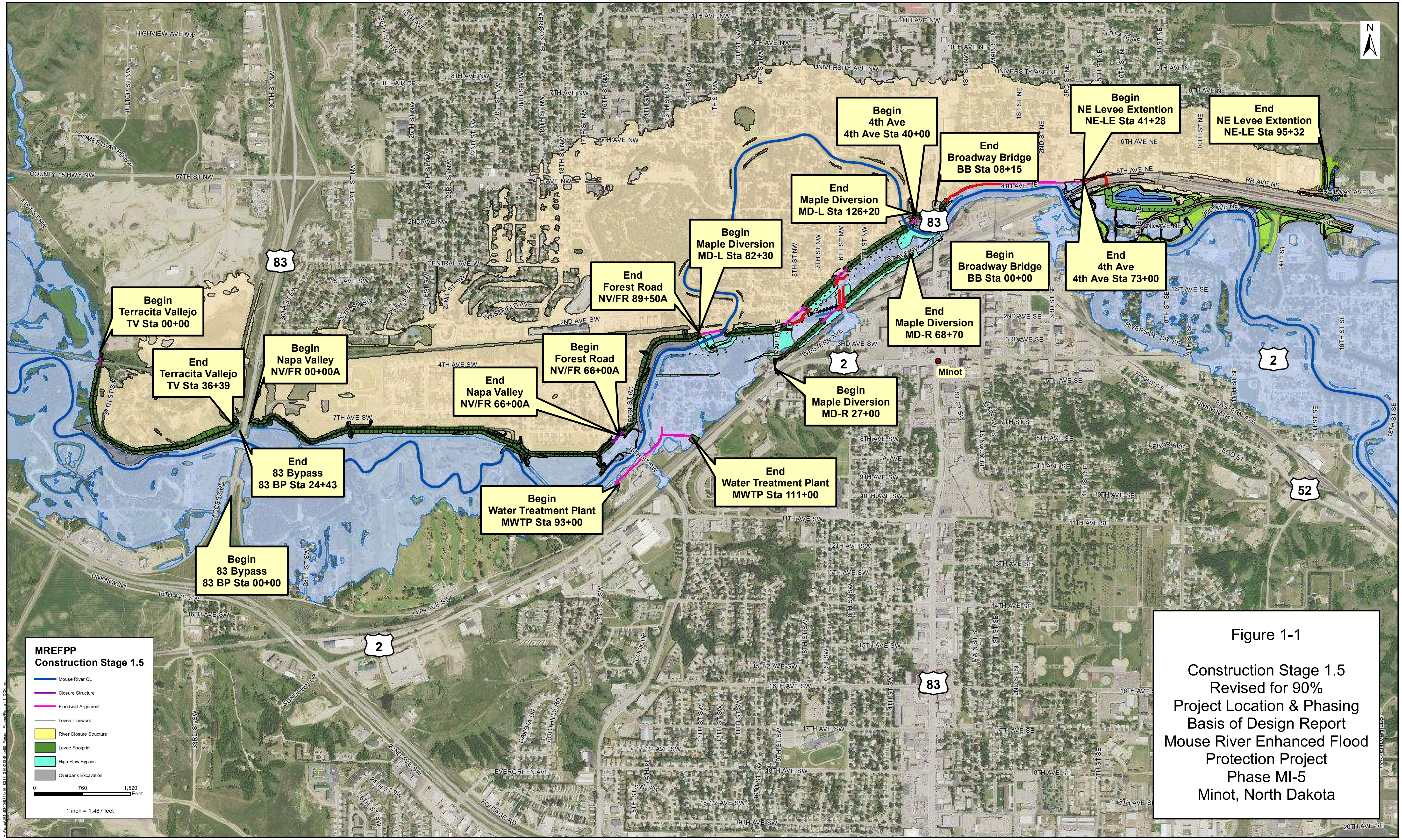


Figure 1-1

Construction Stage 1.5
Revised for 90%
Project Location & Phasing
Basis of Design Report
Mouse River Enhanced Flood
Protection Project
Phase MI-5
Minot, North Dakota

- Limit increases to observed (2011) water surface elevations at the water treatment plant to an acceptable level.
- Maintain key community resources.

After delivery of the PER, the SRJB shifted their attention to the rural reaches of the Mouse River valley, where the land use and flooding characteristics vary greatly from the developed areas.

The SRJB is the project sponsor for the MREFPP. The Project, which is being executed in multiple phases, is based on the PER and will include alignments for new levees, floodwalls, and other flood risk reduction measures. Phases MI-1, MI-2/3 and MI-5 of the Project are shown in Figure 1-2; this report will focus exclusively on Phase MI-5 (Figure 1-3). Concurrently, the SRJB is pursuing other measures to reduce the risk of flooding in the rural reaches of the valley, including implementation of the StARR program and advocating for changes in reservoir operations.

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 this massive flood, residents of the Mouse River valley requested plans for a project that could reduce the risk of flooding from events of similar magnitude. Following the flood, a study was commissioned by the NDSWC and SRJB to develop plans for new flood risk reduction features that would accommodate flows up to 27,400 cfs. The resulting *Preliminary Engineering Report (PER)* was completed on February 29, 2012 and adopted by the City of 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. The preliminary concepts presented in the PER are commonly referred to as the MREFPP.

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 MI-5. The BDR provides design detail, plans, and supporting data for Phase MI-5 features and major components. The report also contains relevant hydrology and hydraulic analyses for the Mouse River between Lake Darling and Verendrye including risk and uncertainty analysis, superiority analysis and impacts analysis. The supporting technical analysis for geotechnical, hydrologic and hydraulic, civil, structural, and pump station design are presented in appendices.

The SRJB retained the services of Houston Engineering, Inc. (HEI) 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, an array of state, local and federal permits, and allow for revisions to the FEMA FIS maps once additional future phases are constructed.

1.4 PRIOR REPORTS AND STUDIES

Efforts to address flooding problems in 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 USACE report conducted as a follow-up to the 1930 report concluded that neither reservoir storage nor local protection provided sufficient benefits to permit federal participation in flood control projects.
- 1957: A USACE examination of the Mouse River in the vicinity of Minot recommended that additional studies be conducted.
- 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.

A thorough review of the documents was conducted as part of the PER development, to gain a better understanding of the original design assumptions, subsequent system improvements, monitoring data, and current issues surrounding the system. Below is a summary of known USACE documentation for the Minot levee system:

- Design Memorandum No. 1, July 1972
- Design Memorandum No. 2, Interior Drainage, December 1973
- USACE As-Built Drawings, Channel Improvement—Reach E, August 1978
- USACE As-Built Drawings, Channel Improvement—Reach E-1, January 1979
- Operations and Maintenance Manual, November 1981
- USACE Periodic Inspection Report Number 1, Minot Flood Risk Management Project, November 13, 2013.
- USACE Annual Inspection Report, 2014, 2015, 2016 and 2017.

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.

- Flood Insurance Study, Ward County, North Dakota and Incorporated Areas, FEMA, February 15, 2002
- City of Minot FIS, Flood Insurance Study Report Data, Swenson Hagen & Company and Houston Engineering, Inc. June 3, 2002
- Mouse River Enhanced Flood Protection Plan (MREFPP): Preliminary Engineering Report, Barr Engineering Co., February 29, 2012
- Mouse River Enhanced Flood Protection Plan (MREFPP): Erosion and Sedimentation Study, Barr Engineering Co., January 18, 2013
- Mouse River Enhanced Flood Protection Plan (MREFPP): Hydrologic and Hydraulic Modeling Report, Barr Engineering Co., April 30, 2013
- Mouse River Enhanced Flood Protection Plan (MREFPP): Rural Flood Risk Reduction Alternatives Evaluation, Barr Engineering Co., May 1, 2013.
- Mouse River Enhanced Flood Protection Plan (MREFPP): Final Programmatic Environmental Impact Statement Mouse River Enhanced Flood Protection Project, US Army Corps of Engineers, July 2017.

1.5 EXISTING FLOOD RISK REDUCTION SYSTEMS BACKGROUND

Numerous federal flood control projects have been constructed in the Mouse River Valley over the last 40 years to reduce the level of flood risk 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

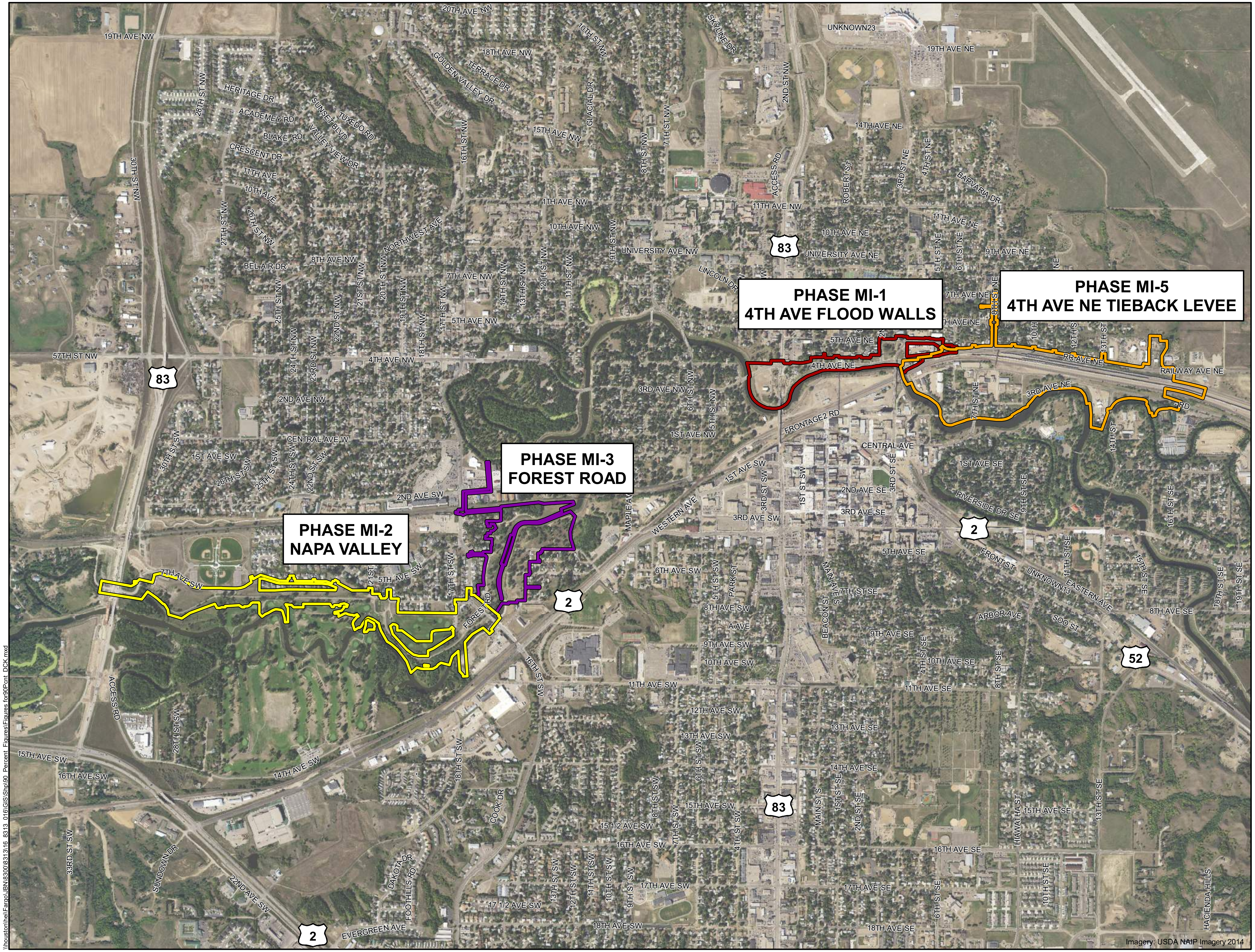
Flood risk reduction projects within the Mouse River Valley were constructed in three phases. The first phase was a channel modification project in Minot. The second phase was a levee project in Velva. The third phase involved multiple features, including:

- 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 USFWS structures in the Upper Souris and J. Clark Salyer National Wildlife Refuges
- Development of a flood warning system

Project features within Minot 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 the City of Minot. The City of Minot project features consist of the following elements:

The Souris River – Minot – Left Bank Levee System is comprised of 5.15 miles of levee, 5 closures, 4 pumping stations, 13 gravity outlets, and gatewells, and was constructed in the 1970s and 1980s. The system was constructed to protect Minot against flooding from the Souris “Mouse” River and was designed to provide 2 feet of freeboard above the project design flood of 5,000 cfs. The Left Bank Levee System has a leveed area of 1,320 acres which includes approximately 570 acres of residential land use, 80 acres of agricultural land use, 110 acres of commercial land use, and 40 acres of industrial land use, including 8 schools, 1 oil gas facility, and 2 oil gas pipelines. The system has a population at risk (PAR) of 5,701 during the day and 5,517 at night. The estimated loss of life for this system is 13 for an overtopping breach and 19 for a breach prior to overtopping. The estimated economic damages resulting from a breach are $279.4 million.

The Souris River – Minot – Right Bank Levee System is comprised of 3.21 miles of levee, 2 closures, 2 pumping stations, 5 gravity outlets, and gatewells, and was constructed in the 1970’s and 1980’s. The system was constructed to protect Minot against flooding from the Souris “Mouse” River and was designed to provide 2 feet of freeboard above the project design flood of 5,000 cfs. The Right Bank Levee System has a leveed area of 573 acres which includes approximately 265 acres of residential land use, 15 acres of commercial land use, and 60 acres of industrial land use, including 1 law enforcement facility,



Project Phase

Phase MI-1

Phase MI-2

Phase MI-3

Phase MI-5

0 700 1,400 2,800

Feet

Houston Engineering Inc.

Figure 1-2

PHASE MI-5
Revised for 90%
Project Location Map
Basis of Design Report
Mouse River Enhanced Flood
Protection Project
Phase MI-5
Minot, North Dakota

Figure 1-3

PHASE MI-5
Revised for 90%
Project Location & Phasing
Basis of Design Report
Mouse River Enhanced Flood
Protection Project
Phase MI-5
Minot, North Dakota

1 oil gas pipeline, 2 schools, and 1 state police facility. The system has a population at risk (PAR) of 1,742 during the day and 3,165 at night. The estimated loss of life for this system is 8 for an overtopping breach and 12 for a breach prior to overtopping. The estimated economic damages resulting from a breach are $132.2 million.

1.6 PHASE MI-5 DESCRIPTION

The proposed Phase MI-5 project will provide flood risk reduction features within the 4th Avenue NE area, located to the east of downtown.

Significant modifications to the existing flood protection system 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:

- Approximately 5,400 feet of new levee, and 400 feet of new floodwall.
- A stoplog removable closure through the floodwall for the BNSF railroad tracks.
- A sheetpile cutoff across the BNSF railroad where the semi-permanent segment of the 4th Avenue NE Tieback Levee turns and extends north.
- Storm sewer gatewell, 4th Avenue NE Pump Station, and associated structures.
- Bank and slope stabilization at various locations within the proposed Project area.
- Storm sewer upgrades for a pipe network crossing through the line of protection at the 4th Avenue NE Pump Station within the proposed project.
- Municipal infrastructure modifications and improvements to accommodate the project, including sanitary sewer, watermain, storm sewer and street reconstruction. This includes penetrations through the line of protection that are necessary for municipal utilities.
- Franchise utility relocation.
- City greenway implementation and features, including a shared-use path system and open space.
- An interim tieback levee located south of 4th Avenue NE to maintain the existing level of flood risk management based on conditions that will exist in the interim after completion of MI-5 and until connecting future phases of the MREFPP are completed.

The USACE 2017 periodic inspection report identified sixteen deficiencies within the Phase MI-5 project reach. These work items are included in the planned system improvements within Phase MI-5.

1.6.1 ALTERNATIVE ALIGNMENTS

Multiple alternative alignments were considered for the Phase MI-5 project reach before the selected alternative was chosen. All alternatives were developed with the same objective of providing continuous protection from the eastern extent of MREFPP Phase MI-1 to high ground east of 13th Street to at least the 100-year Mouse River stage plus freeboard required by FEMA to allow for FEMA accreditation. Alternative concepts were evaluated to assess their potential social, economic, natural resource, and cultural resource impacts. The alignment presented in this report was ultimately selected by the SRJB as the preferred alternative. Additional information on the various alternative concepts will be presented in the PEIS Addendum which shall be included in the 100% submittal.

1.7 FEATURE HEIGHT DESIGN SUMMARY

Project features from the west end to station 74+07 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.

As described previously, the easterly end of Phase MI-5 (east of station 74+07) is intended to provide semi-permanent flood protection by providing a tie-in to high ground on the east end. This is considered semi-permanent because future phases of the MREFPP will ultimately connect to Phase MI-5 at station 74+07 and continue full-height protection along the river on both sides. As a result, beginning at station 74+07, the protection was designed to reduce the risk of flooding from a flood event similar to the 2011 flood of record by incorporating risk and uncertainty analysis without system superiority, due to the semi-permanent nature of flood protection to be provided by this segment and to allow for overtopping to begin at the downstream end of the levee system once all of Construction Stage 1.5 is completed.

Figure 1-4 illustrates key design terms and elevations for levees and floodwalls. These terms are discussed further below.

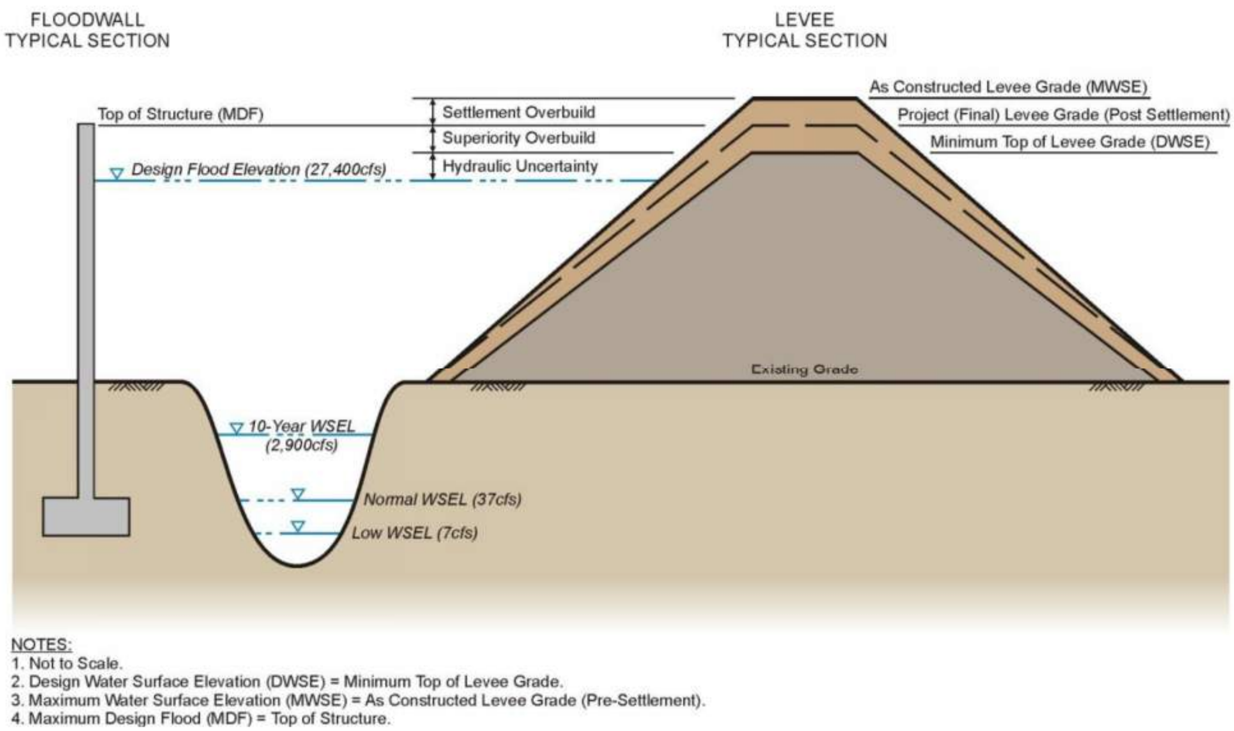


Figure 1-4 Design Elevations for Levees and Floodwalls

Definition of terms for establishing design elevations for flood protection features:

- **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.

- **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 additional height added to flood control features 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.
- **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. For Phase MI-5, superiority was excluded from the levee east of station 74+07.
- **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 amounts of settlement for the flood control systems are defined in Section 2.
- **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 for design of levees as described in Section 2.
- **Top of Structure** is the as-constructed top of a floodwall or closure structure. Floodwall design is described in Section 7.
- **10-year Water Surface Elevation** (10-year WSEL) is the water surface elevation for the 10% 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). The calculation of Normal WSEL is discussed in Section 3.
- **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). The calculation of Low WSEL is discussed in Section 3.

1.8 FORMAT AND CONTENT

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. For Phase MI-5 of the MREFPP, more 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 between Lake Darling and Verendrye, including risk and uncertainty, superiority and impact analysis.

1.9 BASE-MAP DEVELOPMENT AND PROJECT DATUM

Data used to support the design and preparation of base maps for Phase MI-5 in Project design 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 Project length. 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 [83]. 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.

1.9.1 TOPOGRAPHIC DATA AND FEATURES SURVEY

Pictometry International Corp. performed an aerial survey of the Mouse River area within Minot in June 2015. The LiDAR topographic data meets at a minimum the FEMA specifications of 12 cm vertical accuracy, and the contract between the City of Minot and Pictometry International Corp. states a root mean square error (RMSE) of 9.25 cm for bare earth vertical accuracy was the standard for the flight. This data was used to create a surface with 5-foot and 1-foot contours for the project area.

HEI completed numerous additional detailed surveys to supplement the LiDAR information and to acquire additional data specifically needed to develop the Project design.

Bathymetry surveys collected by the USACE (April 2012 through June 2013), HEI and Ackerman-Estvold were used to interpolate channel bathymetry through the city.

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

1.9.2 DIGITAL TERRAIN MODEL (DTM)

A digital terrain model (DTM) was compiled by merging LiDAR topographic data and bathymetric survey information for use in designs and drawings. ESRI's ArcMAP software was used to process all LiDAR and bathymetric survey data and to create the DTM.

1.9.3 PARCEL DATA

Minot maintains a database of parcel information in GIS format. The parcel information is approximate in nature and should not be relied upon to determine legal property boundaries, but it is suitable for basic project planning.

Property surveys have been completed through the Phase MI-5 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 shown based on GIS data provided by the USACE.

1.9.4 FRANCHISE UTILITIES

The locations of existing franchise utilities such as electric, gas, and communication have been acquired and are included as franchise utility maps. Coordination with franchise utilities continues for relocation of existing lines in conflict with the project and proposed alignments within the Project corridor. These relocations will be completed in accordance with the method set forth in Section 9 of Appendix N. The cost for relocation will be shared between the SRJB and each franchise utility owner as appropriate.

1.9.5 WETLAND DELINEATION/OHWL

Wetlands within a portion of the construction limits of the Project were identified and delineated in the field in May/June 2015 by HEI. Based on the initial 2015 review, a wetland delineation report was prepared and is provided in Appendix O2.1. Due to project alignment amendments for MI-5 since the completion of the 2015 report, a supplemental wetland delineation survey was conducted to investigate the additional areas of the project on September 14, 2017. The supplemental wetland delineation report for this additional area is included in Appendix O2.2.

Field surveys to estimate the ordinary high-water level (OHWL) along the entire reach were completed in May/June 2015 by HEI. The results of this evaluation are included in Appendix O2.1.

1.10 DESIGN APPROACH SUMMARY

Geotechnical, hydrologic and hydraulic, civil, structural, pump station, 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 included in the following appendices:

A. Appendix A – Agency Technical Review Report
B. Appendix B – Geotechnical Analysis
C. Appendix C – River Hydrology and Hydraulic Analysis
D. Appendix D – Interior Drainage Analysis
E. Appendix E – Civil Design
F. Appendix F – Structural Design
G. Appendix G – Pump Station Design
H. Appendix H – Permitting and Regulatory
I. Appendix I – Real Estate Summary
J. Appendix J – Opinion of Probable Costs
K. Appendix K – Construction Drawings (Separate Cover)
L. Appendix L – Technical Specifications
M. Appendix M – Independent External Peer Review (IEPR)
N. Appendix N – Project Design Guidelines
O. Appendix O – Environmental Studies
P. Appendix P – Operations and Maintenance Manual (Not Included – Future Section)
Q. Appendix Q – Quality Assurance and Quality Control

2 GEOTECHNICAL ANALYSIS

2.1 INTRODUCTION

To support the 90% design of the proposed project features, a subsurface investigation along with testing and geotechnical analysis was completed by Braun Intertec. A complete copy of this evaluation is included in Appendix B. The following sections provide a general overview of this evaluation.

2.2 FIELD WORK

2.2.1 DOCUMENT REVIEW, RECONNAISSANCE, AND RISK ASSESSMENT

A review of documents describing the area's underlying geologic and hydrologic setting, and historic developmental activities was first performed. A reconnaissance of the project alignment and surrounding environment was then completed to (1) confirm the status of the published information, (2) determine how present-day conditions compare to those described in the reviewed documents, and (3) identify, describe and delineate the present-day extent of visible geomorphological and developmental impacts to the river channel, banks and adjacent floodplains.

The perspective gained from this work was used to assess the project's overall and location-specific vulnerability to failure under the drivers in one or more of the following categories:

- Project Geometry
  - Structure type
  - Proximity to existing/proposed bank
  - Height and/or breadth
  - Grade changes (cuts/fills)
  - Applied loads
- Seepage, Gradient and Uplift
  - Project geometry and anticipated construction materials
  - Subsurface geologic profile and strata material properties
  - Internal erosion and piping potential
- Slope Stability
  - Project geometry
  - Foundation and bank integrity
  - Response to flood event infiltration and drawdown
- Erosion
  - Bank integrity and vulnerability to scour

The proposed alignments were then reviewed, and specific locations with unique flood protection conditions were identified and qualified relative to their vulnerability to potential failure modes (PFMs), including Internal Piping, Interface/Substratum Piping, Uplift, Erosion, Slope or Bearing Capacity Failure, Sliding, and Overtopping, which helped confirm the alignments' most critical features and stability/performance drivers, and focused the scope of exploratory, testing and analytical services.

Multiple exploration and sampling methods were then employed to broaden the understanding and enhance the interpretation of conditions as described in the following sections. A summary of the In-Situ Testing and Instrumentation testing is provided in Table 2-1.

Table 2-1 In-Situ Testing and Instrumentation Log

SPT Borings		Instrumentation	CPT Soundings			
SPT ID	Depth	VW Piezos	CPT ID	Depth	Dissipation Tests	Seismic Shear Tests
1	31'					
2	61'					
3	31'					
4	41'					
5	61'		105, 105A	60', 20'	19', 40', 42', 50'	~ 5' int. to termination
6	61'	14'	106, 106A	16', 33'	14', 27', 33'	~ 5' int. to termination
7	36'					
8	31'					
9	61'	22'	109, 109B/C/D	75', 24'/19'/61'	22', 37'	~ 5' int. to termination
10	76'	15', 50'	110	16'	15'	
11	46'					
12	31'					
13	61'					
14	31'					
15	46'					
16	76'	22', 56'	116	75'	22'	~ 5' int. to termination
17	46'					
18	31'					
19	61'	20', 55'	119	60'	14', 16', 22', 32', 45'	~ 5' int. to termination
20	41'					
21	31'					
22	76'		122	60'	17', 28', 50'	
23	41'					

SPT Borings		Instrumentation	CPT Soundings			
24	31'					
25	61'	20', 55'	125	61'	19', 23', 42', 54', 60', 61'	~ 5' int. to termination
26	31'					
27	61'					
28	31'					
29	31'					
30	46'					
31	46'					
32	61'					
33	41'					
34	46'					
35	41'					
36	41'					
37	61'					
38	41'					
39	41'					
40	61'					

2.2.2 PENETRATION TEST (SPT) BORINGS

Forty SPT borings were drilled at locations identified on the Cross Sections Overall Display aerial in Appendix B using a truck-mounted core and auger drill. Six of the borings were equipped with vibrating wire piezometers in support of an instrumentation program. Penetration tests were generally performed at 2 1/2-foot intervals through 40 feet and at 5-foot intervals thereafter, unless thin-walled tube samples were taken in lieu of penetration test samples where it appeared materials with sufficient cohesion to withstand extraction and handling for laboratory strength, hydraulic or deformation testing could be obtained. All boreholes were grouted.

2.2.3 CONE PENETRATION TEST (CPT) SOUNDINGS

To support the SPT borings and obtain more targeted in-situ material property information, 8 CPT soundings were advanced at locations also identified on the Cross Sections Overall Display aerial in

Appendix B. The soundings were performed with a dedicated 20-ton track rig as companions to eight SPT borings. The soundings were advanced to depths of approximately 16 to 75 feet. One to six dissipation tests were performed as each of the soundings were advanced, or in offset soundings after the initial soundings' data were reduced and strata of interest were targeted. Seismic shear tests were also performed at approximately 1- to 2-meter (3.3- to 6.5-foot) intervals to the termination depths of six soundings as they were advanced.

2.2.4 BOREHOLE INSTRUMENTATION AND PIEZOMETRIC DATA REDUCTION

Considering the potential impact of permeable strata in the project area on levee and floodwall performance, vibrating wire (VW) piezometers were installed in companion boreholes to six of the SPT borings at depths ranging from 14 to 56 feet to explore groundwater conditions at a range of depths and possible patterns in groundwater flow that could impact floodwall, levee, and flood protection infrastructure stability and performance. Material samples were collected from the depths at which the piezometers were installed for visual-manual classification and laboratory index testing. Additional details of these installations and results are included in Appendix B.

2.2.5 MATERIAL SAMPLE LOGGING AND REPORTING

Field logging of SPT boring samples was performed by a geotechnical engineer. The field logging was initially supervised by a senior engineer/geologist to establish consistency in the performance and reporting of visual/manual classifications, and in the procurement of thin-walled tube and bulk samples.

The field logs, and SPT and thin-walled tube samples, were submitted to supervising engineer/geologist for review and selection of material samples for laboratory testing.

2.3 SAMPLE REVIEW, LABORATORY TESTING, AND MATERIAL PROPERTY DETERMINATION

Laboratory index tests were performed to confirm field classifications. Strength, consolidation and permeability tests were performed on thin-walled tube samples of cohesive and semi-cohesive soils (clay and clay with sand and/or silt) to confirm or determine material properties for seepage and slope stability analysis. Since relatively undisturbed samples of granular soils could not be obtained, strength, permeability and deformation properties for those soils were generally estimated empirically.

SPT thin-walled tube and bulk material samples were visually classified under ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The following laboratory tests have been performed through the 90% design phase: 201 moisture content tests, 16 gradations with hydrometer, 15 gradations through a #200 sieve only, 42 #200 sieve washes, 46 Atterberg limit tests, 2 organic content tests, 10 unit weight measurements (independent of other strength and compressibility tests), 10 unconfined compressive strength tests, 8 consolidated-undrained (CU) triaxial shear tests, 1 falling head permeability test, 2 constant head permeability tests, and 2 consolidation tests.

The results of the sample review, laboratory tests, and material property determination are provided in Appendix B.

2.4 GEOTECHNICAL FINDINGS

2.4.1 GEOLOGIC AND PHYSIOGRAPHIC OVERVIEW

The MREFPP – Phase MI-5 project lies within a portion of the Mouse River valley that is approximately one mile in width from north to south. Existing surface elevations throughout the project limits vary within approximately 6 feet.

Geological Survey Water-Supply Paper 1844, titled *Geohydrology of the Souris River Valley in the Vicinity of Minot, North Dakota*, by Wayne A. Pettyjohn, indicates the project area is underlain with between 100 and 250 feet of glacial, alluvial, and river terrace deposits, respectively, over Tertiary Age rocks of the Fort Union Formation. Alluvium is the predominant overburden material as indicated in Figure 2-1.

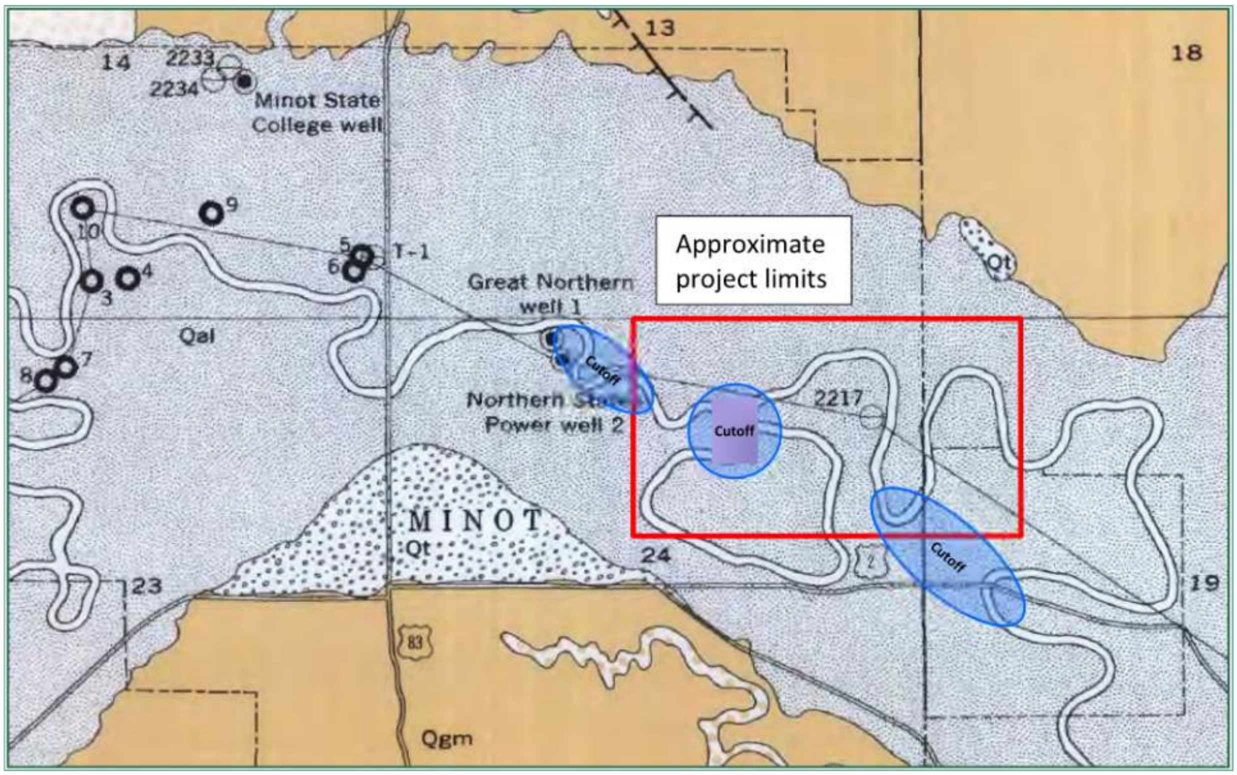


Figure 2-1 From Water-Supply Paper 1844, Plate 1, Surficial Geologic Map of Souris River Valley

Additional details of the Geologic and Physiographic are provided in Appendix B.

2.4.2 SUBSURFACE GEOLOGIC PROFILE

Water-Supply Paper 1844, Lemke's 1960 Geological Survey Professional Paper 325, *Geology of the Souris River Area, North Dakota*, and the Geologic Map of Renville and Ward Counties indicate the project is underlain mainly by Quaternary age alluvium. The SPT borings found the alluvium to be concealed at all locations explored with fill.

The existing fill classifies as primarily as cohesive clayey sand (ASTM classification SC), and lean and fat clay (CL and CH) but consists locally of cohesionless silty sand (SM) and poorly graded sand with silt (SP-SM). The existing fill was approximately 4 to 20 feet thick where encountered. It is anticipated that the fill is variable texturally and in quality (compaction, presence of organic material, debris, etc.) and may vary substantially in thickness between the exploration locations. It is also assumed the fill is prevalent along and perpendicular to the flood protection alignments.

Alluvial soils were encountered below the existing fill to the SPT borings' terminations and to the deepest CPT sounding penetration of 75 feet at CPT-116, CPT-110, and CPT-122. The alluvial soils vary in composition from poorly graded sand (ASTM classification SP) to silty and clayey sand (SM and SC), silt (ML), and lean and fat clay (CL and CH). The strata that comprise the subsurface geologic profile are not necessarily uniform in texture or are segregated by distinct boundaries. More typically the strata are comprised of a predominant soil (silt, for example) that contains lenses or layers of other soil types (clay and/or sand), or grades gradually into another soil type.

As revealed, the alluvial soils were ultimately sorted for analytical purposes into groups possessing similar permeability (primary) and textural (secondary) properties as shown below in Figure 2-2.

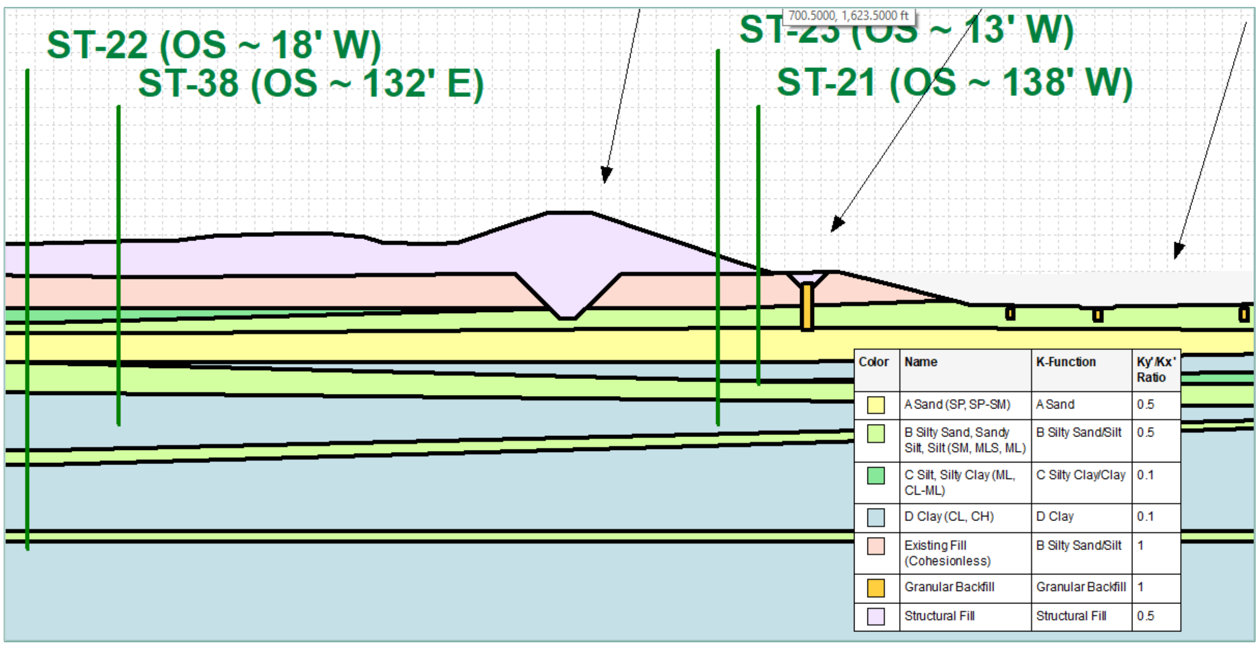


Figure 2-2 Soil Type based strata boundary assignments for Cross Section 2

Strata variability is most apparent within 20 to 30 feet of existing grades where the more granular soils, consisting of poorly graded sand (SP) and poorly graded sand with silt (SP-SM), designated as Type A soil group, and silty sand (SM), sandy silt and silt (ML), designated as Type B soil group, are more dominant. While there are finer-grained, more cohesive soils also present within this range of depths, soils classifying as clayey sand (SC) to silty clay (CL-ML), designated as Type C soil group, and lean clay (CL) or fat clay (CH), designated as Type D soil group, are typically more prevalent deeper.

From a seepage standpoint, the most permeable Type A soils are the least abundant. The Type A soils generally prevailed at depths between 15 and 25 feet, though at some locations they were also prevalent at depths between 35 and 45 feet. The Type A soils were typically overlain with more silt-rich Type B soils and, locally, finer-grained Type C and D soils, indicating an upward-fining sequence.

Groundwater conditions varied among the exploration locations with water levels observed during and/or after drilling within 5 feet and as far as 20 feet below the ground surface.

The data obtained from the VW piezometers shown in the graph of Figure 2-3, was more balanced but still reveals some complexities in groundwater conditions. Five piezometers installed at depths between 14 and 22 feet show groundwater at elevations in the range of 1536 to 1539, consistent with or within a few feet of water levels observed in open boreholes during or after drilling (suggesting the observed water levels were fairly close to having stabilized). Four piezometers installed at depths of 50 to 56 feet, in contrast, show groundwater at elevations in the range of 1512 to 1520, suggesting a lack of flow continuity between shallow and deep strata, as well as the presence of a downward gradient (supporting known municipal water extraction activities described in Water-Supply Paper 1844 – the degree of drawdown having been sufficient to precipitate a pursuit of alternative municipal water sources, the partially completed Northwest Area Water Supply Project being one example).

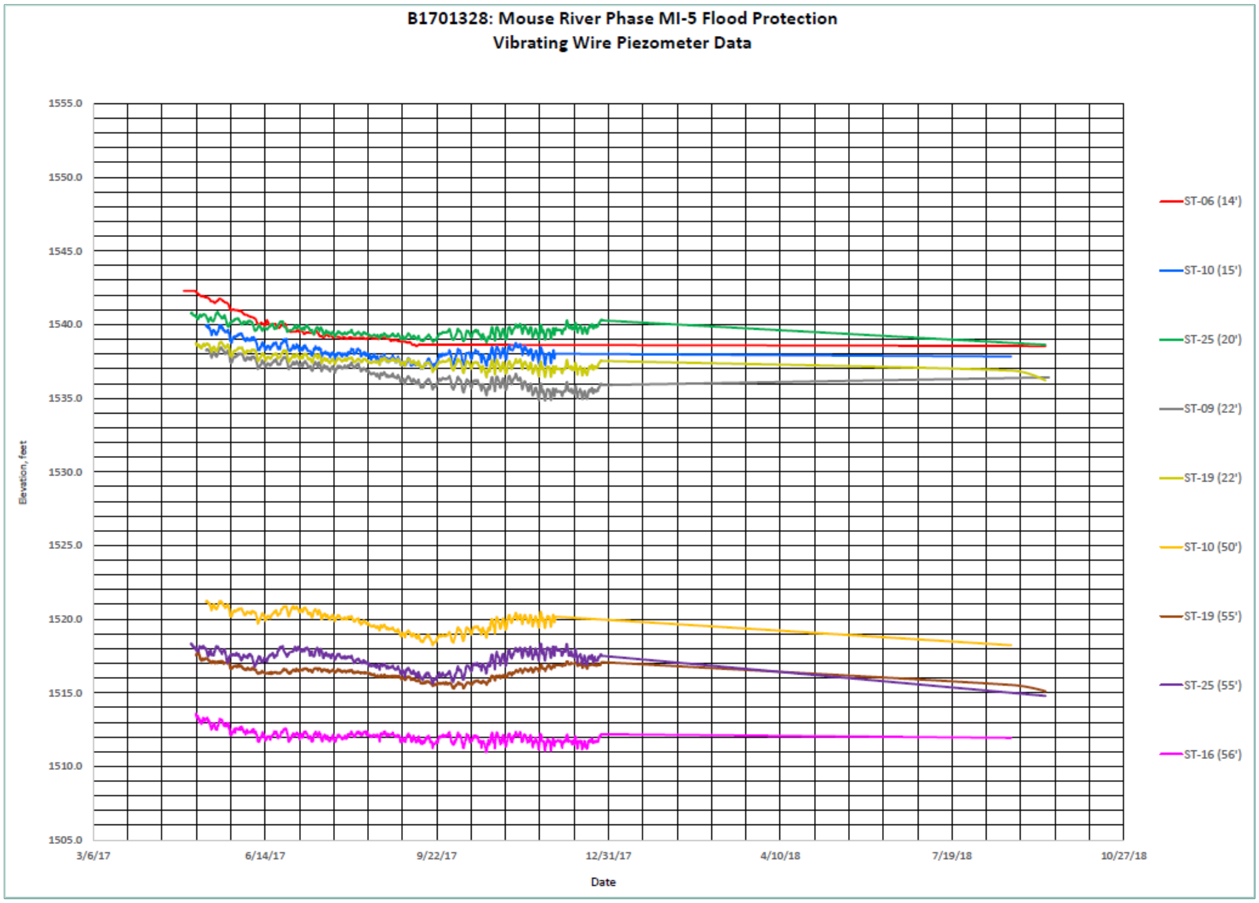


Figure 2-3 Vibrating wire piezometer data.

On the whole, the overall consistency in elevation head among the shallow and deep piezometers, respectively, supports a degree of continuity in stratigraphic conditions laterally (at least indicating that the inter-fingering of alternative soil types has established an overall relatively slow rate of transmissivity through the profile), lending confidence to the analytical modeling. The disparity in elevation head between the shallow and deep piezometers supports this as well, revealing a broad, general retardation of downward flow through the low-permeability strata between the shallow and deep piezometer sets.

Additional details are provided in Appendix B.

2.4.3 MATERIAL CHARACTERIZATION

The following sections provide an overview of the material characterization methods.

2.4.3.1 PENETRATION RESISTANCE TESTING

Penetration resistance values recorded in the alluvial soils were generally in the single digit range but crept up into the teens for all soil types, suggesting the sands and silts were generally loose but locally medium dense, while the clays were soft to medium or rather stiff. Similar values recorded in the overlying fill indicated the fill was variably compact.

2.4.3.2 CPT INFERENCES

Details of the CPT determinations related to permeability, shear strength, Young's and Bulk modulus, and friction angle are provided in Appendix B.

2.4.4 LABORATORY TESTING

Laboratory permeability tests performed on samples of the Type C and D alluvial soils generated vertical (K_v) permeability values approximately an order of magnitude slower (0.008 feet per day, for example, for a sample from Boring ST-16 between 32 and 34 feet) than their horizontal counterparts on soils presumed to be of similar composition and consistency.

While analysis of the VW piezometer and CPT dissipation test data suggested upper limit horizontal (K_h) permeability values for the Type A and B soils of approximately 0.1 and 1.0 feet per day, respectively. Laboratory constant head testing of remolded Type A samples, however, generated values one half to one order of magnitude higher, however, supporting a sensitivity study to qualify the value ultimately applied to the development, analysis and interpretation of the project's geotechnical cross sections.

Unconfined compression testing of the alluvial (Type C and D) soils generated "undrained" total stress shear strength values from 300 to over 1,500 pounds per square foot (psf). However, these results were not as consistent and are not considered as reliable as the in-situ shear strength determinations generated through CPT soundings.

CU triaxial shear strength data was used to construct graphs from which "drained" effective stress and "undrained" total stress 3-stage stress ratios were evaluated and shear strength parameters were determined. Graphs used to determine effective stress shear strength parameters were based on the construction and reduction of K_f stress ratio lines on p'-q' plots. Alluvial soils overall were assigned zero-cohesion effective stress friction angles using the 1/3:2/3 method. Fill materials (existing and future structural) were assigned best-fit effective stress phi/c parameters. The zero-cohesion p'-q' graph from the *Material Property Summaries* appendix for the Type D alluvial clays from both the Phase MI-5 and MI-1 projects reproduced below as Figure 2-4.

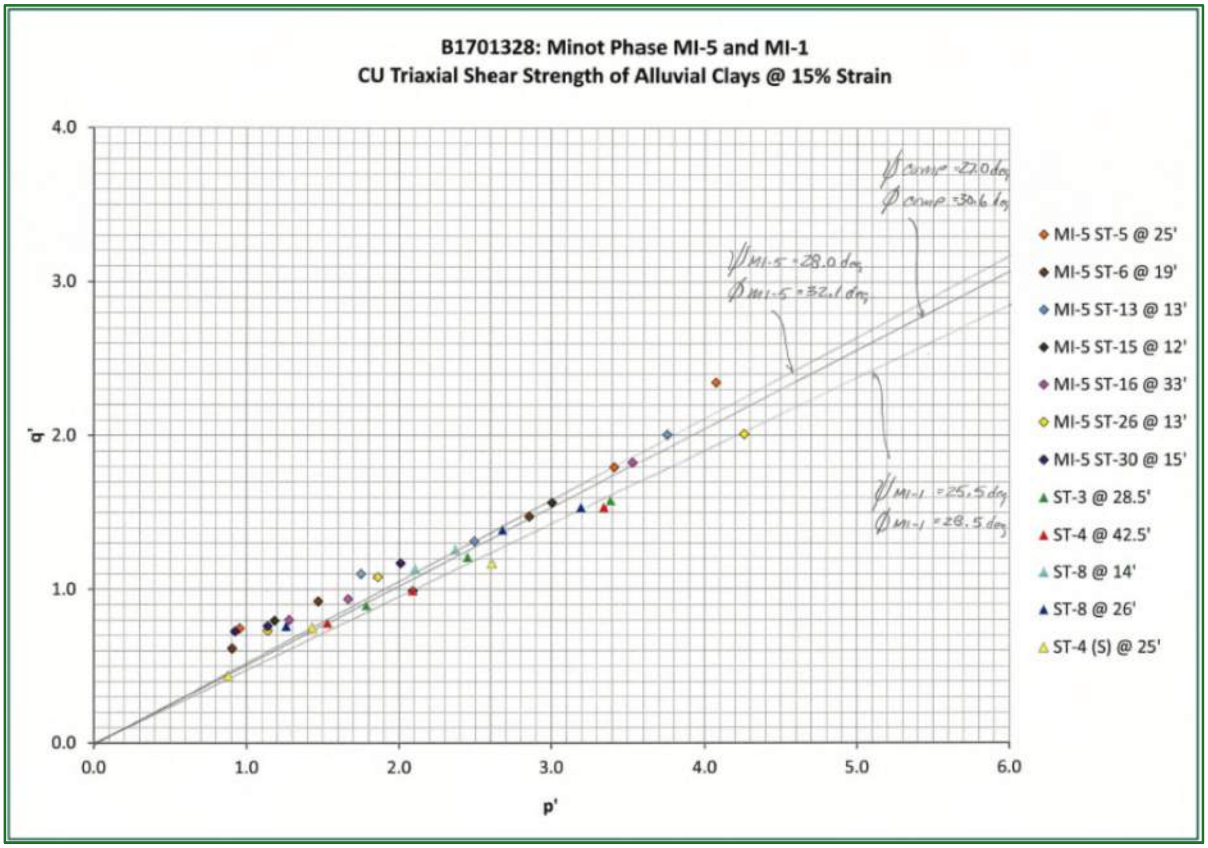


Figure 2-4 CU triaxial shear data for effective stress shear strength determination.

A sensitivity study was also performed on the Type C and D soils to confirm the appropriateness of the values applied to the geotechnical cross sections.

From consolidation tests, pre-consolidation pressures were estimated between 2.3 and 2.6 tons per square foot (4,600 and 5,200 pounds per square foot) for samples of the alluvial (Type C and D) soils taken from between approximately 32 and 40 feet, indicating the soils tested were overconsolidated, with overconsolidation (OC) ratios on the order of 2.

2.5 ANALYTICAL MODEL DEVELOPMENT

2.5.1 CROSS SECTION LOCATIONS AND GEOMETRIES

The Cross Sections Overall Display shows the locations of 22 cross sections considered for inspection and analytical modeling, as well as eight survey sections to assist in model development where location-specific channel bathymetry was not available.

Cross section location was based on a number of considerations: the results of visual reconnaissance; alignment geometry; proximity of river and dead loop channels; the presence and proximity of multiple flood protection elements; the risk assessment/PFM evaluation, and review comments provided by the US Army Corps of Engineers (USACE) and the project's independent peer review team (IEPR).

Like the location of the SPT borings, CPT soundings, and instrumentation locations, the analytical cross sections cover the length of the project alignment but are not evenly distributed along the alignment and instead were clustered in areas considered most vulnerable to distinct failure modes. Cross sections 2 through 4A traverse the storm water retention pond and areas where seepage and associated piping and uplift are a primary concern; cross sections 6 through 10 target pinch-points where levee segments converge on or pass in close proximity to existing river or dead pool banks.

Figure 2-5 shows the location of analysis cross-sections and field exploration locations.

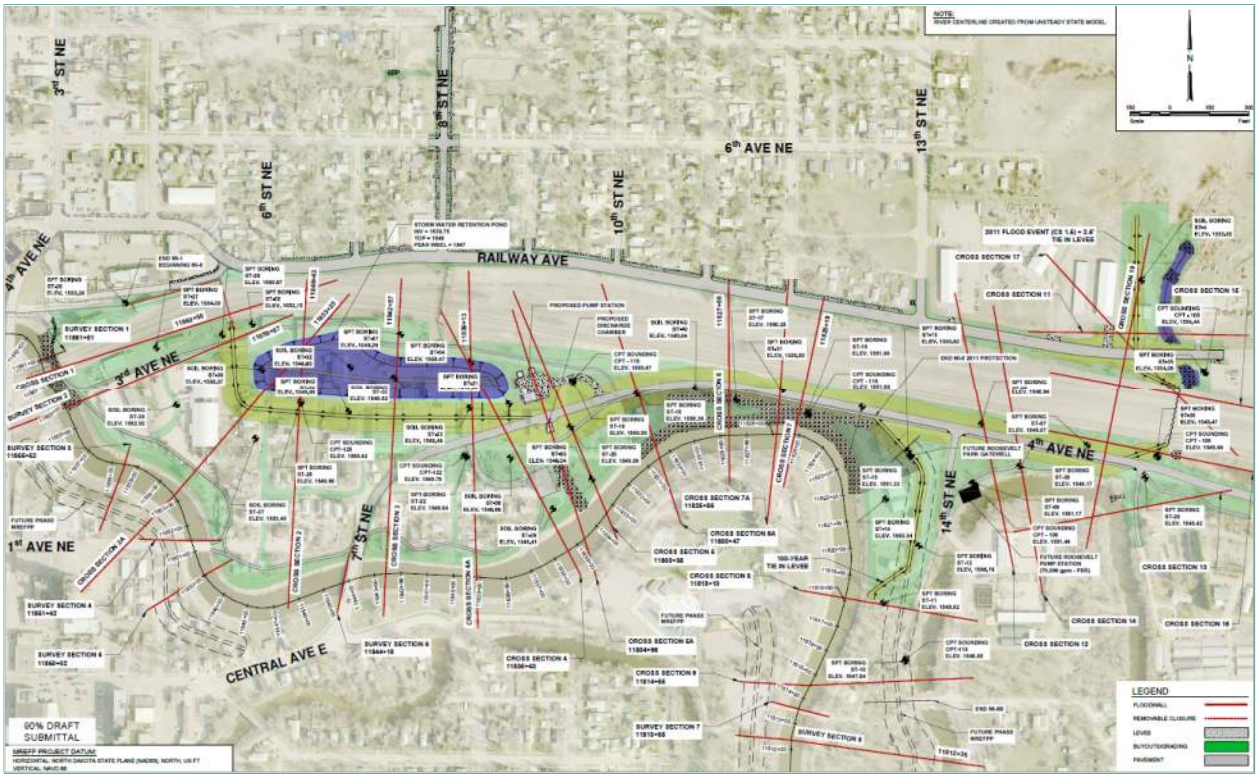


Figure 2-5 From November 6, 2018, Overall Cross Section Display by Houston Engineering, Inc.

2.5.2 GEOMETRIC BOUNDARY CONDITIONS

The geometric boundary conditions governing the analyses, essentially event-based water surface elevation, are presented in Figure 2-6 and Table 2-2. Figure 2-6 shows the design elevations that provide both definitions and visual references for the various water surface elevations.

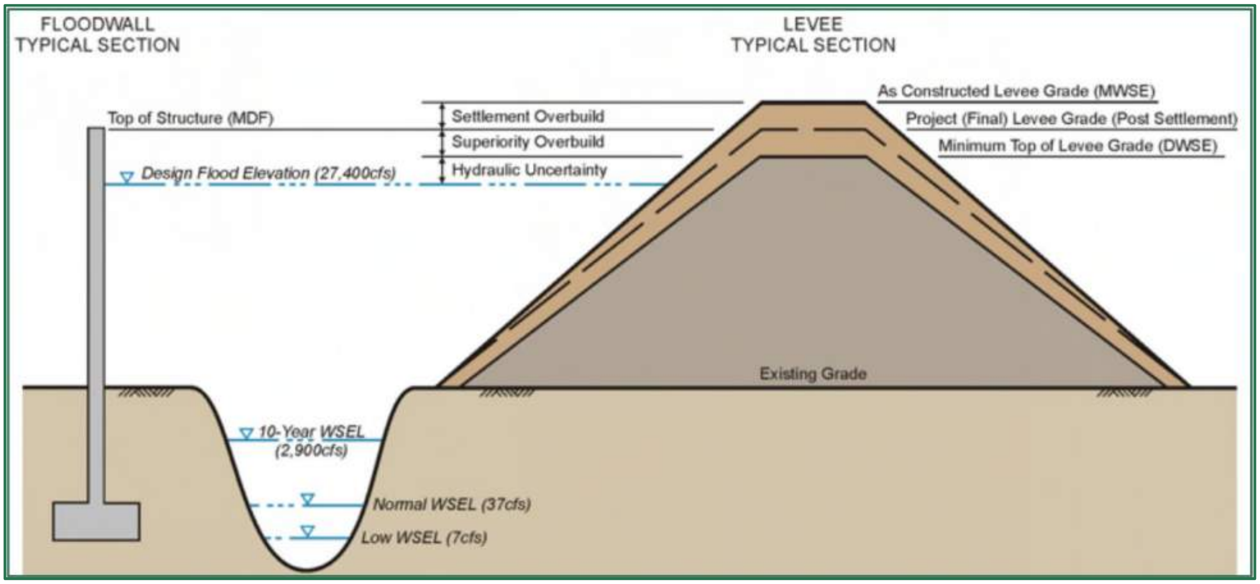


Figure 2-6 Design Elevations

Table 2-2 Analytical Section Boundary Conditions

Analytical Section Boundary Conditions

Section	Design Flood Elevation (DFE)	Hydraulic Uncertainty	Design Water Surface Elevation (DWSE)	Maximum Water Surface Elevation (MWSE, for Levees)	Maximum Design Flood (MDF, for Floodwalls)	Normal Flow Elevation (NFE - 50% Exceedance)	Low Flow Elevation (LFE)
1	1561.92	1.7	1563.62	NA - see Floodwalls	1566.50	1541.40	1541.32
2	1561.99	1.6	1563.59	1567.32	NA - see Levees	1541.39	1541.31
2A	1561.83	1.6	1563.43	1566.16	NA - see Levees	1541.39	1541.31
3	1561.81	1.5	1563.31	1566.18	NA - see Levees	1541.32	1541.23
4	1561.76	1.5	1563.26	1566.16	NA - see Levees	1539.89	1539.81
4A	1561.77	1.5	1563.27	1566.17	NA - see Levees	1539.89	1539.81
5	1561.62	1.5	1563.12	1570.94	NA - see Levees	1539.94	1539.86
5A	Cross Section 5A used only for evaluating structure- and fill-induced settlement.						
6	1561.50	1.5	1563.00	1567.15	NA - see Levees	1539.96	1539.87
6A	1561.55	1.5	1563.05	1567.00	NA - see Levees	1539.95	1539.86
7	1561.49	1.5	1562.99	1567.12	NA - see Levees	1539.93	1539.84
7A	1561.49	1.5	1562.99	1567.12	NA - see Levees	1539.93	1539.84
8	1556.24	NA - outside limits of permanent protection					
10	1557.24	NA - outside limits of permanent protection					
11	1557.24	NA - outside limits of permanent protection					
14	1557.24	NA - outside limits of permanent protection					
16	1557.24	NA - outside limits of permanent protection					
17	1557.24	NA - outside limits of permanent protection					

2.5.3 MATERIAL PROPERTY DETERMINATIONS

The physical, shear strength, hydraulic and deformation properties ultimately assigned to the materials incorporated into the analytical models are summarized in Table 2-3. Note the cell colors within the shear strength columns match those displayed by the analytical output contained in the Analytical Graphics appendices of Appendix B such that soil types can be identified in the analytical graphics and material properties thus referenced.

Table 2-3 Material Parameters

Material Parameters

	Soil Type	A[1]	B	C[2]	D[2]	Existing Fill: Cohesionless	Existing Fill: Cohesive	Structural Fill	Roadway Section	Granular Backfill	Rip Rap	Concrete	Steel
	USCS Classifications	SP, SP-SM	SM, MLS, ML (lower plasticity)	SC, ML, CL-ML (higher plasticity)	CL, CH	SM, MLS, ML	SC, CL	SC, CL	Presumed 3" Bit / 6" CL5 / 12" SP	SP, SP-SM	—	—	—
	Moist Unit Weight (γm), pcf	120	117	119	122	122	122	125	130 (net)	128	138	148	500
	Saturated Unit Weight (γsat), pcf	122	120	122	124	124	124	132	130	130	140	148	500
Effective Stress Stability	Friction Angle (Ø), deg.	32	30	29	30.6	30	28	30.6	30	34	45	—	—
	Cohesion (C), psf	0	0	0	0	0	100	232	0	0	0	"High Strength"	"High Strength"
Total Stress Stability	Friction Angle (Ø), deg.	32	30	0	0	30	0	0	30	34	45	—	—
	Cohesion (C), psf	0	0	1,000	800	0	700	2,000	0	0	0	"High Strength"	"High Strength"
Staged Drawdown Stability	Friction Angle (Ø), deg.	32	30	12.5	11.7	30	12.1	12.8	30	34	45	—	—
	Cohesion (C), psf	0	0	598	800	0	806	410	0	0	0	"High Strength"	"High Strength"
Seepage, Settlement	Horiz. Permeability (k_h), ft/day	10	0.1	0.05	0.01	0.1	0.05	0.002	0.1	10	50	—	—
	Vert. Permeability (k_v), ft/day	0.5	0.05	0.005	0.001	0.1	0.025	0.001	0.1	1	1	—	—
	Young's Modulus (E), psf	1,250,000	1,250,000	1,250,000	1,000,000	500,000	500,000	1,250,000	1,250,000	1,250,000	1,250,000	3.80E+06	4.80E+09

2.5.4 STRATA BOUNDARY ASSIGNMENTS

Strata boundaries were assigned to one of four soil types (Type A-D) as reflected in Table 2-3, and as described in Appendix B and illustrated in Figure 2-2.

2.6 SEEPAGE, SLOPE STABILITY AND DEFORMATION ANALYSES

2.6.1 ANALYTICAL METHOD

2.6.1.1 PROGRAMS

The 90% design relied mainly on the computer programs SEEP/W, SLOPE/W and SIGMA/W from the 2012 suite of GeoStudio software by Geo-Slope International to perform the seepage, slope stability and deformation analyses. SEEP/W and SIGMA/W are finite element programs that allow pore water pressure response and strain due to flooding, post-flood drawdown, and material loading or unloading to be evaluated on a steady-state (single-time step) or transient (multiple-time step) basis.

SLOPE/W was coupled with SEEP/W and SIGMA/W to compute factors of safety based on SEEP/W seepage results and SIGMA/W deformation results using limit equilibrium or, in the case of SIGMA/W, both limit equilibrium and finite element methods. SLOPE/W was also used independently to compute factors of safety for post-construction conditions using total and effective stress parameters, to compute factors of safety for pre-construction setback analyses, and to compute staged drawdown factors of safety for pre- and post-construction conditions using limit equilibrium methods. For the limit equilibrium analyses, the Spencer force and moment equilibrium numerical criteria was employed.

To support the GeoStudio analyses, a series of alternative slope stability, bearing capacity and settlement analyses were also performed as described in additional detail in Appendix B.

2.6.1.2 SEEPAGE

SEEP/W was used to evaluate DWSE, MWSE, and Design Flood Elevation-related steady state piezometric conditions. Hydraulic boundary conditions were applied only to surface grades (channel bottom, banks, proposed and adjacent grades) as shown on Figure 2-7. No boundary conditions were applied to the model bottoms and sides (assumed “no-flow” boundaries) as it was desired to not influence flow through/beneath the flood protection structures any more than necessary.

From a practical standpoint, the analytical models were only as broad and deep as needed to limit model boundary interference with “virtual” flow, and maintain confidence in the continuity of subsurface conditions away from the project alignment and exploration limits. Model limits were checked by visually inspecting flow velocities (attenuation with distance) and flow direction (no flow reversal) in SEEP/W. The dry side model surfaces were also assigned a flux line to determine seepage volumes.

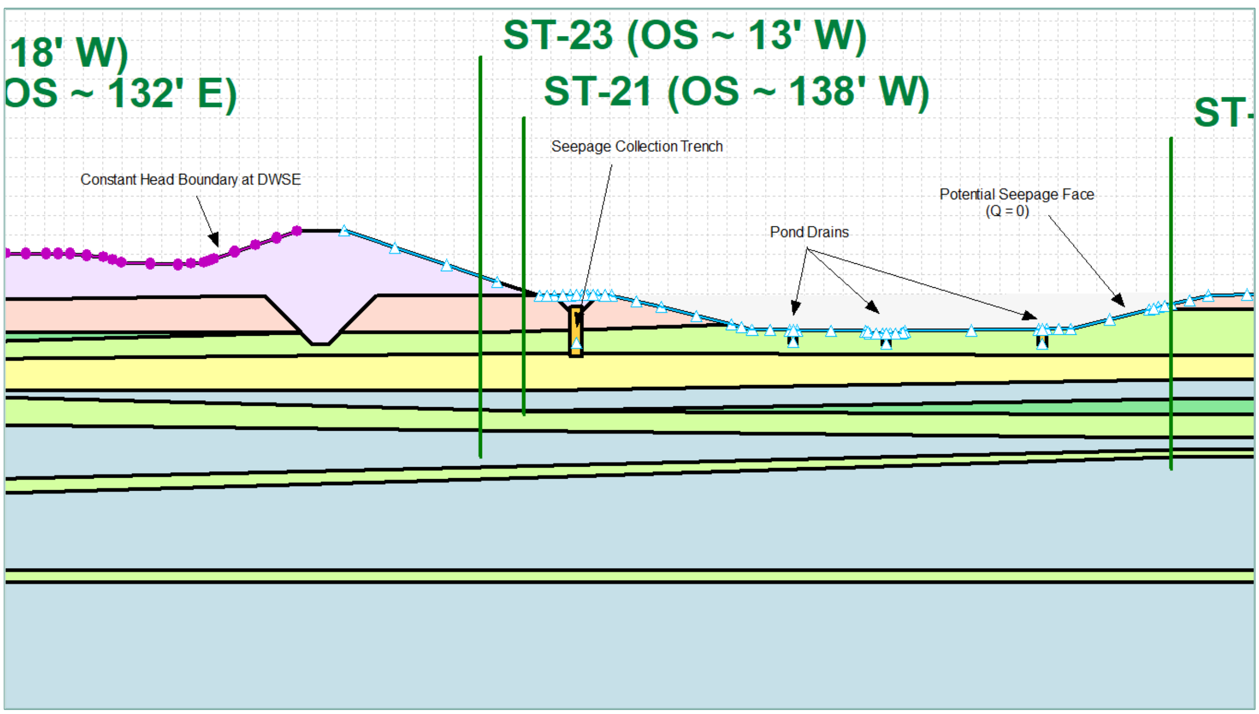


Figure 2-7 Steady State SEEP/W hydraulic boundary conditions for the levee/pond at Cross Section 2A.

Both exit gradient and boundary condition methods were used to compute factors of safety against seepage-based uplift. With the exit gradient method, where higher permeability layers were overlain with a lower permeability blanket, the critical gradient was divided by exit gradient determined from SEEP/W to determine the factor of safety. The critical gradient i_c was determined based on the average saturated unit weight of the blanket material under steady state seepage conditions [$(\gamma_{sat} - \gamma_{h20})/\gamma_{h20}$]. The exit gradient was determined by dividing the total head differential (ΔH) by the blanket thickness (L). The boundary condition method involved determining total downward and upward forces over the thickness of the blanket, and determining the factor of safety by dividing upward seepage forces into downward effective stresses. Where lower permeability blankets were absent, exit gradients were computed over the uppermost 2 to 4 feet of the profile.

Per Figure 2-8, it was determined that factors of safety at points (black arrows) at various points on the dry side of the floodwall or levee segments as gradients often increased away from the structures where surface elevations reach local lows, or where it was assumed subgrade improvements would not be made.

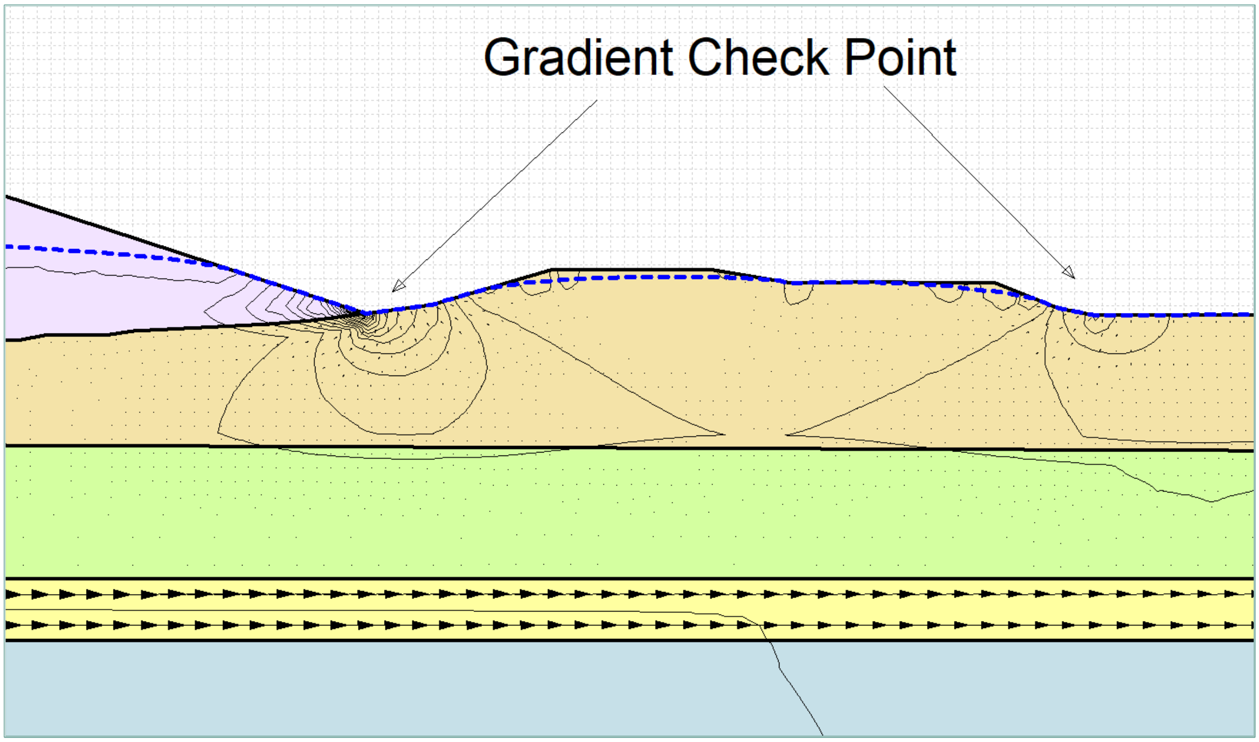


Figure 2-8 Example locations for exit gradient determination for the levee/pond at Cross Section 2A.

2.6.1.3 SLOPE STABILITY

SLOPE/W was used to evaluate the stability of existing river and dead loop banks, and the proposed or end-of-construction stability of floodwalls (wet side under passive mode) and levees (wet and dry sides). Dry side floodwall and levee stability was also evaluated under DWSE and MWSE steady state seepage conditions for structures providing protection to 2011 event elevations, or under steady state seepage at the CS 1.5 2011 WSEL elevation for structures outside the project's permanent protection limits.

Additional details are provided in Appendix B.

2.6.1.4 DEFORMATION (STRUCTURE SETTLEMENT)

SIGMA/W was one method by which structure settlement was evaluated. The SIGMA/W method is based on the use of Young's modulus values, which were computed using raw and seismic shear CPT data. Settlement was assumed due only to fill and structure loads (versus flood loads). Comparative settlement estimates were made using the program CPTe-IT (using the CPT data), and one-dimensional consolidation theory, though these latter two methods were less effective, in the opinion of the geotechnical engineer, at considering the three-dimensional aspects of the structure loads.

Additional details are included in Appendix B.

2.7 ANALYTICAL RESULTS

2.7.1 POTENTIAL FAILURE MODES AND PROJECT RISK

The vulnerability of the Phase MI-5 alignments was qualified relative to Internal Piping, Interface/Substratum Piping, Uplift, Erosion, Slope or Bearing Capacity Failure, Sliding, and Overtopping. While potentially problematic alignment sections were revealed during document review and identified and/or confirmed visually during reconnaissance work, the PFM worksheets contained in the Potential Failure Mode Worksheets section of Appendix B were used to reveal which PFMs should be considered most significant for any or all of the targeted areas.

Additional details are provided in Appendix B.

2.7.2 SEEPAGE AND STABILITY

The analysis indicates that the project complies with seepage factor of safety minimums for seepage and slope stability. The results reflect the inclusion of a seepage collection trench/relief structure along the western and southern portions of the storm water retention pond perimeter. Proposed pond bottom drains were similarly designed (collector pipes buried in filter aggregate) to provide passive support to the seepage collection structure.

The results of the analysis are presented in Appendix B.

2.7.3 SETTLEMENT

Settlement estimates suggest levee segments could settle between ½ and 1 foot, though these are considered upper limits for the range of loads anticipated. Settlements on the order of inches are expected along floodwall segments.

Similar settlements are anticipated in the area of the proposed pump station. Higher fills in this area could expose pump station structures to differential settlements on the order of ½ to 1 inch, depending on how construction is sequenced.

The results of the analysis are presented in Appendix B.

2.8 DESIGN RECOMMENDATIONS

Based on the investigations and analysis described above, Section D of Appendix B contains complete design and construction recommendations. Excerpts from those recommendations are provided here.

2.8.1 REMOVALS

Vegetation and topsoil should be removed from below levee footprints, and an exploration trench advanced beneath the centerline of the levee structures. The exploration trench should extend no less than 10 feet below prepared subgrades but through existing fill and into natural soils where fill is present. It is anticipated that maximum trench depths on the order of 10 feet could be expected.

2.8.2 BEARING CAPACITY

Based on the CPT data, floodwall foundations should be sized to exert a net allowable bearing capacity no greater than 3,000 pounds per square foot (psf). This value includes a safety factor of at least 3.0 and can be increased by 1/3 for occasional transient loads, but not for traffic loads or other live loads such as snow.

Currently it is anticipated that the pump station, gatewell and associated conduits requiring it can be designed based on a net allowable bearing capacity of 3,000 psf. This value includes a safety factor of at least 3.0 and can be increased by 1/3 for transient loads including flood water, but not for longer-term repetitive loads such as traffic, or other live loads such as snow.

2.8.3 MATERIAL PARAMETERS GOVERNING BEARING, BACKFILL AND EARTH PRESSURE

For the 90% Design submittal, the structural design of the floodwall foundation and 4th Avenue NE area pump station structures (including the gatewell, gravity outfall, storm sewer manholes and box culverts) is to be completed using the following material properties:

Table 2-4 60% Design Material Properties

Parameter	Value Above Foundation	Value Below Foundation
Unit density	132 pcf	122 pcf
Effective stress friction angle	30 deg.	30 deg.
Total stress cohesion	2,000 psf	2,000 psf
Total stress Nc	2.4	2.4

The below-foundation unit density of 122 pounds per cubic foot (pcf) was used to purposely disregard the influence of foundation subgrade improvements, it being instead assumed that unimproved alluvial soils would dominate at and below foundation bottom elevations. The effective stress friction angle (30 degrees) is a zero-cohesion value roughly equal to the average or median value for all the bearing material types likely to be present and encountered on the project. The total stress cohesion (2,000 psf) reflects the presence of structural fill immediately below the foundation but is tempered by a bearing capacity factor (Nc) reduced to reflect the presence of weaker bearing materials at depth (per NAVFAC DM 7.02, Ch. 4, Figure 5).

Additional details are provided in Appendix B.

2.8.4 SEISMIC DESIGN

Based on the magnitude of N values recorded during penetration resistance testing and the range in measured shear strengths from CPT testing and laboratory unconfined compression testing, the project falls within Seismic Site Class E according to IBC Chapter 16, Table 1613.5.2.

2.8.5 SELECTION, PLACEMENT AND COMPACTION OF BACKFILL AND FILL

General excavation backfill and additional required fill placed to levee toe elevations as well as to top-of-footing elevations along floodwalls and beneath pump station structures and other minor structures should consist of on-site or imported fill meeting the following criteria:

- No less than 35% of the particles by weight passing a #200 sieve.

- A liquid limit (LL) no higher than 50 on that portion of the material passing a #40 sieve.
- A plastic index (PI) no less than 12 on that portion of the material passing a #40 sieve.
- A liquid limit (LL) and plastic index (PI) plotting above the "A" line on a plasticity chart.

Soils classifying as clayey sand (SC) according to the Unified Soil Classification System (USCS), or soils classifying as sandy lean clay, lean clay with sand, or lean clay (CL), would meet these criteria.

Reuse of soils classifying as silt or sandy silt (ML) is not recommended.

Prior to compaction, backfill should be moisture conditioned to moisture contents within one percentage point below to three percentage points above their optimum moisture contents.

Backfill should be spread in loose lifts 6 to 12 inches thick prior to compaction – the lesser value being considered more appropriate for finer-grained and higher plasticity soils, and the greater value permissible for coarser-grained and lower plasticity soils. Ultimate lift thicknesses will depend on construction conditions including temperature, material moisture, and the equipment used to spread and compact the material, and may require adjustment based on initial compaction test results.

Backfill should be compacted to at least 95% of the material's maximum standard Proctor dry density.

2.8.6 SEEPAGE COLLECTION

A seepage collection trench/relief structure was required to mitigate high Y exit gradients and uplift forces present near the pond slope due to the blanket condition there. The location of the proposed trench is shown in both Figure 2-2 and also in Figure 2-9 below.

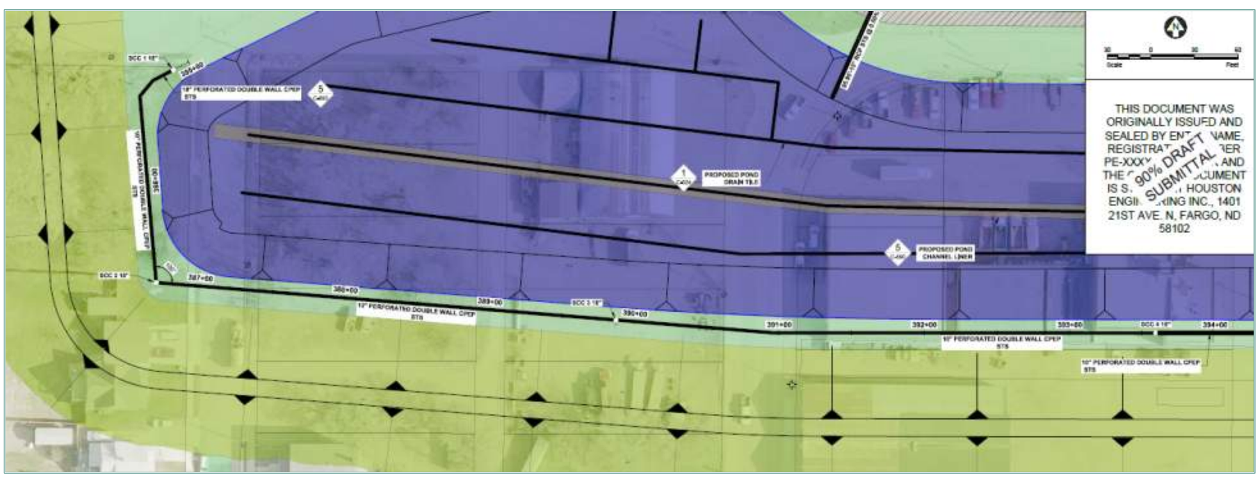


Figure 2-9 From November 30, 2018, Seepage Collection Plan & Profile, by Houston Engineering, Inc.

Extending from the bench between the storm water retention pond and general levee slopes excavations for the storm water retention pond's seepage collection trench will be on the order of 20 feet deep and will likely need to be shored to avoid broader, sloped excavation banks. It is anticipated the trench will be deepest at its western end, sloping up from a nominal bottom elevation of 1529 to perhaps 1536 at its eastern terminus. A profile identifying the bottom of the fine filter aggregate will be added to the seepage collection system construction drawings in Appendix K prior to the 100% submittal. Regardless of depth, it is anticipated the trench alignment will need to be dewatered, likely in advance of excavation.

Additional details of the seepage collection system are provided in Appendix B.

3 RIVER HYDROLOGY AND HYDRAULIC ANALYSIS

3.1 OVERVIEW

The Phase MI-5 - 4th Avenue NE Tieback Levee segment is a portion of the larger Mouse River Enhanced Flood Protection Project (Project). Much of the hydrology and hydraulic information provided in this report is repeated information provided in the 100% design submittal for the Phase MI-1 and MI-2/3 (4th Avenue NE, Napa Valley and Forest Road) segments of the project. Additional details of these hydrologic and hydraulic model results are presented below and in greater detail in Appendix C2.1. Appendix C2.1 was developed and included in the 100% design submittal for Phases MI-1 and MI-2/3, however, it was intended to provide hydrologic and hydraulics data for the design of all phases of the MREFPP between Burlington and the downstream end of Minot.

Since the time of developing Appendix C2.1, the project has undergone Independent External Peer Review. As part of that review, supplemental information was developed and included in the design report for the Phase MI-2 and MI-3 (Napa Valley and Forest Road) segments of the project. For clarity, this same information was reprinted in a memorandum and is expected to serve as a supplement to the Appendix C2.1 dated July 2016. A copy of this supplemental information is included in Appendix C3.

After completion of the hydrologic and hydraulic design for Phases MI-1 and MI-2/3, the SRJB chose to move forward with a revision to the current plan that would take advantage of additional property acquisitions being completed by the City of Minot using funding from the National Disaster Resilience Competition (NDRC) Program. The NDRC is a competitive grant program administered by the U.S. Department of Housing and Urban Development (HUD). In January 2016, Minot was awarded $74,340,770 in NDRC funding from the National Disaster Resilience Community Development Block Grant (CDBG-NDR) to achieve the goals as prescribed in the City's application. During the scoping process for the NDRC application, a number of sites were identified and ultimately screened out of consideration by the City. The City has decided to move forward with additional acquisitions outside of the project footprint identified in the DEIS at three sites – the Moose Lodge neighborhood, the Leite-Brekke neighborhood, and the Kemper neighborhood (Figure 3-1). As part of the changes a revised modeling effort was completed, and the results presented in Appendix C2.2.

The results in Appendix C2.1 and C2.2 build on previous hydrologic and hydraulic studies for the Project, more specifically the 2012 Preliminary Engineering Report (PER) in 2012 and for the Mouse River Enhanced Flood Protection Plan - Hydrologic and Hydraulic Modeling Report in 2013 (2013 H&H Report). Hydraulic models from the 2013 H&H Report were since updated to support the design and permitting of the overall Project and specifically the Project Phases MI-1-3 and MI-5. The design alignment as proposed in this 90% Basis of Design Report deviates from what was shown for Construction Stage 1.5 in Appendices C2.1 and C2.2. As a result, a preliminary supplemental evaluation was completed to determine expected changes in hydraulics and is included in Appendix C2.3.

The following sections describe this in more detail.

3.2 HYDROLOGIC ANALYSIS

Hydrologic modeling described in more detail in Appendix C2 was used to develop flow data needed in the hydraulic models. Hydrographs recorded at USGS gaging stations, or gaged inflows, were used to define inflow hydrographs of historic floods and to generate synthetic hydrographs to represent flood peaks of varying magnitudes.

NDRC Application Neighborhoods

Municipalities

0 750 1,500 3,000

Feet

Figure 3-1

NDRC APPLICATION NEIGHBORHOOD OVERVIEW
Mouse River Enhanced Flood Protection Project - EIS Addendum

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. No updates were made to Mouse River Basin HEC-HMS model for the results in Appendix C2.

The goal of the hydrologic analysis was to develop regulated, balanced hydrographs at the Foxholm, Minot, and Verendrye gaging stations and coincidental hydrographs at the inflow locations between the USGS gaging stations. The methodology used was based on the guidance of the U.S. Army Corps of Engineers (USACE) publication *Hydrologic Engineering Methods for Water Resources Development: Hypothetical Floods, Volume 5*. Balanced hydrographs were developed to simulate intermediate flood peaks not represented by the 2009, 2010, and 2011 historic flood hydrographs.

Additional details of the hydrologic methods are included in Appendix C2.

3.3 HYDRAULIC MODEL DEVELOPMENT

Hydraulic modeling presented in Appendix C2 was used to establish hydraulic design parameters for the MREFP Project reach from Burlington through Minot.

The hydraulic modeling described in Appendix C2 builds on the modeling completed for the 2013 H&H Report. The following sections provide a general overview of the hydraulic modeling. A more detailed description is included in Appendix C2.

3.3.1 EXISTING-CONDITIONS HYDRAULIC MODELING

Hydraulic modeling presented in Appendix C2 was performed using the unsteady flow routine in HEC-RAS. The foundation for all the hydraulic models is the existing conditions model calibrated to the 2011 and 2010 flood events, then validated to the 2009 flood event. The calibrated existing conditions model simulates the emergency levees that successfully held during the 2011 flood fight. The calibration and validation process gives confidence that the hydraulic model can represent a wide range of flood events along the Mouse River.

Two baseline models were developed to evaluate the with-Project scenarios relative to existing conditions and a probable 10,000 cfs flood fight condition. Baseline 1 represents existing conditions with no-flood-fight. Baseline 2 represents existing conditions with a successful flood fight to 10,000 cfs for Burlington thru Velva. Figure 3-2 provides an overview of the hydraulic model geometries included in Appendix C2.

3.3.2 WITH-PROJECT HYDRAULIC MODELING

The with-Project simulations represent the hydraulic conditions that would be present following the construction of the flood risk reduction elements presented in the PER between Burlington and Minot. The implementation of the Project was broken into five construction stages as described below. Maps and additional details of each construction stage are provided in Appendix C2.

- Construction Stage 1 – Project segments currently under construction (Design Phases MI-1 and MI-2/3), plus the Minot WTP system recently constructed, and the NDDOT replacement of the US 83 Bypass and Broadway Bridges.
- Construction Stage 1.5 – Project segments necessary to remove a large portion of Minot north of the river from the regulatory floodplain. It would add Project segments for Terracita Vallejo, Maple Avenue High-Flow Diversion, and Design Phase MI-5 (4th Avenue NE Tieback Levee).

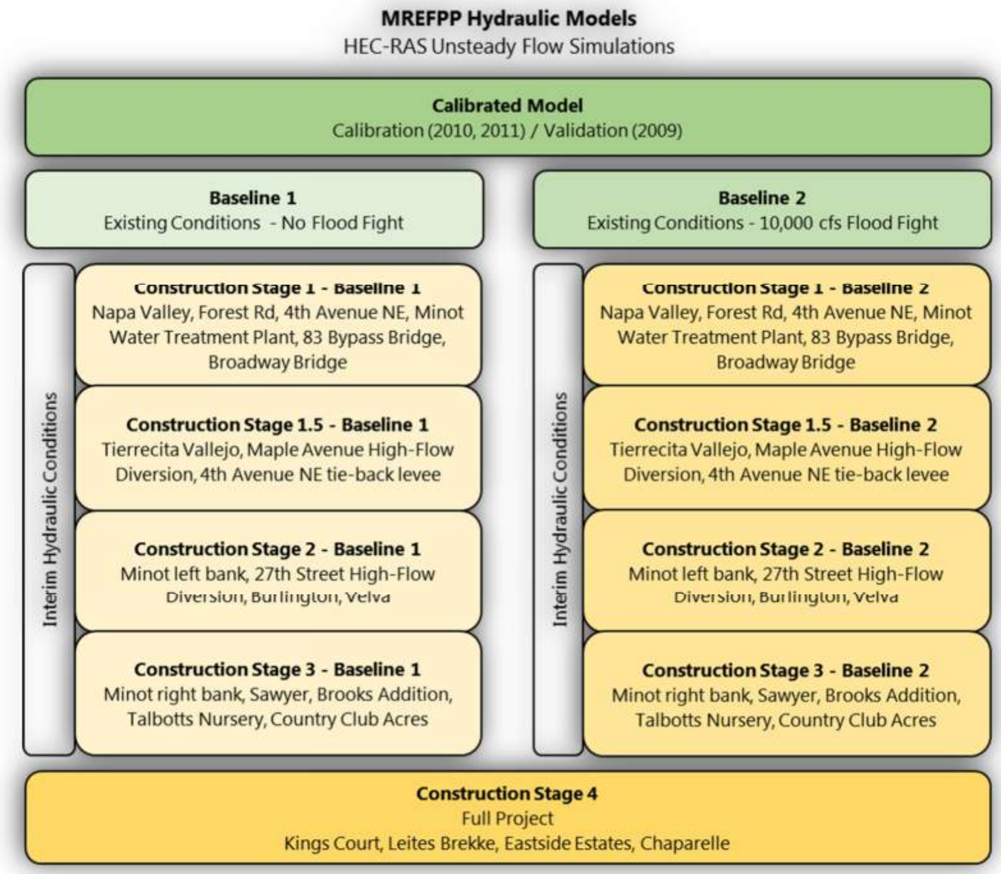


Figure 3-2 Hydraulic Model Geometry Scenarios

- Construction Stage 2 – Project segments necessary to complete all left bank flood risk reduction features through Minot, Burlington, and Velva.
- Construction Stage 3 – Project segments necessary to complete the right bank flood risk reduction features through Minot, Sawyer, and most unincorporated areas between Minot and Burlington.
- Construction Stage 4 (full buildout) – All remaining Project segments between Burlington and Minot with the assumption that all existing levee systems have been replaced.

3.3.2.1 DESIGN EVENT

Project features are being designed to the peak discharge from the 2011 flood event. The USGS collected several flow measurements near the peak of the 2011 flood event. The USGS measured peak discharge at Broadway Bridge in Minot was 27,400 cfs.

3.3.2.2 HYDRAULIC DESIGN WATER SURFACE ELEVATIONS

Only limited alignment changes have occurred in the project area of Phase MI-5 – 4th Avenue NE Tieback Levee since the PER (with the exception of the east end semi-permanent tieback). A map is included in Appendix C1 showing the location of the proposed project features in comparison to the locations proposed in the PER.

The project modeling presented in Appendix C2 was used to establish top elevations for design of project features (levees, floodwalls). These elevations were verified upon development of the adjustments shown in Appendix C1 to ensure that they exceed minimum required levels as presented in Appendix C2.3.

The 90% selected design water surface elevations are shown in Appendix C6.

3.3.3 SLOPE STABILITY WATER SURFACE ELEVATIONS

The evaluation of levee slope stability requires water surface elevations for normal flow conditions, project design flow conditions, and the maximum water surface elevation. Slope stability water surface elevations were based on hydraulic elevations presented in Appendix C2 for this 90% level of design.

3.3.4 HYDRAULIC UNCERTAINTY

Hydraulic uncertainty was evaluated following methods described in EM 1110-2-1619 and presented in Appendix C2. The uncertainties of the natural system and hydraulic models for both existing conditions and with-Project conditions are included along with the method for estimating the standard deviation (uncertainty parameter).

3.3.5 SUPERIORITY

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 objective is to design a system that first overtops at the least damaging location.

The definition for MREFPP established by the SRJB assumes levees and floodwalls will be constructed to at least 3 feet above the design flood elevation for the Project. Superiority was calculated at each hydraulic cross section along Project features and is presented in Appendix C2.

For flood risk reduction segments that are currently in design, superiority was calculated as follows:

Superiority = Final Design Grade - Design Flood Elevation - Hydraulic Uncertainty.

Note that no superiority was included in the east end semi-permanent tieback.

3.3.6 ICE JAMS

Based on information presented in Appendix C2, bridges and closure structures can increase the potential for ice jams. In decades past, ice jams have occasionally formed upstream of some bridges in Minot and Burlington. Ice jam formation typically occurs at flows less than 3,000 cfs, which is roughly a tenth of the design discharge for the Project. Under with-Project conditions, ice jam formation in the Project reach would likely have a maximum impact of a few feet on the upstream hydraulic profile, but the low discharge would mean that Project features crest elevation would be sufficient to contain buildup behind the ice jam.

During high-flow conditions, current reservoir operations tend to make floods through Minot drawn out events that begin with weeks of controlled release of up to 5,000 cfs from Lake Darling. Normal flows in the Mouse River are closer to 100 or 200 cfs. The elevated releases from Lake Darling would tend to melt any remaining ice in the channel. If at some point Lake Darling flood storage is used up and the dam operators have to pass through larger flows, similar to 2011, it is unlikely that there would still be ice in project areas. No additional analyses were performed at the bridges as the majority of the bridges through the study area have been identified for replacement with longer span structures to accommodate additional conveyance capacity.

For these reasons, the potential for ice jams is not a major design consideration for setting the height of levees and floodwalls.

3.3.7 WAVE RUNUP

As shown in Appendix C2, the potential for wave run-up was evaluated to determine if it should be considered in the design of flood risk reduction features. Wave runup is typically a design factor for coastal and lake areas where fetch lengths are long enough for a sustained wind to generate waves. Wave runup is typically not an issue for riverine systems because meandering channels and obstructions in the floodplain, such as vegetation, make fetch lengths too short for wind to generate waves. Wave runup for most riverine systems is also unlikely because it depends on the coincident occurrence of high-water levels, sustained high wind speeds, and the wind direction aligning with narrow open water areas within the floodplain.

A review of aerial photography from the 2011 flood peak shows trees in the floodplain that limit potential fetch lengths to under 1,000 feet. Given the low probability of coincident events creating conditions that allow for wind wave propagation and the short fetch lengths, wave runup is not being used as a design consideration for flood risk reduction features.

3.4 HYDRAULIC ANALYSIS AND PROJECT IMPACTS

Defining the Project impacts is necessary to satisfy the requirements of the 33 USC 408 as outlined in EC 1165-2-216. Appendix C2 presents the analysis of the hydraulic modeling results. The analysis focuses on Project impacts to the depth, duration, location, extents, frequency and probability of flooding. These impacts are quantified in several different ways to present a thorough assessment of interim and post-project impacts both upstream and downstream of project features.

3.4.1 FLOOD PROFILE IMPACTS

Water surface profiles were calculated for the 2009, 2010, and 2011 historic events, and the 10-, 50-, 100-, and 200-year design events. Two sets of water surface profile comparisons plots were generated for each construction stage, one for each of the two Baseline conditions. These results are provided in Appendix C2.

3.4.2 FLOOD HYDROGRAPHS IMPACTS

Comparisons of the downstream flow hydrographs near Verendrye for existing conditions and each of the construction stages are included in Appendix C2.

3.4.3 INUNDATION AREA IMPACTS

Inundation area impacts were evaluated by comparing mapped inundation areas between individual construction stages and their respective existing conditions baseline scenarios. The inundation area analysis was broken out into three locations: upstream of Minot, within Minot, and downstream of Minot. These results are provided in Appendix C2.

3.4.4 STRUCTURE AND PARCEL IMPACTS

Potential impacts to structures were quantified upstream of Minot, within Minot, and downstream of Minot. This highlights structures that were either newly inundated with the Project or interim construction stage, or structures that were no longer inundated. These results are provided in Appendix C2.

3.4.5 DEPTH IMPACTS AT KEY BRIDGES

The change in the water surface profile at key bridges in and around Minot were evaluated across construction stages and for various flood events. The bridges evaluated were:

- County Road 17 / Boy Scout Bridge
- Highway 83 Bypass Bridge
- 16th Street SW Bridge
- Broadway Bridge
- Burdick Expressway Bridge
- Highway 2 Bridge

These results are provided in Appendix C2.

3.4.6 DEPTH-DURATION-FREQUENCY IMPACTS

Depth-duration-frequency (DDF) impacts were evaluated at select locations. The purpose of this impact analysis was to document how the duration of flooding changes at a given location and elevation for a given return frequency.

These results are provided in Appendix C2.

3.5 REGULATORY FLOODPLAIN ANALYSIS

The effective flood insurance study (FIS) for Ward County, North Dakota and Incorporated areas was published in February 2002. The NDSWC and FEMA are working on a revision to the Ward County FIS. The preliminary version of this updated Ward County FIS would change the discharge frequency curve such that the 1% annual chance discharge would go from 5,000 cfs to 10,000 cfs. Given the significant change in discharge, revisions to the regulatory floodway are also proposed.

To the extent possible, the Project avoids work that would cause fill in the current effective regulatory floodway. An area map showing the project features in relation to the effective floodway is shown in Figure 3-3.

An analysis of Construction Stage 1.5 was completed using the Preliminary FIS steady state hydraulic model and the results are presented in Appendix C4. The impacts were further refined for the current alignment for MI-5 and are presented in Appendix C2.3. Additional information related to the regulatory requirements are provided in Section 11.

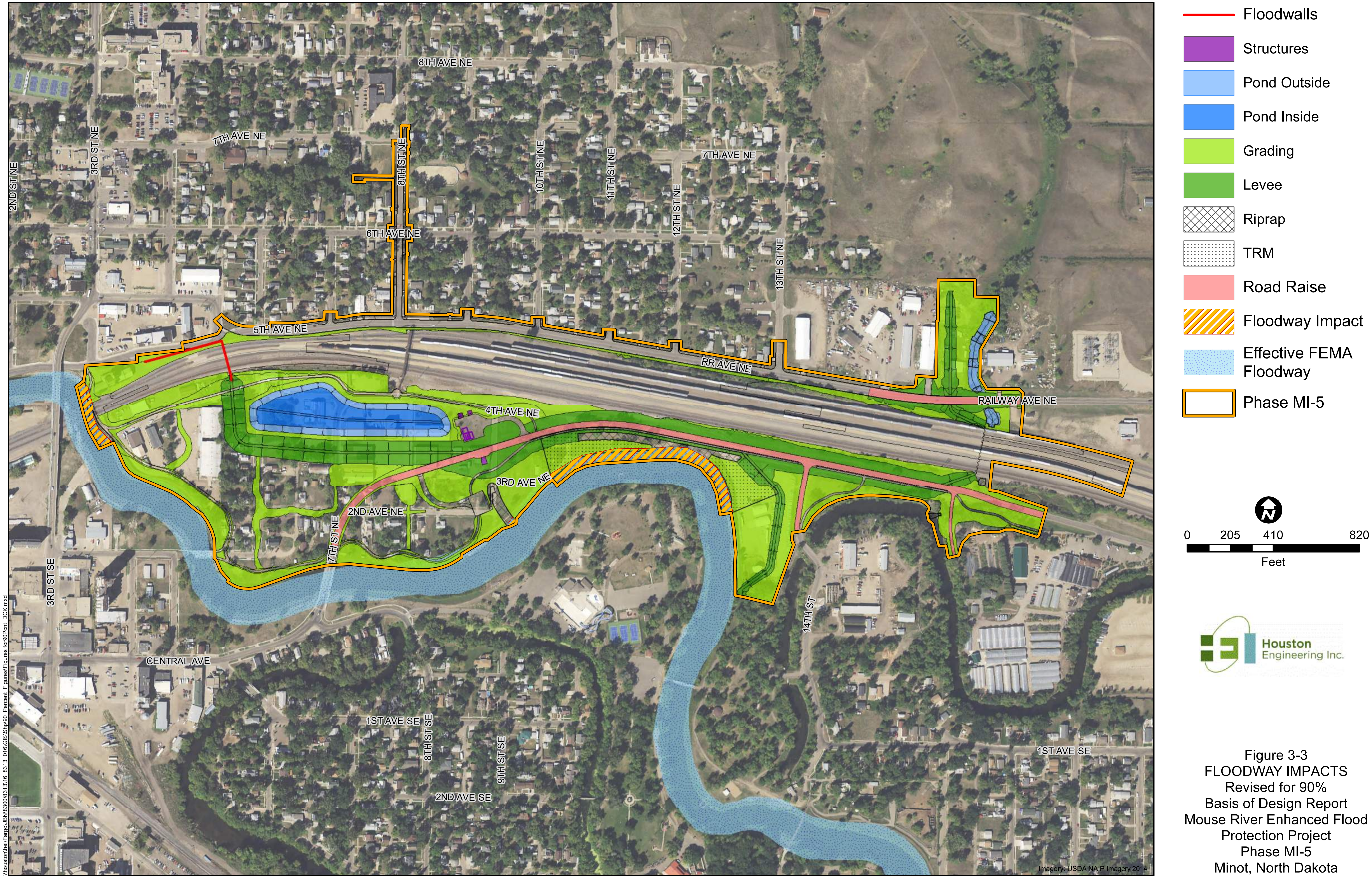
3.6 RIVER HYDROLOGY AND HYDRAULICS SUPPLEMENTAL REVIEW

The results presented in Appendix C2 have been revised based on USACE and IEPR comments received during development of Phases MI-1 – MI-3. As previously stated, a preliminary evaluation was completed to determine expected changes in hydraulics due to the proposed Phase MI-5 alignment variations from the previous Construction Stage 1.5 alignment. Not only were impacts to the steady-state regulatory floodplain modeled, but also unsteady models were analyzed as well. The subsequent hydrologic and hydraulic modeling results have been used for riverine design of Phase MI-5 and are presented in Appendix C2.3.

Since only minor alignment changes were incorporated at the 90% design level, no update to the 60% hydraulics were completed. A final hydraulic evaluation based on the final alignment will be completed and included in the 100% submittal.

3.7 SUMMARY OF PROJECT IMPACTS

Potential impacts were evaluated in several different ways to assess how flooding depths, areas, and durations would change relative to the two baseline scenarios and for different flood magnitudes and are presented in Appendix C2.



Houston Engineering Inc.

Figure 3-3
FLOODWAY IMPACTS
Revised for 90%
Basis of Design Report
Mouse River Enhanced Flood
Protection Project
Phase MI-5
Minot, North Dakota

4 INTERIOR DRAINAGE ANALYSIS

The results of the interior drainage analysis contribute to the Project goal of providing flood risk reduction within the Phase MI-5 area. For the Project, interior areas are defined as areas protected from riverine flooding by the proposed levees and floodwalls (line of protection). Though an interior area may be protected from riverine flooding by an improved line of protection, an inadequate interior drainage system may contribute to flooding, causing or increasing damage to structures or infrastructure from localized large precipitation storm events that might occur when the river stage is high. Interior drainage analysis was completed to identify locations behind the line of protection that may be inundated during the 1-percent rainfall event to develop a drainage plan for gravity and blocked-gravity conditions.

The proposed Phase MI-5 of the Project will provide flood risk reduction features within the Railway Avenue NE area generally between 3rd Street NE and 13th Street NE in Minot. A map of the existing drainage area upstream of Phase MI-5 and the existing interior drainage facility is shown on Figure 4-1. The area south of the BNSF Railway is identified in the PER as the 4th Avenue NE (A430) watershed. Runoff from this watershed is captured by a storm sewer collection system with 3 outfalls to the Mouse River. There are no permanent storm water pumps in this watershed to convey runoff during riverine flood events.

The area north of the BNSF Railway is identified in the PER as the A470 watershed. Runoff from this watershed is captured by a storm sewer collection system with 2 outfalls that cross under the railroad between 12th Street NE and 13th Street NE, and then discharge to the Mouse River. The existing outfalls are 36” and 60” RCP. These outfalls are below the elevation of the weir crest of the downstream Roosevelt Park Control Structure and are perpetually inundated with river water even during low river flows.

The following sections provide a general summary of the interior drainage methodology and proposed modifications to the interior drainage system. The analysis has been conducted using hypothetical storm events for gravity conditions, and both hypothetical and historic storm events for blocked-gravity conditions. The analysis is at a 90% level of design. Additional details of the interior drainage analysis procedure and results are included 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 for Minot [83].

4.1 INTERIOR DRAINAGE ASSESSMENT METHODOLOGY

XP Solutions Storm Water Management Model (XP-SWMM), modeling software accepted by FEMA and widely used in similar applications, was selected for this study. XP-SWMM 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, all of which occur in Minot.

XP-SWMM, Version 2014, was used to model Minot's interior drainage system. Data inputs were (1) hypothetical event and observed precipitation, (2) hydrologic parameters, and (3) hydraulic parameters of the conveyance systems including pipes through the levee and pump station parameters (detailed in Appendix D).

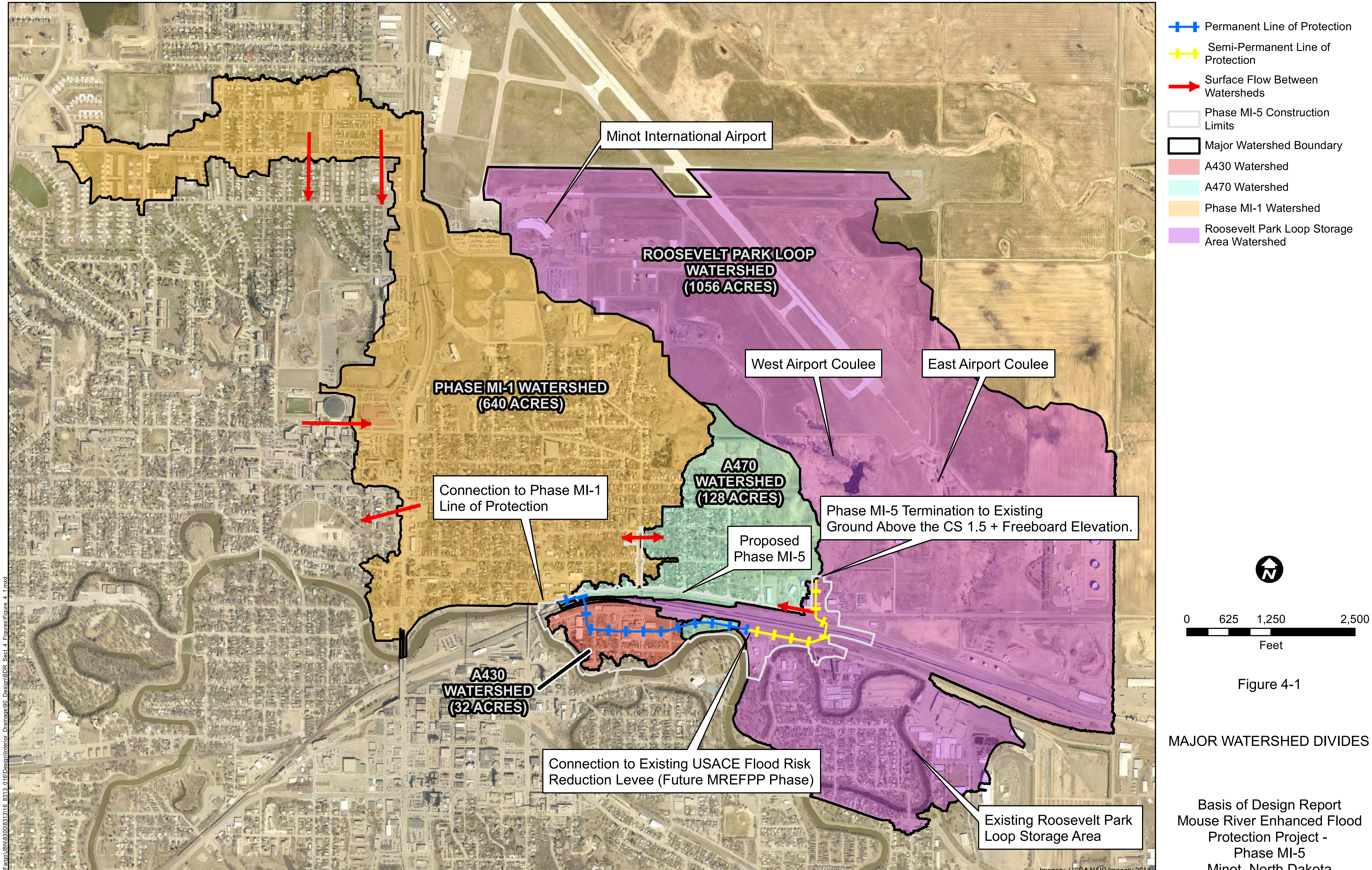


Figure 4-1

MAJOR WATERSHED DIVIDES

Basis of Design Report
Mouse River Enhanced Flood Protection Project - Phase MI-5
Minot, North Dakota

4.1.1 RAINFALL

For the 90% analysis, two types of rainfall data were used in developing the model: Atlas 14 precipitation frequency estimates and observed rainfall events. The following sections summarize these rainfall data sets and how they were applied to model development.

4.1.1.1 ATLAS 14 RAINFALL

Hypothetical rainfall depths were obtained from the precipitation frequency estimates in NOAA Atlas 14 for MINOT INTL AP, Minot, North Dakota. The Annual Maximum Series Atlas 14 rainfall depths were selected.

The methodology from USACE Training Document 15 (TD15), *Hydrologic Analysis of Ungaged Watershed Using HEC-1* [117], was used to develop the 24-hour design distribution for the interior drainage analysis. The design distribution was developed by combining precipitation intensities from shorter-duration storm events that have an equal return period to create a "nested" hyetograph for the overall 24-hour event.

4.1.1.2 OBSERVED RAINFALL EVENTS FOR MODEL VALIDATION

The simulated rate of storm water runoff is sensitive to the time of concentration, drainage area, and timing and duration of storm events. To partially address the uncertainties in timing and duration of storms, Next-Generation Radar (NEXRAD) data were used to simulate the rainfall events that occurred on June 4, 2014, and June 28, 2014. Additional detail regarding rainfall depth and distribution for these events is provided in Appendix D.

4.1.1.3 OBSERVED RAINFALL EVENTS FOR MULTIPLE DISCRETE EVENT ANALYSIS

The Multiple Discrete Event method as described in EM 1110-2-1413 is utilized in the blocked-gravity analysis of the project. This method requires the use of historic observed rainfall data for events that have occurred when the river discharge is above the defined closure discharge. A system of comparing the rivers gage data provided by the USGS to the NOAA rainfall records was created to determine observed rainfall events that have occurred coincident to river discharges in excess of the closure discharges. Additional information regarding the use of this rainfall data with the Multiple Discrete Event method is provided in Section 4.2.1.2.

4.1.2 HYDROLOGIC ANALYSIS

The Natural Resources Conservation Service (NRCS) curve number methodology described in the Technical Release 55 Manual, *Urban Hydrology for Small Watersheds* [66], was used to simulate the conversion of rainfall to storm water runoff. The primary hydrologic input parameters are subwatersheds, curve number, and time of concentration; each are described below.

4.1.2.1 SUBWATERSHEDS

Subwatersheds contributing runoff to the study areas were delineated using topographic information derived from the 2014 Project LiDAR and the 2010 Ward County LiDAR. Once all storm sewer data for the study area was mapped out and inlet points were determined, LiDAR data and GIS tools were used to automatically produce preliminary subwatersheds divides. These divides were then verified and revised based on available aerial photography. Subwatersheds used for this analysis are shown in Figure 4-2 and Figure 4-3. Additional discussion regarding the methodology used to delineate subwatersheds is included in Appendix D.

4.1.2.2 CURVE NUMBER

Existing and future land use within the study area was primarily determined using the *Minot Comprehensive Plan* [75]. For areas not included in that plan the *2011 National Land Cover Database* (NLCD) [63] was used. Existing land uses were used to evaluate existing conditions and to analyze validation events. Future land-use data was used to evaluate modifications to the interior drainage system.

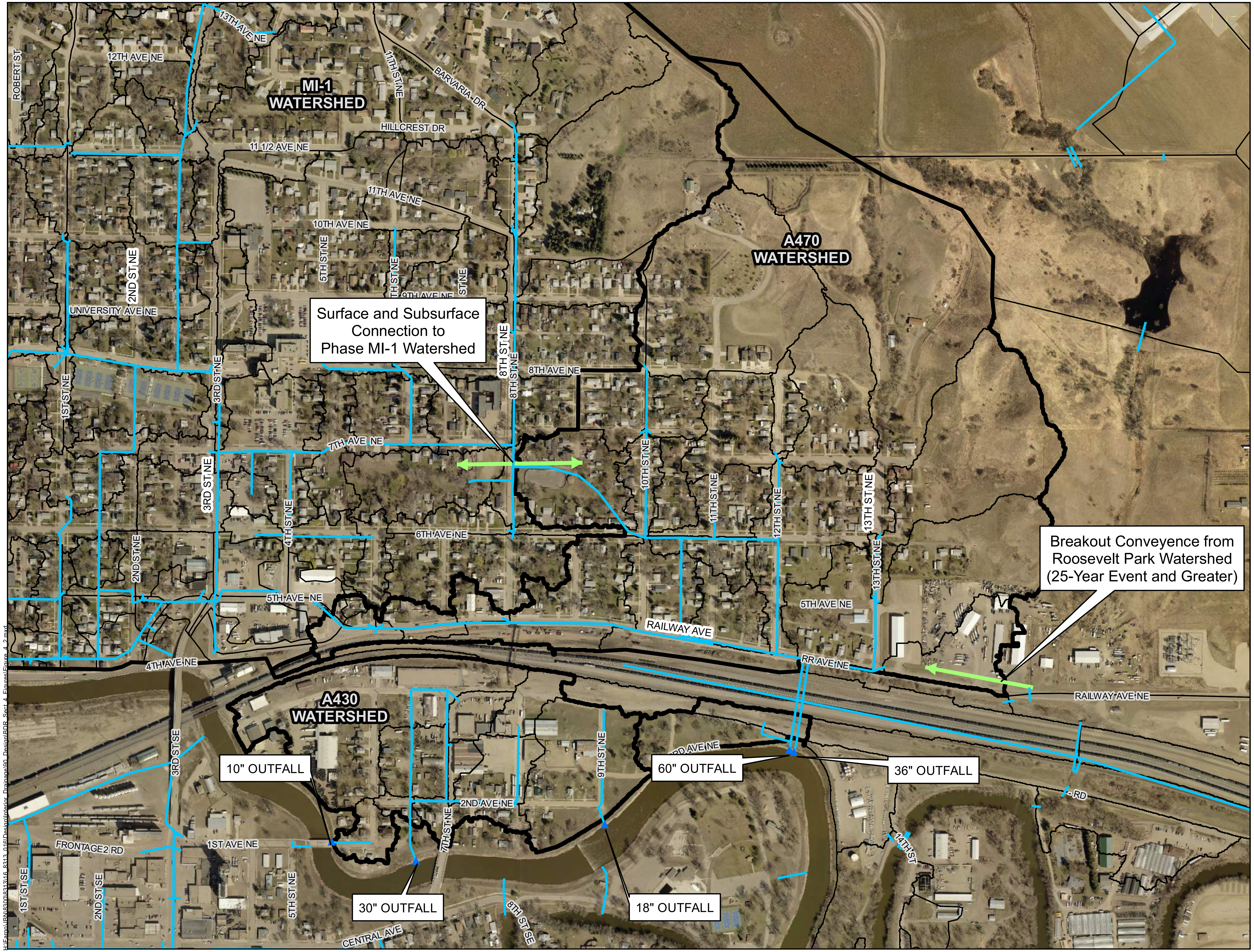
Hydrologic soil data was gathered from the Web Soil Survey (WSS) produced by the National Cooperative Soil Survey Geographic (SSURGO) database [65].

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 [66]. 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 4-1.

Table 4-1 Composite Curve Numbers

Land Use Classification	Comprehensive Plan Land Use	Percent 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, and 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



Storm Sewer Outfalls

Existing Storm Sewer

Surface Flow Between Watersheds

Subcatchment Boundary

Major Watershed Boundary

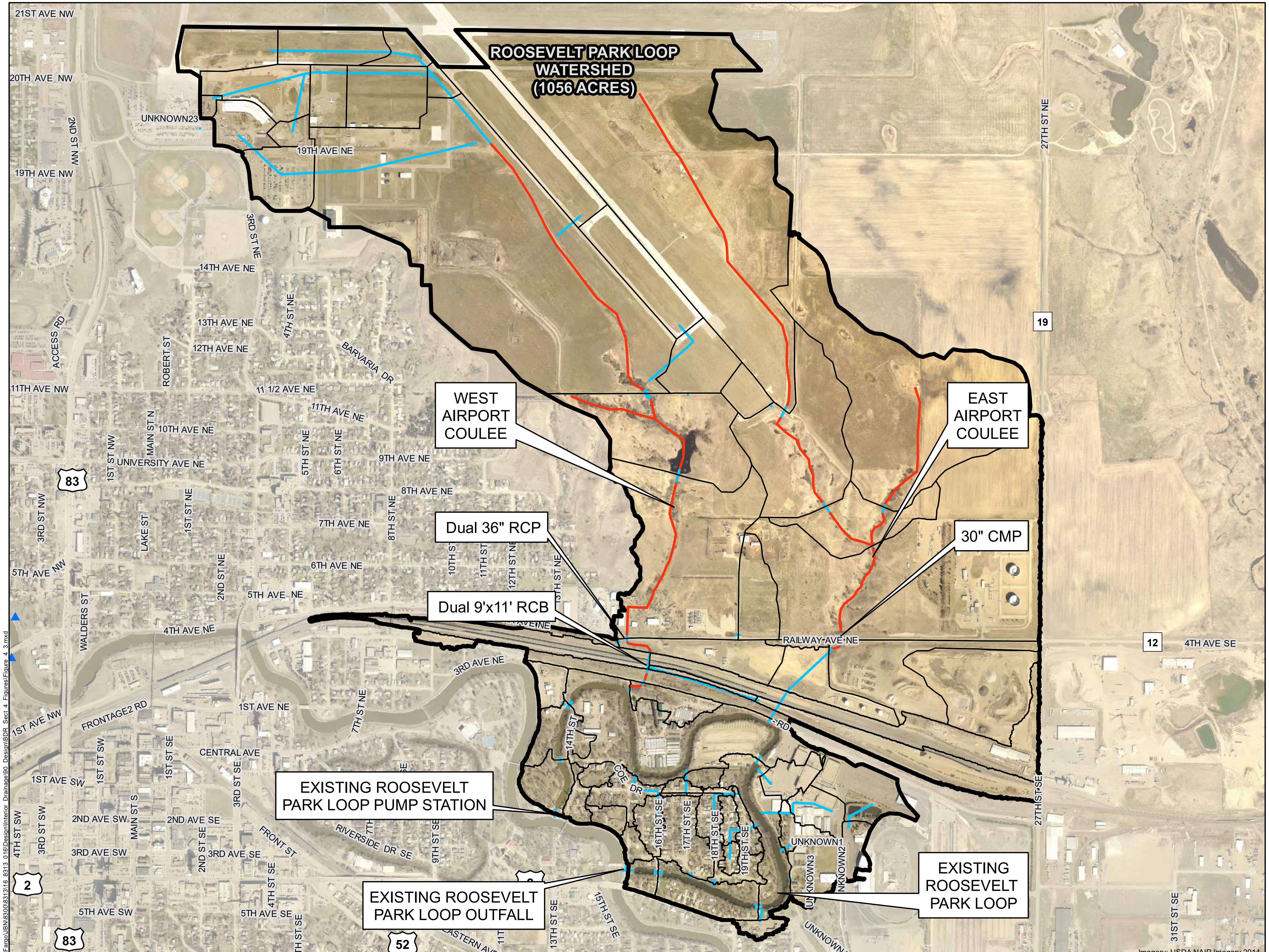
0 225 450 900

Feet

Figure 4-2

PHASE MI-5 WATERSHED

Basis of Design Report
Mouse River Enhanced Flood Protection Project - Phase MI-5
Minot, North Dakota



Storm Sewer Outfalls

Surface Flow Between Watersheds

Subcatchment Boundary

Major Watershed Boundary

Existing Storm Sewer

Existing Open Channel

0 1,000 2,000

Feet

Figure 4-3

ROOSEVELT PARK LOOP WATERSHED

Basis of Design Report
Mouse River Enhanced Flood Protection Project - Phase MI-5
Minot, North Dakota

4.1.2.3 TIME OF CONCENTRATION

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 [64]. Modifications for urbanization were based on the Federal Highway Administration's (FHWA) Hydraulic Engineering Circular 19: Hydrology (HEC-19) [42].

Following the guidance in TR-55 [66], a minimum time of concentration of 6 minutes was used for each watershed.

4.1.3 HYDRAULIC ANALYSIS

The storm sewer pipe network was developed based on survey data collected from the existing storm sewer system along with a combination of Minot and USACE as-built information. Data included pipe sizes, materials, lengths and invert elevations, manhole invert elevations, and inlet configurations and types. Pipe roughness and manhole losses were assigned consistent with the Minot Storm Water Design Standards Manual [79] and FHWA HEC-22: Urban Drainage Design Manual [40].

The storm sewer inlet capacities were modeled consistent with FHWA HEC-22 [40] and manufacturers' information [68]. In locations where multiple catch basins exist at a single street junction or low point, catch basins were modeled as a single inlet to the storm sewer network.

In general, pipes smaller than 18-inches were not modeled in XP-SWMM unless they were perceived to have a significant impact on hydraulic routing. In places with a pipe diameter smaller than 18-inches, a single subwatershed was delineated to the inlet or cluster of inlets where the pipe diameter increases to 18-inches or greater.

4.1.4 MODEL VALIDATION

The June 4, 2014, and June 28, 2014, rainfall events were selected to be used to validate the XP-SWMM models project-wide. These events were selected because they are relatively recent events that have the greatest amount of anecdotal and recorded flood inundation information available. The goal of validation is to match the model-simulated water surface depths in the study area to available photographs taken by residents during the event and/or to anecdotal evidence provided by the local reviewing agencies. Flood inundation record information was not available in the Phase MI-5 study area. As a result, inundation maps for each of the validation events were produced and presented to City of Minot staff for review and validation.

4.1.5 4TH AVENUE NE PUMP STATION AND STORAGE AREA OPERATION

A gate closure elevation of 1541.2’ was selected. This elevation corresponds to a river flow rate of 500 cfs. This elevation was selected because it is approximately 2’ below the elevation where above ground ponding would occur in the lowest point of the drainage area north of the BNSF Railway. For river discharges below 500 cfs, a flap gate will prevent river water from backing into the interior drainage facility while allowing partial gravity discharge from the facility in case of a rain event in which the gravity bypass was utilized. When the river discharge exceeds 500 cfs, a slide gate will be operated which will prevent flow in either direction of the gravity bypass. When the gate is closed, all discharge from the interior area will be achieved via pumping.

4.1.6 TAILWATER CONDITIONS

For the 90% submittal, gravity analysis was conducted as if the river was at average summer depth and flow (181 cfs). For blocked analysis, it was assumed that the river was at 2011 flood depth and flow (27,400 cfs) and that gravity discharge was not possible. Details on tailwater condition and closure discharge are provided in Appendix D.

4.2 INTERIOR DRAINAGE SYSTEM PROPOSED MODIFICATIONS

Modifications to the interior drainage system will be required to prevent impacts to existing infrastructure within Minot. The proposed line of protection intersects the existing storm sewer outfalls in the Phase MI-5 area. Trunk storm sewer in Railway Avenue NE will be rerouted to convey runoff to the intersection of Railway Avenue NE and 9th Street NE. From this confluence, runoff from the A470 watershed will be routed under BNSF Railway to an interior drainage facility between the railway and line of protection. The interior drainage facility will consist of a detention pond, the 4th Avenue NE Pump Station, a gravity bypass pipe, and a gatewell closure structure. Both the pump station and gravity bypass will discharge runoff into the proposed gatewell structure and ultimately to the Mouse River.

Additional detail of the interior drainage facility is provided in the following sections.

4.2.1 DESIGN CONSIDERATIONS

Interior drainage modifications are based on the premise that interior flooding should be minimized and the need for FEMA flood insurance should be reduced for commercial or residential structures landside of the levee systems. 44 CFR §65.10 requires that the extent of areas flooded as a direct result of the placement or improvement of the line of protection be identified and a base flood elevation should be assigned if the average depth of flooding is greater than one foot. To meet the requirements of 44 CFR §65.10 and remove the mandatory flood insurance requirements in the future, the interior drainage conveyance systems and pump stations must be able to reduce interior flooding and minimize structures affected. However, if flooding occurs a significant distance from the proposed project that is caused by an in-place storm conveyance feature that is outside of the project area, and the extent of the flooding is not negatively impacted by the proposed project, a base flood elevation will not be assigned. 44 CFR §65.10 also requires that the interior drainage analysis be based on a coincident evaluation of river levels and interior levels and include both gravity and blocked-gravity conditions analysis.

Storm water management infrastructure such as streets, storm sewers, and catch basins necessary to convey storm water to the project modified interior drainage facilities have been designed based on criteria set forth in the *Minot Storm Water Design Standards Manual* [79].

4.2.1.1 INTERIOR DRAINAGE CRITERIA

The proposed gravity systems have been analyzed using the 100-year, 24-hour hypothetical event to develop 1% interior flood elevations for the gravity condition.

The proposed storm systems have also been analyzed based on the Multiple Discrete Events method presented in EM1110-2-1413 in order to develop 1% interior flood elevations for blocked gravity (high tailwater) conditions.

The Residual Flood Hazard Area based on the interior drainage analysis was developed using the higher results of the 1% flood depths for the gravity and blocked gravity conditions. According to 44 CFR §65.10, only areas with flood depths greater than 1 ft will be mapped as Residual Hazard Areas.

Although it does not have an impact on base flood determination, the proposed storm systems were also analyzed utilizing the 50-year, 24-hour hypothetical event for the partial-blocked gravity and blocked conditions as requested by the SRJB for comparison purposes and sensitivity evaluation.

4.2.1.2 MULTIPLE DISCRETE EVENTS METHOD

The Multiple Discrete Events (MDE) method determines a 1% coincident flood elevation by creating a flood elevation – frequency plot based on Weibull plotting position of the historic coincident events. The Weibull plotting position for each event in the data set is determined as follows:

$P = m/(N+1)$

Where: P = plotting position

m = ordered sequence of flood elevations with the highest equal to 1

N = number of events in the data set (65)

Both historic rainfall records and Mouse River stream flow records were available from 1952 through 2016; therefore, 65 events were included in the data set.

The discharge rate used for the blocked gravity analysis is 500 cfs. Based on a review of historic stream flow and rainfall data, there have been 83 instances where the Mouse has exceeded a flow of 500 cfs. Coincident rainfall has occurred during 54 of those 83 instances.

Details of the historic events selected for the MDE event method are provided in Appendix D.

4.2.1.3 STORM WATER MANAGEMENT INFRASTRUCTURE IMPROVEMENTS

The stormwater management infrastructure improvements in the Phase MI-5 watershed include extending a new 72” RCP under the BNSF Railway to connect the portion of the watershed upstream of the railway to the proposed interior drainage facility, replacing the existing trunk sewer under Railway Ave with larger pipe, installing storm sewer under 8th St from 7th Ave NE to Railway Ave, and connecting a 30” pipe in the BNSF Railway to the collection system. The 72” RCP, the trunk storm sewer in Railway Ave, and the storm sewer in 8th St NE were primarily sized as required to produce acceptable interior drainage analysis results. Additionally, storm sewer, storm sewer inlets, streets, and curb and gutter were designed consistent with the requirements from the City of Minot SWDSM for local and minor arterial streets. 8th and 13th St NE are classified as local streets, while Railway Ave NE is classified as an arterial roadway. These criteria are presented in Table 4-2.

Table 4-2 Storm Water Management Infrastructure Design Criteria

Roadway Class	Structure	Design Storm	Design Storm Condition	100-Year Storm Condition
Local	Storm Sewer	2-Year	All flow conveyed in pipe.	N/A
Local	Street, Curb, Gutter	2-Year	No Curb Overtopping. Flow may spread to the crown of the street.	Inundation not to exceed limits of street right-of-way or 9-inches above crown, whichever is less.
Arterial	Storm sewer	5-year	All flow conveyed in pipe.	N/A
Arterial	Street, Curb, Gutter	10-year	No curb overtopping. Flow spread limited to 10’ from face of curb.	Inundation not to exceed limits of street right-of-way or 3-inches above crown, whichever is less.

In addition to the improvements in the MI-5 watershed, improvements are required for roadway crossings of the west airport coulee in the Roosevelt Park watershed where the proposed semi-permanent portion of the MI-5 line of protection blocks existing breakout flow conveyance paths. The crossings at Railway Ave and 4th Ave NE are sized in accordance with the requirements in Article 89-14 of the North Dakota Century Code. Additional consideration was given to estimated flood depths upstream of the crossings in relation to existing structures. The length of the existing west airport coulee is decreased due to the semi-permanent tieback levee. As a result, rock structures are required to control the grade of the channel by providing near-vertical drop sections with integral channel protection. The design of the rock grade control structures was based on the guidance from Chapter 7 of the City of Minot SWDSM. The rip rap size suggested by the SWDSM was translated to an equal-sized USACE rip rap gradation. In addition to the rock grade control structures, weir walls will be installed at the upstream ends of the existing 11' x 9' box culverts that cross through the BNSF right of way south of Railway Ave. This weir wall will create a vertical drop of approximately 2' into the existing box and will reduce the slope in the channel between the proposed Railway Ave crossing culvert and the existing culvert, as well as dissipate energy in the channel.

4.2.1.4 SEEPAGE COLLECTION SYSTEM

A seepage analysis was conducted as part of the Geotechnical Analysis in Section 2. The analysis indicated the need for a seepage collection system along the landside of the line of protection, primarily in the proposed stormwater pond of Phase MI-5. The seepage collection system will consist of a perforated drain tile that will collect seepage and convey it to the pump station. Additional details of the seepage collection system are provided in Section 6.5.7. Seepage rates, however, will not be significant in comparison to the pump station capacity of 30,000 gpm.

4.2.2 RECOMMENDED OPERATION PLAN

The recommended plan consists of trunk storm sewer along 8th Street NE and Railway Avenue NE, a pump station (4th Avenue NE Pump Station), a detention pond, a gravity bypass pipe, and a gatewell closure structure.

Improvements to roadway crossings (Railway Ave and 4th Ave NE) of the west airport coulee in the Roosevelt Park watershed are also recommended.

Trunk storm sewer will collect runoff from the MI-5 watershed and convey it under the BNSF Railway in a 72" pipe near 9th Street NE. To the east of 9th Street NE, near the line of protection, a trunk storm sewer will connect to an existing 30" storm sewer that captures runoff in the BNSF Railway yard. This trunk storm sewer will convey runoff westward under Railway Avenue until connecting to an existing 48" storm sewer at 13th Street NE. This pipe will increase in size as it connects to additional storm sewer laterals and conveys runoff westward toward 9th Street NE. To the north and west of 9th Street NE, a trunk storm sewer will be installed beginning in a low area near Roosevelt School. This pipe will also connect to trunk storm sewer in 8th Street NE. Runoff from this area will be conveyed south in a 48" pipe to Railway Avenue NE. At Railway Avenue NE, the pipe will connect to additional storm sewer in Railway Avenue NE west of 8th Street NE. This pipe will then connect to the 72" pipe at 9th Street NE.

The invert of the 72" pipe south of the BNSF Railway will be approximately 1535', where it discharges into a junction structure upstream of the pump station known as STS 2. This elevation is below the normal river elevation at the discharge location. The normal river elevation is set by the USACE-constructed river control structure at the Roosevelt Park Loop. The invert elevation of this control structure is approximately 1540'. Since the 72" pipe will be lower than the normal river elevation, all low flows in the storm sewer collection system will be routed to the proposed pump station and will be required to be pumped into the river. A gravity bypass pipe will be constructed to convey runoff during moderate to high intensity storm events during gravity conditions. This pipe will be constructed above normal river elevations to reduce maintenance needs by keeping the pipe dry and free of sediment-laden river water during normal river flows. The invert of the upstream end of the bypass pipe at STS 2 will be approximately 1541.2' and the downstream invert will be approximately 1540.5' at the outfall to the Mouse River. Analysis indicates that the gravity bypass will be used for all storms with a return period greater than 2 years. A detention pond will also be constructed and connected to STS 2 with a bottom elevation of 1539.75'. This pond operates to store runoff which helps reduce the required pumping capacity. Preliminary analysis indicates that a 2-year, 24-hour hypothetical storm event will produce a peak water surface of approximately 1540.9' in the detention pond.

Both the pump station and gravity bypass pipe will convey runoff to a proposed gatewell in the line of protection. This gatewell will be constructed to the 2011 flood elevation plus extra height for hydraulic uncertainty, anticipated settlement, structural superiority, and to provide redundant closure to protect against riverine flooding. Runoff will be conveyed from the gatewell to the Mouse River by a pipe and open channel.

Figure 4-4 provides a map of the proposed improvements associated with the Phase MI-5 project.

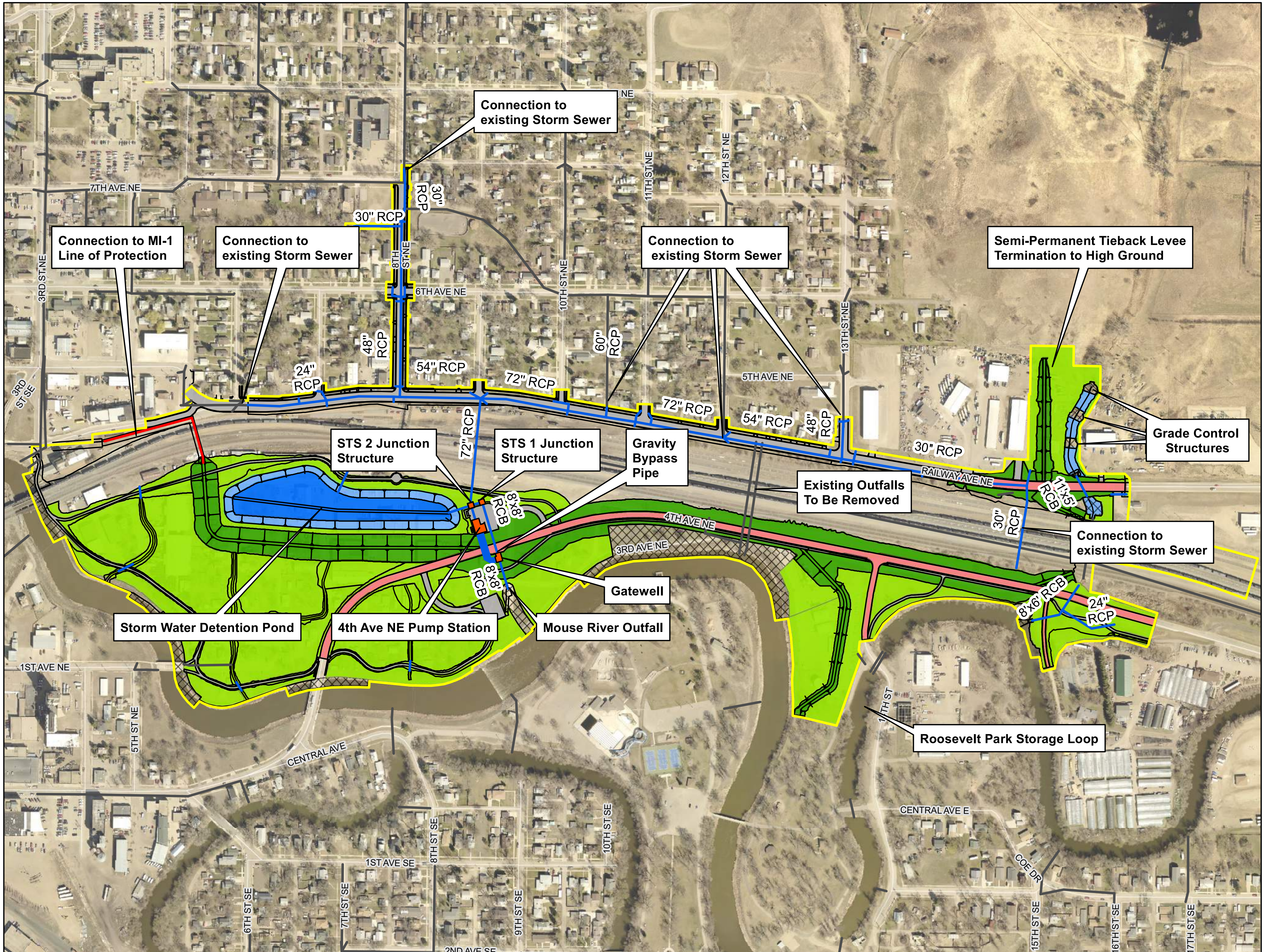
4.2.3 PROPOSED PUMP STATION

The proposed pumps, pump station, and storage system for the new station will be designed based on guidance presented in ANSI/HI Pump Standards, Version 3.1 [51], USACE EM 1110-2-3102 General Principles of Pumping Station Design and Layout [107], the Federal Highway Administration's HEC-24 Highway Storm Water Pump Station Design (HEC-24) [41], as well as local pump station design requirements from the City of Minot and additional resources recommended by the USACE as defined in Appendix G.

The pump station will consist of 3 - 10,000-gpm submersible pumps in a rectangular wetwell. The purpose of these pumps is to provide discharge capacity for runoff from low flow water entering the collection system, runoff from events that are not large enough to utilize the gravity outlet, and all events that occur during a blocked-gravity condition. These pumps will be activated when the water surface in the wet well has risen to 1536', which is 1' above the invert of the upstream collection system invert at STS 2, and will be deactivated when the water surface in the wet well is low enough to have completely drained the collection system (1534.5').

The type and configuration of this set of pumps was selected for multiple reasons. The MREFPP Project Design Guidelines[49] require that pump redundancy be included so that in the case of a single pump failure, the remaining pumps would be able to provide 2/3rds of the design capacity of the station. Having 3 pumps provides adequate redundancy in this case.

Additionally, submersible pumps provide for more cycles per hour than most other pump types and are easily removable for maintenance purposes. Details of the operating elevations and cycle time computations for the submersible pumps are provided in Appendix D and Appendix G.



Legend

- Interior Drainage Structures
- Proposed Storm Sewer
- Existing Storm Sewer
- Phase MI-5 Construction Limits
- Floodwall
- Levee
- Grading
- Pond Inside
- Pond Outside
- Road Raise
- RipRap
- Pavement

0 187.5 375 750
Feet

Figure 4-4

MI-5 & ROOSEVELT PARK LOOP DRAINAGE AREA PROPOSED IMPROVEMENTS

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Additional details regarding the 4th Avenue NE Pump Station can be found in Section 6.8 and Appendix G of this document.

A schematic of the interior drainage facility operations during gravity and blocked conditions is provided as Figure 4-5 and Figure 4-6 respectively.

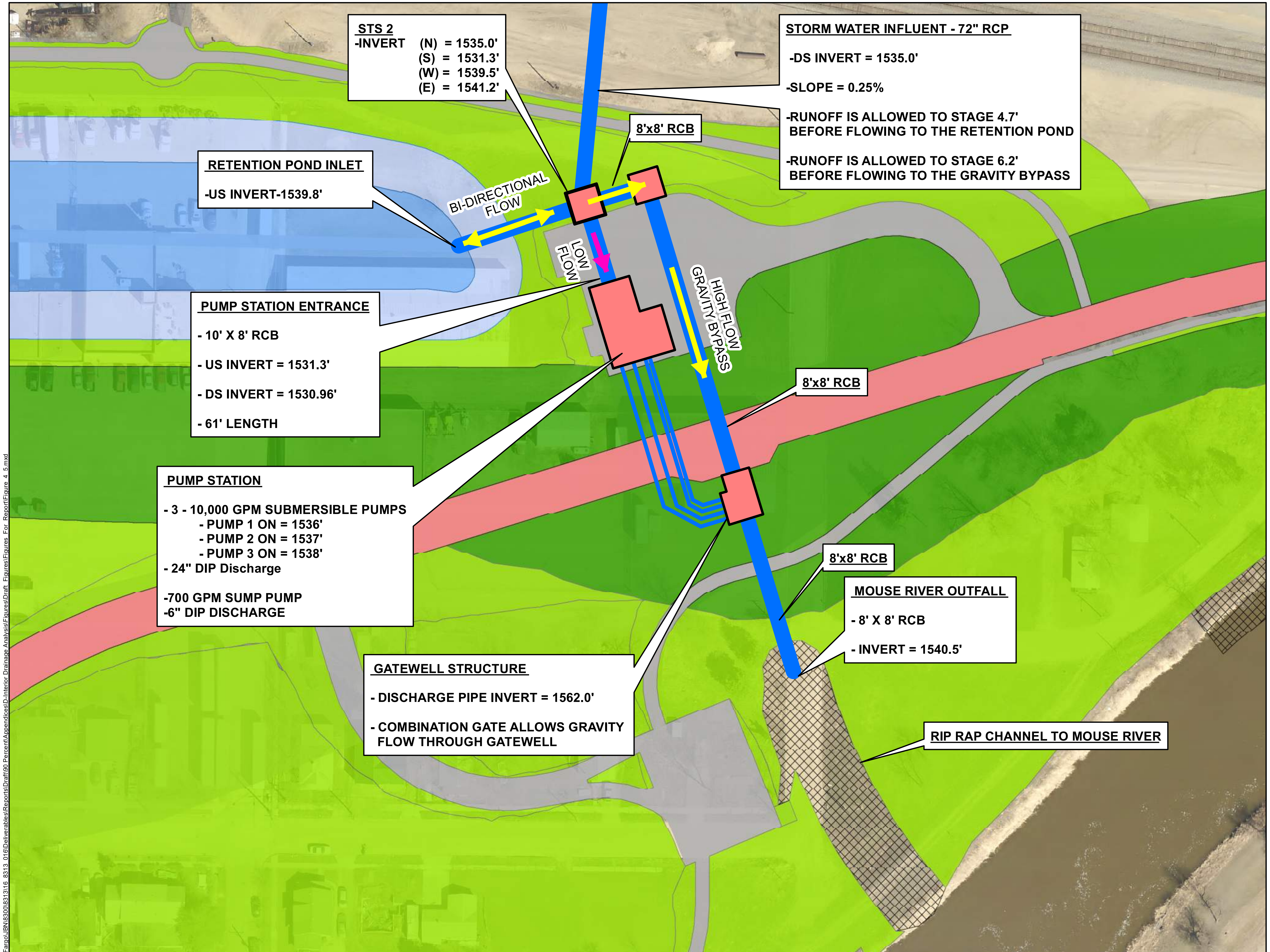
4.2.4 ROOSEVELT PARK

An existing breakout conveyance path from the Roosevelt Park watershed to the A470 watershed will be blocked by the proposed MI-5 line of protection as shown on Figure 4-1. This blockage will induce water surface elevations upstream of Railway Ave that are above existing conditions water surface elevations, would impact upstream structures, and would overtop the roadway, causing potential erosion problems near the line of protection that could lead to levee failure. For this reason, the crossing conduits through Railway Ave and 4th Ave must be increased to convey this additional flow to the Roosevelt Park cutoff loop. However, the blockage of the breakout path and the increased crossing pipe capacity will act to increase both the peak discharge and total volume contributing to the Roosevelt Park cutoff loop.

An analysis was conducted to determine the impacts of this increased conveyance to Roosevelt Park. The 0-damage elevation in the cutoff loop area is approximately 1544.9, which corresponds to the floor elevation of a greenhouse on the SW corner of the Lowe's Garden Center facility. While the impacts are minor for the gravity condition, an increase in peak water surface elevation is realized when the 50-year, 24-hour hypothetical event is analyzed with a blocked-gravity condition. The existing conditions water surface elevation for this event is 1544.2', while the proposed water surface elevation is 1545.2' (0.3' above the 0-damage elevation). Analysis of stream flow statistics for the Mouse River indicates that the closure discharge of 500 cfs for the Roosevelt Park area is exceeded approximately 15% of the time. The combined probability of receiving a 50-year, 24-hour rainfall during a time when the gates are closed is 0.003 (0.02 X 0.15), or a return period of 333 years. An MDE evaluation was also completed on this watershed using the same historic events used for the other MI-5 analysis, since the closure discharges are the same. The plotted 1% chance WSEL for the Roosevelt Park cutoff loop is 1541.8', which is 3' below the 0-damage elevation.

Despite the residual flood hazard area not impacting any structures within the Roosevelt Park area, it is suggested that the City of Minot maintain portable stormwater pumps on standby during a flood in case an event occurs that exceeds the capacity of the existing Roosevelt Park pump station.

As stated in Section 4.2.1.3, improvements to the existing airport channel are required. Details of those improvements are found in Appendix K. The detention areas along the Airport Coulee upstream of the Roosevelt Park cutoff loop and Railway Ave are currently under evaluation by the City of Minot and the Airport Authority to determine their effectiveness at mitigating increased runoff volume and discharge rates due to recent airport improvements. This analysis is expected to be complete in December of 2018. However, the discharge rates from the airport area used in the MREFPP analysis are based on the most current preliminary design for the airport and are not expected to change significantly. The effects of the final evaluation will be studied prior to the 100% BDR submittal to determine impacts to the Roosevelt Park Loop storage area and upstream roadway crossings. To determine the residual flood hazard along Airport Coulee channel upstream of Railway Ave, peak water surfaces were analyzed at four places along the proposed channel. The first location is directly upstream of the proposed box culvert through Railway Ave. The remaining three locations are at the upstream crest of the three rock grade control structures. Full period of record MDE analysis at these locations was completed and the Weibull plots are included in Appendix D.



Legend

- Levee
- Grading
- Pond Inside
- Pond Outside
- Road Raise
- RipRap
- Pavement

0 30 60 120
Feet

Figure 4-5

PROPOSED GRAVITY CONDITION OPERATION

Basis of Design Report
Mouse River Enhanced Flood Protection Project - Phase MI-5
Minot, North Dakota

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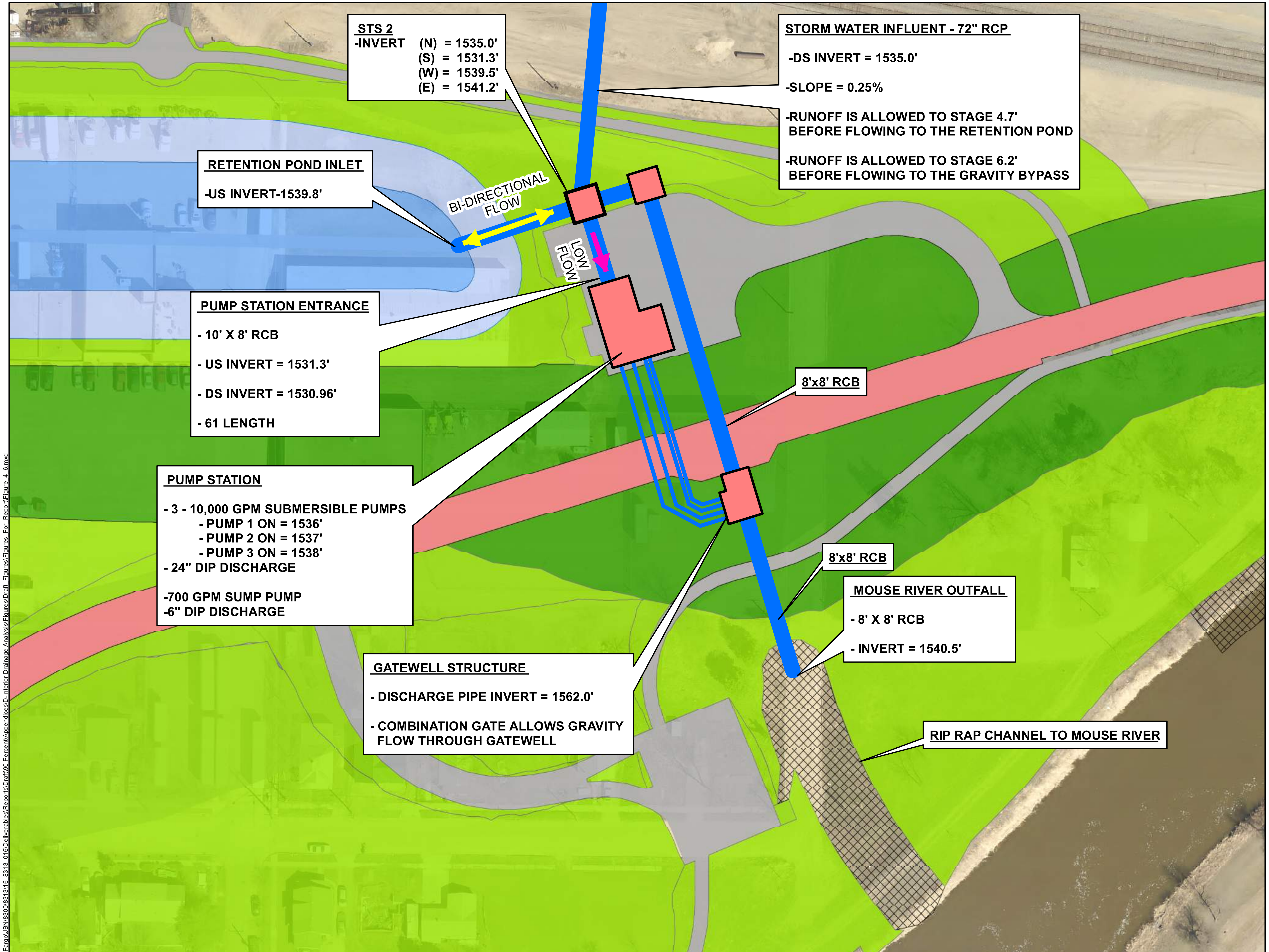


Legend

- Levee
- Grading
- Pond Inside
- Pond Outside
- Road Raise
- RipRap
- Pavement

N

0 30 60 120
Feet

Figure 4-6

PROPOSED BLOCKED CONDITION OPERATION

Basis of Design Report
Mouse River Enhanced Flood Protection Project - Phase MI-5
Minot, North Dakota

5 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. Non-federal proposals to alter or modify an existing USACE project, such as the proposed MREFPP, must be evaluated similar to that which is required for 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).

Several environmental surveys and inspections have been completed in Phase MI-5 to collect data for environmental review and permitting, document existing conditions at the site, and assist in design and engineering of the Project. These surveys and inspections include wetland delineations, ordinary high-water level (OHWL) determination, biological studies, cultural resources investigations, and a review of potential hazardous, toxic, and radioactive waste (HTRW) sites in or near Phase MI-5. A pre-demolition inspection of any remaining structures to be removed from the Project area 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 MREFP Project has been conducted to comply with NEPA regulations (33 CFR Part 230). A Final Programmatic Environmental Impact Statement (FPEIS) has been developed for the MREFPP and covers general impacts associated with construction of the full project from Burlington through Minot. General impacts associated with construction and operation of the Project from Burlington through Minot as well as site-specific impacts associated with features purposed for construction through Stage 1.5 are described in this document. This FPEIS has been 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" [82]. The FPEIS evaluates resources listed in Section 122 of the Rivers and Harbors Act of 1970 and includes an analysis of Project alternatives and the direct, indirect, and cumulative impacts on significant area resources. The Notice of Availability for the Final Programmatic Environmental Impact Statement (FPEIS) was published in the Federal Register for the review period of July 14, 2017 to August 14, 2017. The Record of Decision was issued on December 19th, 2017, and a copy is included in Appendix O1.2. A copy of the FPEIS (less appendices) is provided in Appendix O1.1 as part of the 90% submittal.

5.1.1 PROJECT ALIGNMENT CHANGES

As described in Section 1.6.1, multiple alternative alignments were considered for Phase MI-5. The alignment that was selected and has been carried through to this 90% level of design varies from the alignment proposed at the time of the submission of the FPEIS (Appendix O1.1) and the original version of the Section 404 Permit Application for Construction Stage 1.5 (Appendix H3.1) that was approved by the USACE. The originally proposed flood protection alignment for Phase MI-5 was located north of the BNSF railroad. As originally proposed, the entire length of MI-5 would have been semi-permanent in nature because ultimately future phases of the MREFPP (i.e. Construction Stage 2) would have replaced MI-5 when it continued full-height protection along the river on both sides.

The current alignment generally follows the alignment of Construction Stage No. 2 of the MREFPP until station 74+07. From there the easterly end of Phase MI-5 (east of station 74+07) is intended to provide semi-permanent flood protection by providing a tie-in to high ground on the east end. This is considered semi-permanent because ultimately future phases of the MREFPP will connect in to Phase MI-5 at station 74+07 and continue full-height protection along the river on both sides.

5.1.2 ENVIRONMENTAL REVIEW ADDENDUM

Due to an amended alignment for Phase MI-5, an addendum to the FPEIS will be required for Phase MI-5. The addendum is currently under development and will be included in Appendix O1 in the 100% submittal.

5.2 WETLAND DELINEATION

Wetland delineations over the majority of the Project corridor were conducted (May 18-22, June 22-25, and July 23rd, 2015) for field reconnaissance and data collection. Following guidance from USACE Wetland Delineation Manual (1987), the offsite wetland review consisted of examination of the National Wetland Inventory (NWI, US Fish and Wildlife Service), aerial photography (2010, 2014), NRCS Hydric Soil Ratings (USDA-NRCS), topographic maps, and LiDAR elevation imagery. Layers for photography, LiDAR, and NWI were viewed using ArcMap (ArcGIS 10.2.2 and 10.3 ©ESRI) and other layers were added including boundaries (project corridor, construction phases) and proposed construction features (pump stations, floodwalls, levees, high flow diversions, interior ponding areas, and overbank excavation areas). The Wetland delineation report is provided in Appendix O2.1.

The offsite study identified 117 potential wetland sites (304 total acres) throughout the Project corridor between Burlington and Minot. Most of these features, such as oxbows, resulted from the meandering of the Mouse River in the relatively flat landscape. Some of these wetland areas correspond to the NWI-listed wetlands. Regarding the rivers and streams in the corridor, only the Mouse River is listed in NWI. In Burlington, the Des Lacs River is the main tributary to the Mouse River. Closer to Minot several smaller tributaries drain into the Mouse River, most of which are unnamed, but the named ones are Gassman Coulee, South Branch Coulee, First Larson Coulee, Second Larson Coulee, and Livingston Creek.

The majority of the wetlands identified in the offsite study were not investigated further but were noted in the event of adjusted or additional planned construction activities. Of the total potential wetland sites, 39 were potentially located at sites of planned construction areas between Burlington and Minot. These 39 wetlands were investigated further, either with detailed wetland delineation or field verification.

Due to project alignment amendments, part of the current Phase MI-5 project area was not included in the 2015 study and therefore a supplemental wetland delineation was performed on September 14, 2017. The supplemental wetland delineation report is included in Appendix O2.2.

Based on the wetland delineation completed in 2015 and supplemented in September 2017, an estimate of wetland impacts within the construction limits was determined and is included in Table 5-1. The wetland impact areas impacted are also shown on Figure 5-1.

SUMMARY TABLE: WETLAND IMPACTS			
WETLAND IMPACT NUMBER	PERMANENT IMPACT AREA (ACRES)	TEMPORARY IMPACT AREA (ACRES)	IMPACTING FEATURE(S)
1	0.00	0.00	GRADING
2	0.04	0.00	LEVEE, GRADING
3	0.03	0.00	GRADING
4	0.04	0.04	GRADING
5	0.02	0.00	GRADING, ROAD
6	0.00	0.00	GRADING, PATH
7	0.00	0.00	GRADING, PATH
8	0.07	0.00	GRADING
TOTAL	0.20	0.04	-

LEGEND

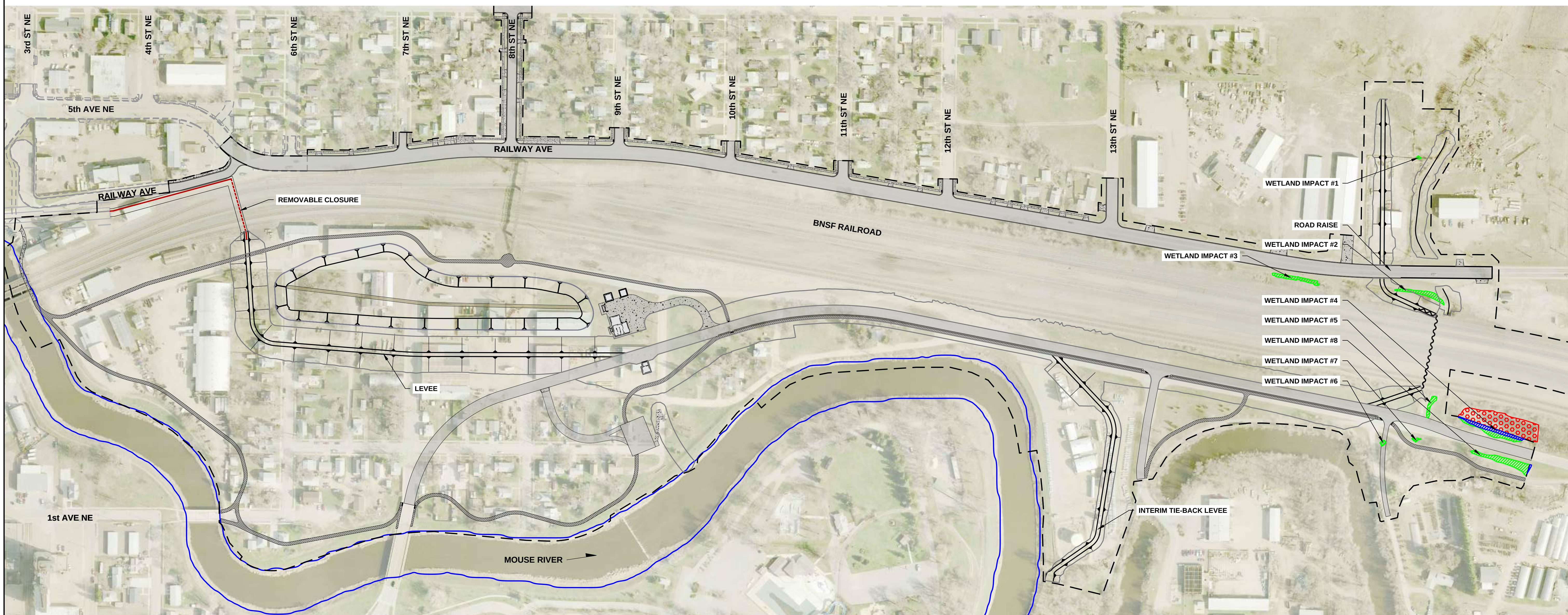
NE LEVEE EXTENSION CONSTRUCTION LIMITS

OHWL

WETLAND BOUNDARIES

PERMANENT WETLAND IMPACTS

TEMPORARY WETLAND IMPACTS



1 PLAN: WETLAND IMPACTS
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150 0 150 300
Scale Feet

N

Figure 5-1 Supplemental Survey Wetland Areas
Phase MI-5
Revised for 90%

MREFP PROJECT DATUM:
HORIZONTAL: NORTH DAKOTA STATE PLANE (NAD83), NORTH, US FT
VERTICAL: NAVD 88

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION

THIS DOCUMENT WAS ORIGINALLY ISSUED AND SEALED BY ENTER NAME, REGISTRATION NUMBER PE-XXXX ON X-XX-XX AND THE ORIGINAL DOCUMENT IS STORED AT XXXX.

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CONSTRUCTION							
	DATE RELEASED						

Houston Engineering Inc.
Ph: 701.237.5065

Scale	AS SHOWN
Date	11/20/18
Drawn	DEK
Checked	JDB
Designed	KAL
Approved	JDB

SOURIS RIVER JOINT WATER RESOURCE BOARD
MINOT, NORTH DAKOTA

MOUSE RIVER - NE LEVEE EXTENSION
ENHANCED FLOOD PROTECTION PROJECT

SECTION 404 PERMIT APPLICATION
ADDENDUM WETLAND IMPACTS

ENG. PROJECT No. 8313-007
CLIENT PROJECT No. 3529.02/3529.03
DWG. No. 1 OF 1
REV. No. -

CADD USER: Thomas Kornkven FILE: H:\FARGO\JBN\8300\8313\14_8313_007\CS 1.5 PERMIT APPLICATION\CAD\IMPACTS-FEMA EXTENSION - WETLANDS (WITH MI-5) - 90%.DWG PLOT SCALE: 1:799.5449 PLOT DATE: 11/27/2018 2:13 PM

Table 5-1 Wetland Impact Estimates

Wetland Impact Number	Permanent Impact Area (Acres)	Temporary Impact Area (Acres)	Impacting Feature(s)
1	0.00	0.00	Grading
2	0.04	0.00	Levee, Grading
3	0.03	0.00	Grading
4	0.04	0.04	Grading
5	0.02	0.00	Grading, Road
6	0.00	0.00	Grading, Path
7	0.00	0.00	Grading, Path
8	0.07	0.00	Grading
Total	0.20	0.04	-

5.3 ORDINARY HIGH-WATER LEVEL 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 ordinary high-water level (OHWL) is used to determine the jurisdictional boundaries of these waterbodies. The OHWL of the Mouse River was determined at several transects throughout the Project area in accordance with the State Water Engineer OHWL guidance document[72]. Identifying the OHWL 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 OHWL data points were georeferenced with a Trimble GPS unit. A summary of the OHWL determination results is provided in Appendix O2.1. Construction activities will be conducted below the OHWL of the Mouse River as part of Phase MI-5. Most of the permanent impacts are the result of placement of erosion and scour protection along the shoreline to prevent bank erosion. OHWL impact estimates are given in Table 5-2. The OHWL impact areas are shown on Figure 5-2.

Table 5-2 River Impact Estimates

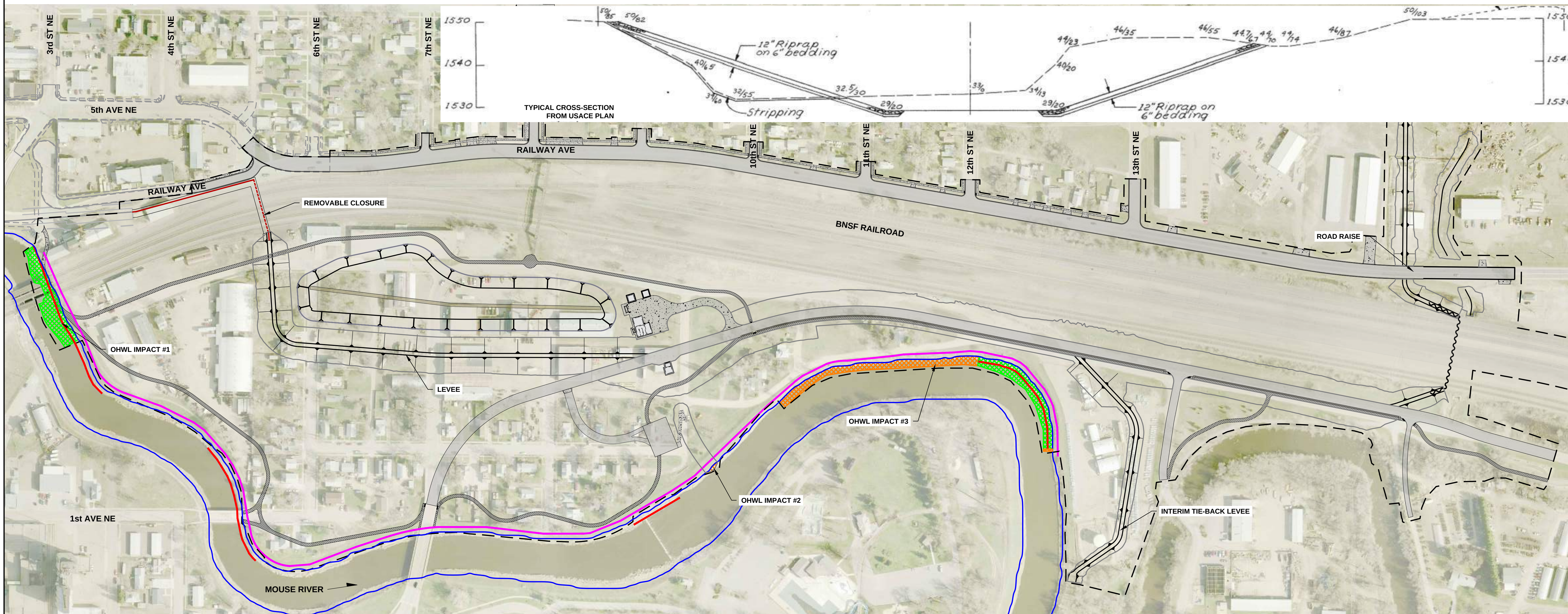
OHWL Impact Number	Permanent Impact Area (Acres)	Temporary Impact Area (Acres)	Length of Proposed OHWL Impact (LF)	Length of Proposed OHWL Impact Outside of Prior USACE Riprap Placement (LF)	Length of Proposed OHWL Impact Outside Prior USACE Riprap and Impact of Channel Bank Excavation (LF)	Impacting Feature(s)
1	0.33	0.00	315	0	0	Riprap/Path
2	0.01	0.00	51	51	0	Riprap
3	0.61	0.00	1048	645	0	Riprap
Total	0.95	0.00	1414	696	0	

SUMMARY TABLE: IMPACTS BELOW OHWL

OHWL IMPACT NUMBER	PERMANENT IMPACT AREA (ACRES)	TEMPORARY IMPACT AREA (ACRES)	LENGTH OF PROPOSED OHWL IMPACT (LF)	LENGTH OF PROPOSED OHWL IMPACT OUTSIDE OF PRIOR USACE RIPRAP PLACEMENT (LF)	LENGTH OF PROPOSED OHWL IMPACT OUTSIDE PRIOR USACE RIPRAP AND IMPACT OF CHANNEL BANK EXCAVATION (LF)	IMPACTING FEATURE(S)
1	0.33	0.00	315	0	0	RIPRAP/PATH
2	0.01	0.00	51	51	0	RIPRAP
3	0.61	0.00	1,048	645	0	RIPRAP
TOTAL	0.95	0.00	1,414	696	0	-

LEGEND

- NE LEVEE EXTENSION CONSTRUCTION LIMITS
- OHWL
- PERMANENT IMPACTS BELOW OHWL
- PERMANENT IMPACTS BELOW OHWL - AREA OF PRIOR USACE RIPRAP
- PERMANENT IMPACTS BELOW OHWL - AREA OF PRIOR USACE CHANNEL EXCAVATION
- EXTENTS WITHIN PROPOSED PROJECT OF PRIOR USACE CHANNEL BANK EXCAVATION AS PART OF ORIGINAL FLOOD CONTROL PROJECT DESIGN
- EXTENTS WITHIN PROPOSED PROJECT OF PRIOR USACE RIPRAP PLACEMENT AS PART OF ORIGINAL FLOOD CONTROL PROJECT DESIGN



1 PLAN: IMPACTS BELOW OHWL

150 0 150 300
Scale Feet

Figure 5-2 River Impact Area
Phase MI-5
Revised for 90%

MREFP PROJECT DATUM:
HORIZONTAL: NORTH DAKOTA STATE PLANE (NAD83), NORTH, US FT
VERTICAL: NAVD 88

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION

THIS DOCUMENT WAS ORIGINALLY ISSUED AND SEALED BY ENTER NAME, REGISTRATION NUMBER PE-XXXX ON X-XX-XX AND THE ORIGINAL DOCUMENT IS STORED AT XXXX.

CLIENT
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BID
CONSTRUCTION
RELEASED TO/FOR
A B C 0 1 2 3
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Houston Engineering Inc.
Ph: 701.237.5065

Scale	AS SHOWN
Date	11/20/18
Drawn	DEK
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Approved	JDB

SOURIS RIVER JOINT WATER RESOURCE BOARD
MINOT, NORTH DAKOTA

MOUSE RIVER - NE LEVEE EXTENSION
ENHANCED FLOOD PROTECTION PROJECT

SECTION 404 PERMIT APPLICATION
ADDENDUM IMPACTS BELOW OHWL

ENG. PROJECT No. 8313-007
CLIENT PROJECT No. 3529.02/3529.03
DWG. No. 1 OF 1
REV. No. -

CADD USER: Dan Korf FILE: H:\FARGO\JBN\8300\8313\14_8313_007\CS 1.5 PERMIT APPLICATION\CAD\IMPACTS-FEMA EXTENSION - OHWL (WITH MI-5) - 90%.DWG PLOT SCALE: 1:799.5449 PLOT DATE: 12/13/2018 10:23 AM

- OHWL Impact No. 1

  Impact No. 1 is in the area of the existing BNSF railroad bridge. Riprap is required in this area for slope stabilization and erosion protection necessary to protect this critical structure. The proposed riprap will generally be a replacement of riprap that already exists at this location but will be upsized based on current design guidance.

- OHWL Impact No. 2

  Impact No. 2 is in the area of the proposed Pump Outfall. Riprap is required in this area for bank stabilization and erosion protection.

- OHWL Impact No. 3

  The Mouse River currently meanders within close proximity of the existing 4th Avenue NE alignment. The current MI-5 project proposes incorporating flood risk management features within the proposed 4th Avenue NE section at this location; therefore, slope stabilization and erosion protection is necessary to protect this critical infrastructure. Riprap is the proposed method of slope stabilization and erosion protection due to Mouse River flow velocities. The proposed Impact No. 3 riprap will be a replacement of riprap that already exists at this location for approximately 40% of the length of the impact but will be upsized based on current design guidance.

5.4 BIOLOGICAL INVENTORY

A biological inventory was also conducted for the project area, which included an evaluation of raptor nests (primarily bald eagles) in the Project area and bird use under bridges in the Project corridor as well as an estimate of the number of trees within potential disturbance areas. These studies were conducted in late spring and early summer of 2015. A summary of the biological inventory is provided in the Houston Engineering, Inc. 2015 report entitled “Wetlands, Waters, and Biological Inventory Report” (Appendix O2.1).

No eagle or other raptor nests were observed within the vicinity of Phase MI-5 during the initial inventory. To verify raptor nesting sites that could be incidentally impacted by the work are not within the project site, a more thorough raptor nesting site inventory study is planned to be conducted in the spring of 2019. In the prior 2015 investigation, Cliff swallow (*Petrochelidon pyrrhonota*) nests were observed under most bridges within the river reach of phase MI-5.

For Phase MI-5, impacted trees have been identified, and a preliminary determination of the mitigation requirement has been made consistent with the Construction Stage 1.5 Mitigation Plan prepared as part of the current USACE Final Section 404 Permit Submission (Appendix H3.1). The proposed tree impacts and preliminary determination of mitigation requirement can be seen in the maps in Appendix H3.2.

5.5 CULTURAL RESOURCES INVESTIGATION

Cultural resources investigations were conducted at the Project site to assess the historic and cultural resources in Phase MI-5 and to comply with Section 106 of the National Historic Preservation Act. The initial investigation included a Class I cultural resources inventory to search existing records for known archaeological sites and historic structures in the area of potential effect (APE) and 1-mile buffer zone. Based on the results of the Class I inventory, Class III cultural resources inventories were also designed and conducted to field inspect for cultural resources in the construction areas and assess potential impacts on any historic structures and archaeological sites identified in the APE. The cover pages of the reports completed as part of this initial evaluation are included in Appendices O4.3 – O4.6. Due to confidentiality requirements, the full reports are not included, however they are on file at the North Dakota State Historic Preservation Office (ND SHPO). A Programmatic Agreement (per 36 CFR 800.14 (b)) was executed between the USACE and the North Dakota State Historic Preservation Office (ND SHPO) to cover effects that could not be fully determined in advance. All future work on the project is subject to the terms of the Programmatic Agreement including Phase MI-5.

To cover additional MREFPP phases, including the Phase MI-5 project area, a Supplemental Class III Archaeological Investigation for the MREFPP CS 1.5[2] (Reference in Appendix O4.7) and a Supplemental Class III Architectural History Inventory for the MREFPP CS 1.5[3] (Reference in Appendix O4.8) have also been completed. The cover pages of the reports completed as part of this initial evaluation are included in Appendices O4.7 – O4.8. Due to confidentiality requirements, the full reports are not included; however, they are on file at the North Dakota State Historic Preservation Office (ND SHPO). The Supplemental Class III Archaeological Investigation and the Supplemental Class III Architectural History Inventory were completed to satisfy the requirements for Section 106 of the National Historic Preservation Act (NHPA), to obtain an inventory of the standing structures, and to evaluate the structures for the National Register of Historic Places (NRHP). The inventories, which were conducted through literature research and pedestrian survey, did not identify any cultural resources during field investigations. To determine eligibility for listing in the existing NRHP district, the Minot Industrial Historic District, three structures were recommended for further evaluation in the August 2017 reports included in Appendix O4.7 and O4.8. Subsequently, as part of the City's acquisition program described below, these three structures were further evaluated and determined to not be eligible for the NRHP District.

Parallel to the MREFPP development, the City of Minot has also been involved in flood recovery efforts since the 2011 flood of the Mouse River. The City has been actively engaged in the acquisition of properties using Community Development Block Grant (CDBG) funding through the US Department of Housing and Urban Development (HUD). In many cases, these acquisitions overlap with acquisitions required for the MREFPP.

The HUD and USACE programs both require review under the National Environmental Policy Act (NEPA) process and coordination with ND SHPO. As part of this effort the City of Minot, under its delegated authority from HUD, developed a memorandum of understanding (MOU) with ND SHPO, to expedite the environmental review process and provide the framework for identifying properties that required further coordination.

The HUD buyout program was reviewed and considered to be within a Categorical Exclusion under NEPA upon issuance of the Environmental Clearance to Release funds signed and dated September 24, 2012. As part of this review, ND SHPO was consulted on the cultural resources assessment and identified conditions for identifying historic properties, developing suitable mitigation, and concurrence with a finding of no significant impact for the buyout program. A copy of this information is included in Appendix O4.

As a result, the City will continue with acquisition and relocation of desired properties following the rules of the current HUD CDBG program and the MOU with ND SHPO. All remaining structures are anticipated to be removed by the City of Minot prior to this project. Copies of the MOU (City of Minot and ND SHPO) and the HUD environmental reviews are included in Appendix O4.

5.6 HAZARDOUS, TOXIC, AND RADIOACTIVE WASTE

An HTRW assessment has been completed in general conformance with ER 1156-2-132 U.S. Army Corps of Engineers Water Resources Policies and Authorities, (*HTRW Guidance for Civil Works Projects)* [110]. The purpose of the HTRW assessment was to identify issues and problems associated with waste in Phase MI-5. The assessment includes: 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 proposed Project area to identify land-use practices and potential sources of contamination. The HTRW report is included in Appendix O3.1 as part of the 90% design submittal.

The following environmental risks were identified as having the potential to affect Phase MI-5 of the Project:

- Numerous vehicle repair and storage facilities with observed ASTs with an unknown status for USTs.
- Former filling stations and dumpsites.
- Non-operational meth labs.
- Reported USTs identified as inactive.
- Spill reports.
- Hazardous waste handlers.
- Cleaned-up Superfund site.
- Documented environmental compliance activity issues.

As seen in Appendix O3.1, due to long-term industrial use, there are three properties with the potential for adverse environmental conditions recommended for additional assessment. A Phase II Environmental Site Assessment (ESA) is currently underway on these three properties and will be completed prior to the 100% submittal. Once completed, the results will be included as Appendix O3.2.

5.7 PRE-DEMOLITION INSPECTION

Currently it is anticipated that all homes and other structures will be removed by others prior to starting this phase of the MREFPP. However, should removal under this project be needed, inspections of any homes or structures to be demolished as part of this project phase 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. If hazardous materials are located, a report will be prepared to document hazardous materials identified during onsite inspections and specify procedures for proper management and disposal of the materials.

6 CIVIL DESIGN

6.1 CIVIL DESIGN FEATURES

Civil design focused on Phase MI-5 elements related to alignment and definition of Project features, feature geometry, vertical profiles, utility design, and corridor requirements. USACE standards and guidelines were used for the design development. Civil features include the following:

- Erosion Control
- Demolition and Corridor Preparation
- Flood Risk Management Features
    - Earthen Levees
    - Floodwall
    - Removable Closure Structure
    - Sheetpile Cutoff
    - Seepage Collection
    - Levee Access Ramp
- Municipal Utilities
- Franchise Utilities
- 4th Avenue NE Pump Station Layout, including Access, Grading, and Sitework and 4th Avenue NE Pump Discharge Gatewell and Detention Pond
- Slope Erosion Protection
- Roadway Modifications
- Traffic Control During Project Construction
- Railroad Modifications
- Recreational Facilities
- Landscape Design
- Restoration
- Borrow Area Selection and Design
- Earthwork
- Disposal Options
- Staging
- USACE Inspection Items

6.2 DESIGN CONSIDERATIONS

Civil design was completed to be consistent with the following constraints:

- MREFPP Design Guidelines Version 2.0.
- General conformance with the Preliminary Engineering Report (PER[31]) dated February 29, 2012.
- Accommodating the general configuration of river hydraulic analysis and interior drainage features as determined during the feasibility planning and prior Basis of Design Report planning.
- Considering geotechnical subsurface investigation and geotechnical modeling results.
- Attempting to limit, to the extent possible, property acquisitions beyond those proposed in the PER.
- Attempting to minimize environmental, social, and economic impacts of the project.

6.3 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 Storm Water 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 storm water discharges associated with land-disturbing activities.

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
- Slope Tracking
- Fiber Roll Staking
- Temporary and Permanent Vegetation
- Dewatering Controls

BMPs shown in the current construction drawings are intended to serve as a baseline level of implementation. During the work, the contractor shall perform inspections and monitoring to verify that BMPs are functioning correctly and are providing adequate functionality for construction phasing and scheduling. It is fully intended that the contractor will customize the baseline BMPs as work proceeds and, if needed, furnish and install additional BMPs to accomplish the requirements of the SWPPP. Such modifications to recommended BMPs shall be documented in the SWPPP by the contractor and be included on any required contractor submittals and/or work plans.

6.4 DEMOLITION AND CORRIDOR PREPARATION

Throughout the construction phase of this project, demolition and corridor preparation will be required. This includes full removal of roadway surfacing, franchise utilities, trees and other vegetation, parking lots, sidewalks and other miscellaneous facilities. In an effort to decrease costs, public utilities will be partially removed to a set depth below ground and the rest abandoned in place. Public utilities within 15 feet of the levee toe and in any excavations will be completely removed. Furthermore, a significant number of both commercial and residential structures will be removed; however, it is currently assumed that these will be removed by others in advance of the project. In addition, unsuitable and contaminated subsurface material will also need to be removed from the project corridor as set forth in the geotechnical or other ESA report recommendations.

6.4.1 REMOVALS

The estimated removal quantities have been measured and are included in the project opinion of probable costs in this submittal. Plan sheets showing the locations of these removals are in the Construction Drawings included in Appendix K of this submittal.

6.4.2 EXPLORATION TRENCH

Prior to placing the levee embankments, the levee corridor shall be cleared and grubbed, and all topsoil, previous fill, or other objectionable material will be stripped and stockpiled. In addition, an exploration trench shall be excavated, verified, and recompacted with appropriate embankment materials. The purpose of this trench is to verify that the corridor is clear of unknown utility penetrations. This excavation will be done in accordance with Section 7-2 of EM 1110-2-1913[96]. It was assumed that this exploration trench would have a depth of 10 feet below the stripping surface beneath all proposed levees. The trench section is proposed to be a trapezoidal section with a minimum bottom width of 4 feet and minimum side slope of 1:1 as shown below in Figure 6-1. These dimensions could be increased if necessary, for constructability.

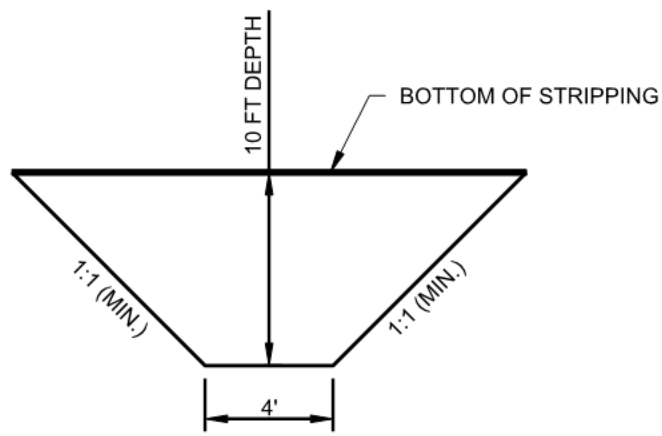


Figure 6-1 10-ft Exploration Trench

An exploration trench is not to be shown in the area of the floodwalls, removable closure, or sheetpile cutoff structures. Excavation for the construction of the footings and substructures for the floodwalls and removable closures will be used to verify that the corridor is clear of utility penetrations or undesirable materials. The depth of the sheetpile will also ensure adequate cutoff in the location of the sheetpile cutoff. The depths of these proposed excavations are shown in the Construction Drawings in Appendix K.

6.4.3 STRUCTURE DEMOLITION

Houses, sheds, and garages on properties where buyouts have been accepted are currently being removed by the City of Minot and/or Souris River Joint Board; demolition includes removal of foundations and known subsurface utilities. Where needed, removal of individual sanitary sewer and water service to the mainline will be completed as part of this project.

6.4.4 VEGETATION REMOVAL

Vegetation will be removed to the extents of the vegetation-free zone. The minimum width of the vegetation-free zone shall be the width of the levee and floodwall, 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 USACE *Engineering Technical Letter (ETL) 1110-2-583: Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures*[113].

6.4.4.1 VEGETATION-FREE ZONE

As stated in the previous section, it can be seen that *Engineering Technical Letter (ETL) 1110-2-583* [113] describes the vegetation-free zone as a “three-dimensional corridor surrounding all levees, floodwalls, embankment dams, and critical appurtenant structures in all flood damage reduction systems. The vegetation-free zone applies to all vegetation except grass.” Furthermore, “the primary purpose of the vegetation-free zone is to provide a reliable corridor of access to, and along, levees, floodwalls, embankment dams, and appurtenant structures. This corridor must be free of obstructions to assure adequate access by personnel and equipment for surveillance, inspection, maintenance, monitoring, and flood-fighting.” The minimum vegetation-free zone has been determined by the USACE based on lessons learned from flood-fighting experience.

Figure 6-2 and Figure 6-3 are a re-print of the vegetation-free zone as defined by the USACE.

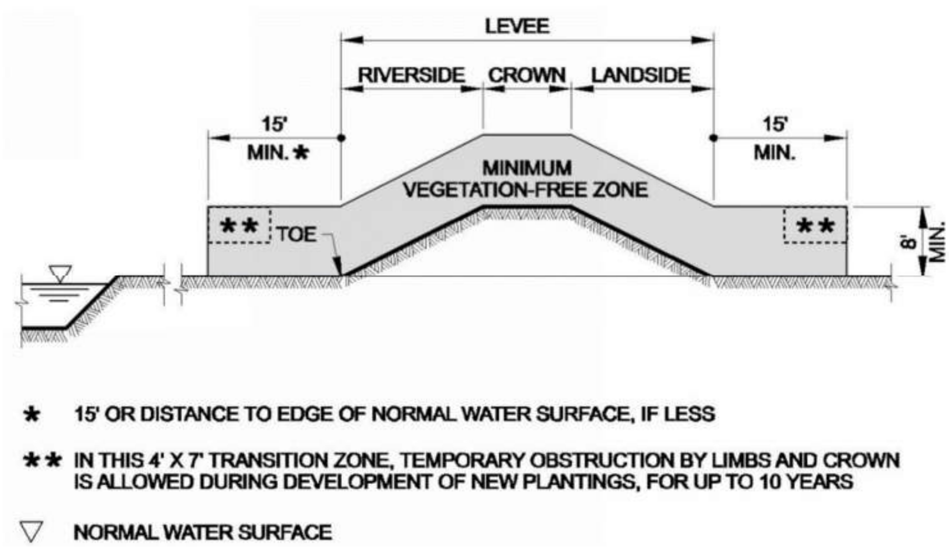


Figure 6-2 Vegetation-Free Zone

6.4.4.2 ROOT-FREE ZONE

Along with the vegetation-free zone specified by the USACE, a root-free zone is also to be established. “Planting design must consider the possible implications to foundation strength and performance. The integrity of the foundation could be compromised if potential seepage paths were created by root penetration and/or root decay. The root-free zone provides a margin of safety between the greatest expected extent of plant roots and the beginning face of any structure that is critical to the performance and reliability of the flood damage reduction system.” Critical structures as defined by the USACE, which are a part of Phase MI-5, include levees and floodwalls.

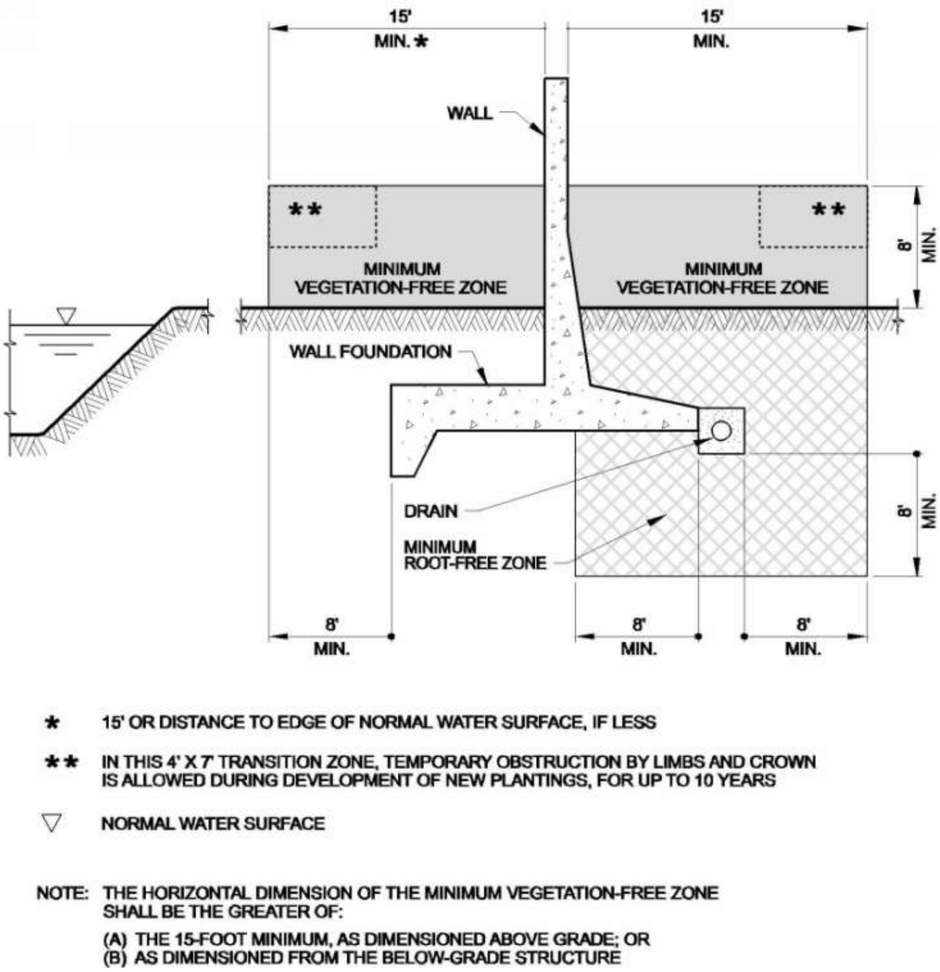


Figure 6-3 Vegetation-Free Zone: Inverted-T Type Floodwall with Drain

6.4.5 STREET AND UTILITY DEMOLITION

Exiting streets and utilities will be removed as indicated in the Construction Drawings in Appendix K. Roadway pavements and base aggregates will be removed down to suitable subsoils. Public utilities will be removed and/or abandoned as shown in the Construction Drawings in Appendix K.

6.4.6 LEVEE REMOVAL

The design assumes complete removal of existing levees and/or other berms within the reach of Phase MI-5. The construction drawings define the extent of existing levee removal in plan view and cross sections. To the greatest extent possible, existing levee/berm removal will be sequenced to maintain the existing level of flood risk management for Minot during construction. The Contractor shall sequence demolition of the levee 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.

6.5 FLOOD RISK MANAGEMENT FEATURES

The flood risk management features for the Phase MI-5 area will include a combination of floodwalls, earthen levees, sheetpile cutoff and a removable closure in the floodwall just east of the Mouse River on the western side of the project. Additional details of these flood risk management features are included in the following sections.

6.5.1 HORIZONTAL ALIGNMENT

Permanent flood risk management features for Phase MI-5 will be constructed along a horizontal alignment in which stationing increases in the downstream direction. The horizontal alignment location represents the centerline of the floodwall, closure structure, or levee crest.

The flood protection alignment is on the north (left) side of the river and extends along the Mouse River from the termination of the Phase MI-1 Project just east of 3rd Street NE (the upstream boundary) to just east of 13th Street NE (the downstream boundary). The alignments of this flood protection are shown in Section C-300 of the Construction Drawings included in Appendix K. The following sections identify elements affecting the horizontal alignment of the flood risk management system for Phase MI-5.

6.5.1.1 REAL ESTATE ACQUISITIONS

Minimizing the need of additional property acquisition was a significant consideration in determining flood risk management features horizontal alignments. Property acquisitions were primarily defined by alignments developed as part of the PER. Revisions to these preliminary acquisition estimates were made to account for the size/location of the primary features included in this submittal. The City of Minot and SRJB continues to acquire properties needed to construct the Project. Additional details of the Real Estate needs are included in Section 12.

6.5.1.2 INTERIM TIEBACK LEVEE

The interim tieback levee located south of 4th Avenue NE near station 74+00 will be constructed to maintain at least the existing level of flood risk management based on conditions that will exist in the interim until connecting future phases of the MREFPP are completed. Figure 6-4 shows the location of this interim tieback levee.

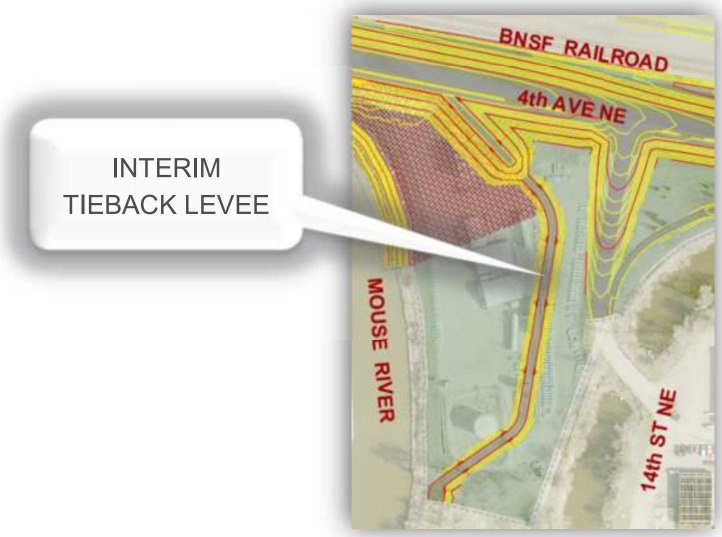


Figure 6-4 Interim Tieback Levee

6.5.1.3 GEOTECHNICAL CONSIDERATIONS

Geotechnical field investigations are being completed by Braun Intertec, which includes soil borings, CPT testing, and other soil samplings. The subsurface investigation data is included in Appendix B, and an overview is provided in Section 2. As part of the initial alignment review, Braun identified critical cross sections that had the potential to affect the alignment. Identified cross sections were then analyzed for slope stability to determine potential impacts to the initial alignment. Recommendations from this report have been included in the current proposed alignment.

6.5.2 VERTICAL ALIGNMENT

The minimum vertical alignment/elevation for the top of the flood risk management features was determined from hydraulic modeling based on the 27,400 cfs design event plus 4 feet of added height to account for settlement overbuild, superiority overbuild, and hydraulic uncertainty, as discussed in Section 1.7. The design provides additional elevation above the design water-surface elevation based on the 2011 flood discharge to account for risk and uncertainty, superiority overbuild and settlement. One foot of overbuild on the sections of levees was added to account for long-term settlement. Additional discussion regarding settlement is included in Section 2.

The Floodwall section (Sta. 41+28 to Sta. 46+95) is currently designed to a vertical elevation that matches the Phase MI-1 top of floodwall elevation (1566.50). The levee section from Sta 46+90 to Sta. 74+07 is designed with the full build-out of the MREFPP Construction Stage 4 design flood event of 27,400 cfs plus 4 feet (1566.05). Near station 74+07, a flood protection levee was stubbed out in the southeasterly direction to provide a tie-in point for a future phase of the MREFPP that will be constructed as part of Construction Stage 2. These future phases were presented in more detail in the Final Programmatic Environmental Impact Statement – MREFPP included as Appendix O1.1 of this report.

As noted previously, the easterly end of Phase MI-5 (east of station 74+07) is intended to provide semi-permanent flood protection by providing a tie-in to high ground on the east end. This is considered semi-permanent because ultimately future phases of the MREFPP will connect to Phase MI-5 at station 74+07 and continue full-height protection along the river on both sides. As a result, beginning at station 74+07 the top of proposed protection was reduced due to the semi-permanent nature of flood protection to be provided by this segment. The lower design elevation is based on the Construction Stage 1.5 design flood event of 27,400 cfs plus hydraulic uncertainty for the sheet pile cut off (1558.75), and an additional 1-foot for settlement for the remainder of the flood protection levee (1559.75).

The interim tieback levee described in Section 6.5.1.2 will be constructed to an elevation of 1555.45, which ties in to an existing flood protection levee adjacent to the Mouse River.

The top of floodwall elevations and vertical alignment is also discussed further in Section 7, Structural Design.

6.5.3 EARTHEN LEVEES

The design of the levee sections was completed using the USACE design criteria set forth in USACE *EM 1110-2-1913- Design and Construction of Levees*. Seepage, slope stability and settlement design information was based on information obtained from the site-specific geotechnical investigation.

Approximately 5,400 linear feet of levee is proposed for MI-5. The maximum height of the levee plus freeboard is 18 feet. At approximately Station 74+00, the Interim Tieback Levee extends south approximately 550 feet. Additional details of the earthen levee designs are shown in the Construction Drawings included in Appendix K.

Floodwalls will be tied in to levees by extending the floodwall a minimum of 5 feet into the full levee section. Sheet pile should be extended a minimum of 20’ or the height of the floodwall, whichever is greater, into the levee beyond the end of wall concrete. Within the 5-foot transition period, the sheet pile will be extended to within 1’-6” of the levee crown elevation. Details of these connections are shown in the Construction Drawings in Appendix K.

6.5.3.1 LEVEE CROSS SECTION

The majority of the proposed levees will have a 12-foot top width. The top-of-levee cross slope for levees is shown as a 2.0% grade from landside to riverside to maintain positive drainage of the levee crest, except in areas where the levee is within 4th Avenue NE where it is flat. Proposed levee side slopes are shown as 3H:1V due to maintenance requirements. As stated in EM 1110-2-1913 [96], a 3H:1V slope is typically the steepest slope that can be conveniently mowed and walked on during inspections. A typical section of a levee is shown in Figure 6-5.

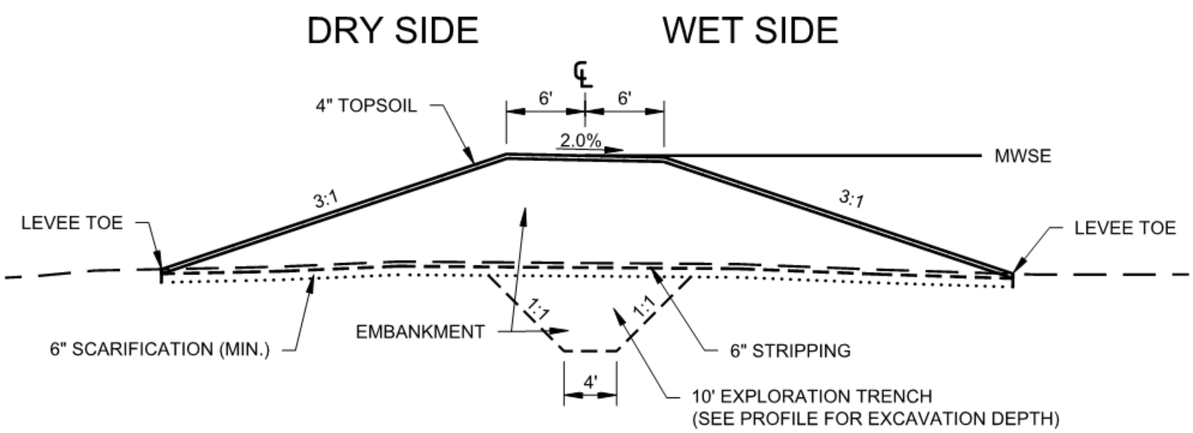


Figure 6-5 Typical Levee Section

The levee section on the eastern side of the project is designed to have a paved road on top. The pavement design is discussed in Appendix E5.2. A typical section of the roadway above the levee is shown in Figure 6-6.

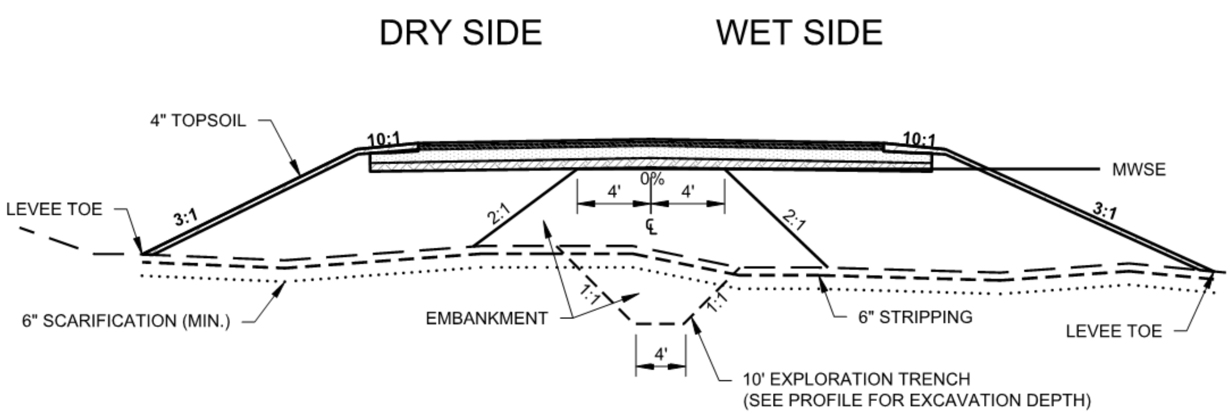


Figure 6-6 Typical Roadway above Levee

6.5.3.2 LEVEE CONSTRUCTION AND MATERIAL

The levee will be constructed following guidance outlined in EM1110-2-1913 [96]. Clearing and grubbing will be done within the levee corridor, including the removal of all vegetation, roots, stumps, etc., as discussed earlier in this section. Existing topsoil will be stripped from native ground or from the existing levee, followed by scarification to prevent surface compaction planes. An exploration trench will be excavated to expose or intercept any undesirable underground features.

The levees will be constructed with impervious material, placed in specified lifts, and compacted according to the technical specifications. Material will be acquired mainly from designated borrow areas described in Section 6.15. Impervious fill material will meet specified gradations and be clearly described in the technical specifications.

Topsoil will be installed on all slopes to a minimum 4-inch thickness as 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.

6.5.4 FLOODWALL

Reinforced concrete floodwalls will be inverted-T type walls. The walls will extend from Sta. 41+28 to 45+06 and from 46+69 to 46+95. Information regarding the structural floodwall design and cross sections are provided in Section 7, Structural Design.

6.5.5 REMOVABLE CLOSURE STRUCTURE

A removable closure structure will be constructed on the north side of the river across the existing BNSF railroad lines just south of the proposed intersection of 5th Avenue NE and Railway Avenue NE between Sta. 45+06 and Sta. 46+69 (centerline of column stationing on either end). The location of the removable closure structure is shown below in Figure 6-7.

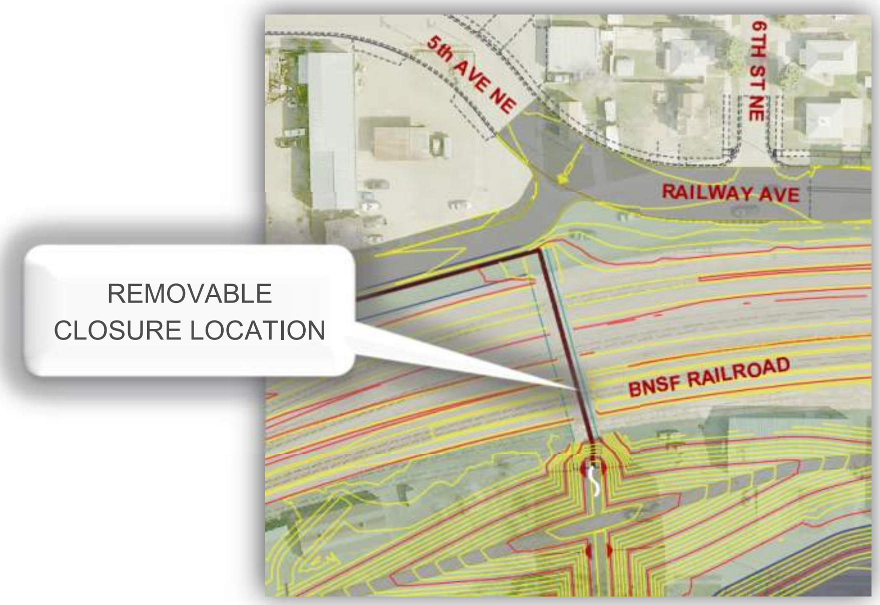


Figure 6-7 Removable Closure Location

The location of this closure will allow for closure of the BNSF railroad during times of flooding. The removable closure structure will consist of aluminum planks that are erected before a flood event occurs. After the flood, the closure panels would be removed and stored. The removable closure will be approximately 159.50 feet long, with the northern portion approximately 9 feet tall and the southern portion approximately 7.5 feet tall. The northern portion of the sill for this closure will be at 1557.47, while for the southernmost 49.3 feet it will be at 1559.03. These elevations are both above the 100-yr flood level of elevation 1554.36 as defined in the Ward County Preliminary FIS. The top elevation will be at 1566.50. Figure 6-8 below shows an elevation view of the proposed removable closure structure.

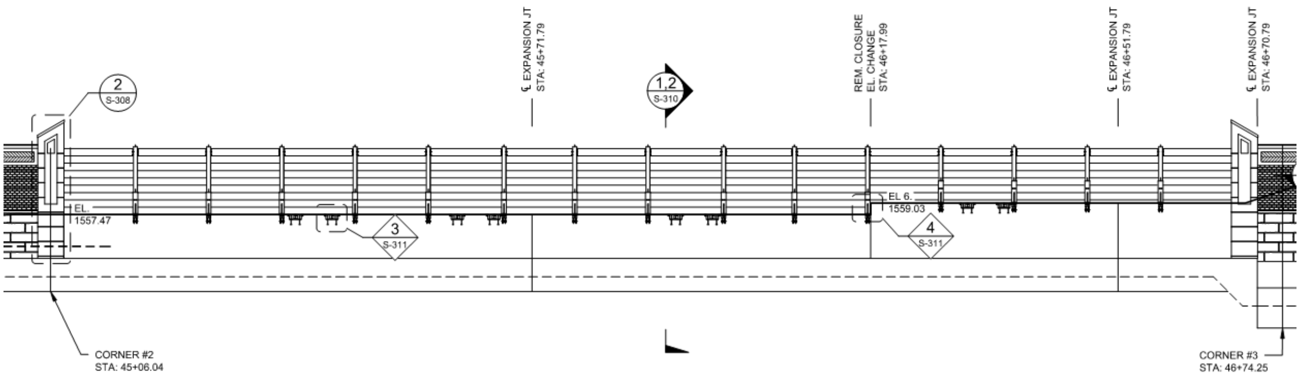


Figure 6-8 Removable Closure Elevation

Changes to the removable closure from the 60% design submittal include the following:

- Horizontal Location
- Flood Protection Threshold
- Closure Geometry

The horizontal location of the proposed removable closure was moved approximately 350 feet to the east of the crossing location that was detailed in the 60% Phase MI-5 submittal. Shifting the location further to the east did result in the need to cross another BNSF track, but also provided the opportunity to be installed at a higher threshold elevation.

In the 60% design submittal, the railroad closure structure was designed to a single threshold elevation, while in the 90% submittal, the southernmost railroad mainline is protected to a higher level than the second mainline and adjacent siding lines. This option was selected in order to allow rail traffic to remain operational as long as possible prior to deploying the floodwall planks for full closure.

Additional design discussion regarding this structure is included in Section 7, Structural Design, as well as adjacent track design and modifications in Section 6.11 Railroad Modifications. Drawings are included in Appendix K.

6.5.6 SHEETPILE CUTOFF

Sheetpile cutoff will be installed across the BNSF railroad where the semi-permanent segment of 4th Avenue NE Tieback Levee turns and extends north. The sheetpile will span from approximately Sta. 85+17 to Sta. 88+79. The top elevation of the installed sheetpile is based on the Construction Stage 1.5 design flood event of 27,400 cfs plus hydraulic uncertainty for an elevation of 1558.75. The bottom elevation of the sheetpile will be approximately 1530.00. Additional details of the sheetpile cutoff design are shown in Figure 6-9 below and in the Construction Drawings included in Appendix K.

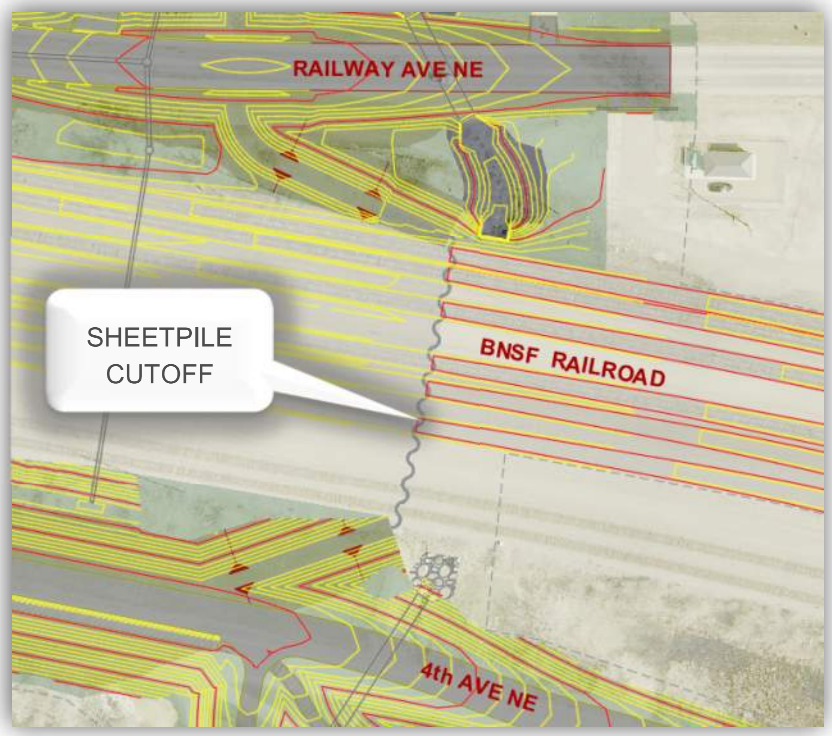


Figure 6-9 Sheetpile Cutoff

6.5.7 SEEPAGE COLLECTION

A seepage collection system is proposed along the levee landside to intercept seepage during flooding and provide improved geotechnical stability near the proposed stormwater pond. The seepage collection system is designed in accordance with EM 1110-2-1913[96]. Figure 6-10 shows the typical seepage collection pipe proposed on the landside of the levee with additional details included in Appendix K.

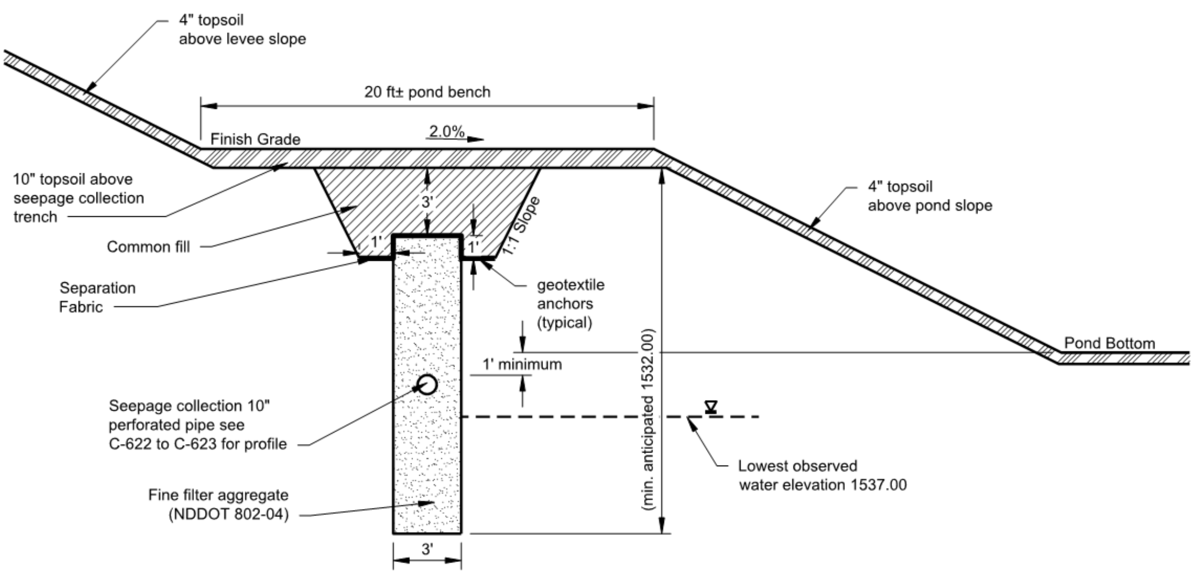


Figure 6-10 Seepage Collection Trench

A backflow preventer (flap gate) will be installed at the connection to STS 2 to prevent surcharging of the seepage collection system during storm sewer flooding conditions.

6.5.8 LEVEE ACCESS RAMP

Just to the west of the 4th Avenue NE Detention Pond, a shared-use path up and over ramp will be constructed. This ramp will allow for a 10-foot pedestrian path to be constructed, which also will serve as levee access for maintenance. The access ramp can be seen in Figure 6-11. Additional details of the access ramp design are shown in the Construction Drawings included in Appendix K.

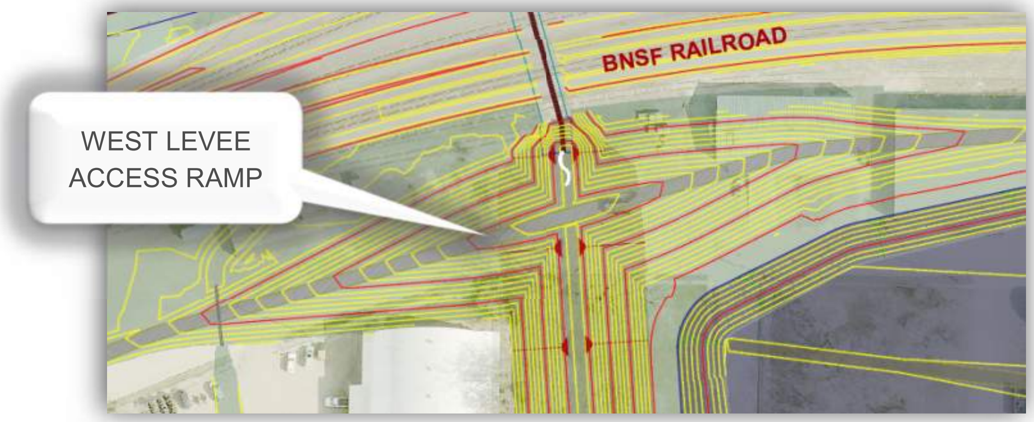


Figure 6-11 Levee Access Ramp

6.6 MUNICIPAL UTILITIES

Municipal utility modifications are required due to flood protection construction and roadway modifications. Affected utilities will include sanitary sewer, watermain, and storm sewer. Utilities crossing through the flood risk management feature will be upgraded to USACE standards.

Existing utility types and locations have been defined by survey, as-built records, and GIS resource information supplied by Minot and onsite data collection of visible surface items.

Water, storm sewer, and sanitary sewer utilities have been designed in accordance with *City of Minot Standard Specifications and Details–2013* [35] and the Great Lakes-Upper Mississippi River Board's *Ten States Standards for Water and Wastewater Facilities* [43][44].

Where practical, utilities are located within defined right-of-way corridors. As a part of the utility modifications associated with this project, portions of the existing utility networks will be relocated through BNSF property. Utility easements will be acquired in these areas to accommodate access and future maintenance and repairs. In addition, water, storm sewer, and sanitary sewer terminations and mains paralleling the dry side of the flood risk management features have been placed horizontally no closer than 15 feet from the toe of the proposed levee or the footing of the floodwall.

6.6.1 WATER

As part of the project, a significant portion of the watermain in the vicinity of the project will be replaced or relocated. Much of it will run parallel with and under the proposed Railway Avenue NE. The watermain for the most part will be placed on the north side of Railway Avenue NE. The proposed 16” watermain will tie in to existing watermain east of 6th Street NE, and continue east and connect the existing Railway Booster Pump Station located north of the BNSF railroad, and south of Railway Avenue NE near the eastern end of the project.

Before tying in to the Railway Booster Pump Station, the 16” PVC water line will pass through the proposed line of protection at approximately Station 90+29. At this location, the water line has been designed to meet USACE standards. The utility will be located and constructed to minimize the risk of pipe leaking, rupture, and other failures that could negatively impact the line of protection. In addition, gate valves will be placed on both the dry side and the wet side of the line of protection, in order to provide isolation from the riverside in the event of a line failure. Also, the watermain will be installed with no gravel bedding or backfill under the proposed levee and within 15’ of the toe of the line of protection.

Existing watermains which currently run north on 7th Street NE, 9th Street NE, 10th Street NE, 11th Street NE, 12th Street NE and 13th Street NE will be shortened to tie in to the new proposed 16” alignment along Railway Avenue.

Additionally, an 8” watermain will be reconstructed to the north from the intersection of Railway Avenue and 8th Street NE, where it will tie in to existing watermain near the intersection of 8th Street NE and 7th Avenue NE.

An 8” PVC watermain will also be extended beneath the BNSF railroad to the 4th Avenue NE Stormwater Pump Station to provide water service to the proposed pump station as well as a fire hydrant for fire protection. This line will be placed through an existing casing pipe the City of Minot installed for a future crossing that will no longer be needed.

A watermain river crossing will be relocated as part of Phase MI-5 on the western edge of the project near 3rd Street SE. To provide this crossing, the Phase MI-5 portion of this 16” watermain will connect on the wet side of the MI-1 floodwall to a 16” watermain that was included in MI-1 as a flood protection crossing. From there it will cross under both the BNSF railroad and Mouse River and tie in to an existing 14” watermain just southwest of the existing BNSF bridge over the Mouse River.

Horizontal alignments were selected to ensure the 10 States Standard minimum horizontal separation between watermain and other utilities – i.e., 10 feet from outside edge of pipe to outside edge of pipe between water and sewer – was upheld.

There is currently an abandoned 48” raw water line from the City of Minot Sundre well fields located within the project extents. This has been abandoned by others outside the project limits under a separate contract; prior to beginning construction of this project. The water line will need to be removed from the project limits as part of this contract.

For more specific information on the watermains see the Construction Drawings in Appendix K.

6.6.2 SANITARY SEWER

The project proposes modifications to both gravity sanitary sewer and forcemains. With the reconstruction of Railway Avenue NE and 8th Street NE, all gravity sanitary sewer under these roadways will be replaced. This phase will provide a gravity collector that conveys sanitary discharge south under 8th Street NE to its intersection with Railway Avenue NE. From this location, the new collector will convey sewage westward toward the 4th Avenue Sanitary Lift Station constructed as part of the MREFPP Phase MI-1 project. The 4th Avenue Sanitary Lift Station then pumps into the existing Valley Forcemain, which has a large sewershed.

The existing Valley Forcemain conveys sewage under pressure via a 24” ductile iron forcemain through the MI-5 project area from west to east and discharges to gravity east of the project extents. The existing Valley Forcemain begins on the north side of the BNSF railroad and crosses to the south side, west of the pedestrian bridge over the railroad. As a result, the existing Valley Forcemain conflicts with the proposed flood protection alignment and must be relocated. The proposed new 24” ductile iron forcemain will be constructed under the reconstructed Railway Avenue NE and will cross the BNSF railroad outside the eastern extent of the MI-5 project. As part of this relocation, the existing 15” PVC Roosevelt Lift Station Forcemain will also need to be relocated, utilizing a new 14” ductile iron forcemain to tie into the relocated Valley forcemain. The connection to the Valley Forcemain will be relocated to the north side of the BNSF railroad. This will require an additional crossing of the BNSF railroad near 14th Street NE. Railroad crossings will be completed by jacking and boring a steel casing pipe. The relocated Valley Forcemain will cross the line of protection at approximately Station 77+64 and Station 89+95. At the crossing, one plug valve will be installed on each side of the line of protection. There will be no gravel bedding or backfill material placed between these valves during the installation of the forcemain. Due to the lack of aggregate bedding, the depth required to cross below the levee inspection trench, and the height of levee fill, the pipe crossing the levee will be thickness class 56 ductile iron pipe.

Due to the large sewersheds, both the Valley Forcemain and Roosevelt Lift Station Forcemain will remain in service at all times. This will be accomplished using temporary forcemain bypasses. Additional details of the Valley Forcemain relocation are provided in Appendix E1.1.1 and also in the Construction Drawings in Appendix K. Prior to the 90% BDR submittal, a hydraulic analysis was conducted on the Valley Forcemain that evaluates the impacts to existing pumping facilities outside of the MI-5 project area as a result of the proposed forcemain modifications. The results of this analysis are provided in Appendix E1.1.3.

In addition, a portion of the existing gravity sanitary line will be relocated between 12th and 13th Avenues along Railway Avenue to provide adequate separation from the relocated Valley Forcemain and adjacent waterline.

All sanitary sewer impacts and proposed relocations are shown in the 90% Construction Drawings in Appendix K.

6.6.3 STORM SEWER

Storm sewer, consisting mostly of reinforced-concrete pipe (RCP) and reinforced concrete box culvert (RCBC), will be used to convey runoff within the project site. Details of the proposed storm sewer collection system and interior drainage facility, including hydraulic design criteria, are provided in Section 4 as well as the 90% Construction Drawings in Appendix K.

In addition to the interior collection system, an 11’x5’ RCB will be installed under a portion of Railway Avenue to maintain conveyance outside the semi-permanent line of protection to the existing dual 11’x9’ RCB that currently cross beneath the BNSF railroad. Downstream of the existing railroad crossing, an 8’x6’ RCB will be installed to provide conveyance to the Roosevelt Park Loop from the upper reaches of the Roosevelt Park Watershed in the Minot International Airport.

Civil design of the storm sewer lines includes consideration of the following elements:

- Pipe Loading
- Pipe Class
- Pipe Bedding and Foundation

Design and construction will be in accordance with Chapter 8 of EM 1110-2-1913 and USACE *EM-1110-2-2902: Conduits, Culverts, and Pipes* [105].

Pipe used in levees is required to meet the American Water Works Association C302 or ASTM C76 standards. Additionally, pipe loading calculations following Section 3-7 of EM 1110-2-2902 [105] have been reviewed for two conditions: (1) RCP within the levee footprint and (2) RCP in non-levee areas. The design approach for both conditions is based on American Association of State Highway and Transportation Officials (AASHTO)[7], with exceptions as stated in EM 1110-2-2902 [105]. Design computations using the D-Load analysis based on a D0.01 crack are provided in Appendix E1.2.1.

Pipe bedding and foundation design is based on EM 1110-2-2902 [105] for trench installations. Pipes located in city road right-of-way will be bedded to be consistent with City of Minot standards. Impervious material is required for pipe placed within the levee to minimize seepage.

As part of the storm sewer system, near the center of the project, a 72” RCP flowing toward the 4th Avenue NE Pump Station is proposed to be installed beneath the BNSF railroad. This pipe conveys the majority of runoff from the interior collection area to the proposed interior drainage facility. Also, a 30” CMP is proposed to be installed beneath the BNSF railroad on the east end of the project to connect existing storm sewer within the railroad to the proposed interior drainage facility. For more specific information on the storm sewer see the Construction Drawings in Appendix K. The phasing plan can also be seen in Appendix K on sheets C-103 through C-107.

6.6.4 UTILITY PENETRATION

The proposed flood protection utility crossings have been limited to as few as possible. An evaluation of each levee utility crossing was completed in accordance with FEMA Publication Number 484, *Technical Manual: Conduits through Embankment Dams.* Guidance on the techniques used for the design of levee penetrations is provided in Chapter 8 of EM 1110-2-1913 [96].

Utility-penetration locations for public utilities within the flood protection footprint are shown in Table 6-1. Station locations are approximate.

Table 6-1 Proposed Public Utility - Penetration Locations

Flood Protection Station	Utility Name	Utility Size	Proposed Outcome	Elevation
61+52	4th Avenue NE Pump Discharge Gatewell	8’ x 8’ Box Culvert	New	1540.87 (Invert)
60+95	4th Avenue NE Pump Station Discharge Piping	24” DIP	New	1565.00 (Top)
61+05	4th Avenue NE Pump Station Discharge Piping	24” DIP	New	1565.00 (Top)
61+14	4th Avenue NE Pump Station Discharge Piping	24” DIP	New	1565.00 (Top)
61+17	4th Avenue NE Pump Station Discharge Piping	6” DIP	New	1563.50 (Top)
77+64	Sanitary Sewer Forcemain	14” DIP	New	1544.70 (Top)
89+95	Sanitary Sewer Forcemain	24” DIP	New	1533.27 (Top)
90+29	Watermain	16” PVC	New	1533.50 (Top)

Utility penetrations through the flood protection have been minimized to reduce the risk of seepage, pipe line leakage, or other negative impacts. All pipes penetrating the levee will be provided with 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 safety valve locations are given in Table 6-2.

Table 6-2 Utility Safety Valve Locations

Flood Protection Station	Utility Name	Safety Valve Landside	Safety Valve Riverside
77+64	Sanitary Sewer Forcemain	14” Plug Valve 348’ Lt	14” Plug Valve 61’ Rt
89+95	Sanitary Sewer Forcemain	24” Plug Valve 51’ Lt	24” Plug Valve 58’ Rt
90+29	Watermain	16” Gate Valve 50’ Lt	16” Gate Valve 50’ Rt

6.7 FRANCHISE UTILITIES

6.7.1 MONTANA-DAKOTA UTILITIES

Removal of several Montana-Dakota Utilities (MDU) gas lines will be required as part of the project. MDU is planning to completely remove all lines throughout the project area located south of the BNSF railroad between the BNSF railroad and the Mouse River. No MDU penetrations are anticipated through the line of protection.

Additional relocation information is presented in the memorandum and relocation plans provided in Appendix E2.

6.7.2 MIDCO

Midco is planning for a complete removal of the Midco network located throughout the project area located south of the BNSF railroad between the BNSF railroad and the Mouse River. No Midco penetrations are anticipated through the line of flood protection.

Additional relocation information is presented in the memorandum and relocation plans provided in Appendix E2.

6.7.3 SOURIS RIVER TELEPHONE

Souris River Telephone (SRT) has a 144 ct fiber line that will be required to be relocated to the north side of the tracks between 8th Street NE and 13th Street NE to accommodate the new project alignment. SRT will also have a penetration through the line of protection along the north side of Railway Avenue on the east end of the project; however, the lines will be installed in an overbuilt area above the project design elevation.

Similar to the other franchise utilities, all services will be removed throughout the project area to the south of the BNSF railroad between the BNSF railroad and the Mouse River.

Additional relocation information is presented in the memorandum and relocation plans provided in Appendix E2.

6.7.4 XCEL ENERGY

Similar to the other franchise utilities, Xcel will remove all utilities within the project area south of the BNSF railroad between the BNSF railroad and the Mouse River.

Currently Xcel feeder lines branch just south of BNSF right-of-way to run along 6th Avenue NE and 8th Avenue NE. Prior to construction of the MI-5 project, Xcel will extend the feeder lines to 2nd Avenue NE before splitting them to tie in to 6th Avenue NE and 8th Avenue NE again.

Similarly to SRT, Xcel anticipates a penetration through the flood protection crossing Railway Avenue. It is currently anticipated that this will be direct buried along north side of Railway Avenue on the east end of the project; however, the lines will be installed in an overbuilt area above the project design elevation.

Additional relocation information is presented in the memorandum and relocation plans provided in Appendix E2.

6.8 4TH AVENUE NE PUMP STATION AND POND

The proposed system will consist of the storm water collection system (storm sewer/box culvert), the 4th Ave NE Pump Station, the 4th Ave NE Detention Pond, a gravity bypass pipe, a gatewell structure, and an outfall to the Mouse River. Skid steer access will be provided to the outfall pipe, gatewell structure and gravity bypass pipe for maintenance purposes from the downstream end of the outfall pipe.

A detailed interior drainage analysis was conducted to determine the required capacity of the interior drainage facility features. Results from this analysis are provided in Section 4 and Appendix D.

6.8.1 DESIGN CONSTRAINTS

The site layout was mainly constrained by:

- Providing adequate space for ponding to reduce required pumping capacity.
- Placement of the pump station and gatewell in relation to the roadway embankment of the reconstructed levee/roadway.
- Providing conveyance from the north side of the BNSF Railway to the interior drainage facility.
- Geotechnical modeling and stability required for the proposed levee alignment and pond.
- The desire to maintain gravity discharge above the normal river elevation.

6.8.2 HYDRAULIC SYSTEM DESCRIPTION

Storm runoff will be collected in the MI-5 watershed by the City of Minot storm sewer collection system. Once collected, this runoff will be conveyed to the pump station site via a proposed 72” storm sewer pipe under the BNSF Railway. A junction manhole (STS 2) upstream of the pump station will allow runoff to be conveyed toward the 4th Avenue NE Pump Station, the detention pond, and the gravity bypass pipe depending on river levels and interior water levels.

The invert elevation of the pipe under BNSF Railway will be below normal river water elevation even during times of low river stage. As a result, runoff from minor storm events will be conveyed directly to the 4th Ave NE Pump Station, where it will be pumped to the gatewell and allowed to outfall to the river via a reinforced concrete box culvert and open channel. When low river tailwater elevations exist but storm discharge from the interior area exceeds the pump rate of the 4th Avenue NE Pump Station, the water surface elevation will rise in the collection system until it is above the pond and gravity bypass pipe invert elevations, at which point storm discharge will also be conveyed both to the pond and through the gravity bypass pipe. Runoff conveyed through the gravity bypass will pass through the gatewell and ultimately discharge in the river.

During times of riverine flooding the gatewell gates will be closed. At this time, all runoff from the interior area will be stored in the detention pond and collection system until it can be pumped over the line of protection. Table 6-3 provides a summary of the hydraulic system elements. Descriptions of each element are provided in the following sections.

Table 6-3 Hydraulic System Elements

Feature	Description or Value
Watershed Details	
Watershed Area	160 Acres
River Elevation Details	
Low River Level (Roosevelt Park Control Structure Invert)	1539.95
Low River Flowrate	0 cfs
Average River Level	1540.3
Average River Flowrate	181 cfs
Design Event	2011 flood event (with MREFPP)
Design Flood Water Level	1561.8
Design Flood Flowrate	27,400 cfs
Gravity Bypass Invert at Pump Discharge Gatewell	1540.87
Pump Station Details	
Station Type	Rectangular Wet Well
Station Design Flowrate	20,000 gpm
Number of Pumps	4 (3 stormwater/flood control, 1 dewatering)
Stormwater/Flood Control Pump Type	Centrifugal Submersible
Flood Control Pump Discharge Capacity	10,000 gpm
Flood Control Pump HP	125 hp
Flood Control Pump Drive Type	Constant Speed, RVSS
Flood Control Pump Head	29.2’
Flood Control Pump Discharge Pipe Diameter	24”
Flood Control Pump Off Elevation	1534.5’
Pump Station Operating Floor Elevation	1555.5
Overall Wetwell Length (Internal)	56.5’
Overall Wetwell Width (Internal)	26’
Dewatering Pump Discharge Capacity	700 gpm
Dewatering Pump Head	41.1
Dewatering Pump Discharge Pipe Diameter	6”
Dewatering Pump Type	Centrifugal Submersible
Pump Discharge Gatewell Details	
Gravity Bypass Pipe Size (from collection system)	8’ x 8’ RCB
Gravity Bypass Invert Elevation (from collection system)	1541.2
Primary Gate	8’ x 8’ Combination Gate

Feature	Description or Value
Primary Gate Invert Elevation	1540.7
Primary Gate Location	River Side of Center Dividing Wall
Secondary Gate	8’ x 8’ Slide Gate
Secondary Gate Invert Elevation	1540.7
Secondary Gate Location	Upstream Side of Center Dividing Wall
Gravity Outlet Pipe Size	8’ x 8’ RCB
Gravity Outlet Pipe Invert Elevation at Structure	1540.6
Gravity Outlet Pipe Invert at River	1540.5
All Discharge Pipe Invert Elevations	1562
Flood Control Pump Discharge Pipe Gates	3 – 24” Flange Mounted Flap Gates
Dewatering Pump Discharge Pipe Gate	1 – 6” Flange Mounted Flap Gate
Pond Details	
Invert Elevation	1539.8
Peak Water Surface Elevation (100-yr gravity)	1545.3
Area at Pond Bottom	2.4 Acres
Area at Peak Water Surface (100-yr gravity)	3.1 Acres
Volume at Peak Water Surface (100-yr gravity)	11.1 AC-FT

6.8.3 DETENTION POND DESIGN

The depth of the pond was limited by groundwater. The area of the pond was determined by the amount of runoff storage volume required to reduce the risk of interior flooding. The side slopes of the pond were determined by geotechnical evaluation of slope stability and will be 4:1 (H:V). Geotechnical evaluation results are provided in Appendix B. A dry pond is desired by the owner for maintenance purposes. The bottom of the pond will be graded at 1.5% from the edges to the center. A 10’-wide, 6”-deep concrete channel liner will be constructed along the center of the pond. The channel liner will have a longitudinal slope of 0.1% from the western edge of the pond to the outlet pipe on the east edge of the pond.

As an additional precaution, drain tile will be installed longitudinally along the pond bottom at maximum 60’ spacing intervals and 3’ depth. This arrangement allows for a drainage coefficient of approximately 0.3 in/day. Details of the drain tile design are included in Appendix D.

6.8.4 MECHANICAL AND PROCESS DESIGN

6.8.4.1 4TH AVENUE NE PUMP STATION COLLECTION SYSTEM

The collection system consists primarily of existing storm sewer upstream of Railway Avenue. Storm runoff is collected by inlets in low areas and streets in the collection area and is routed to Railway Avenue via existing storm sewer. A new trunk storm sewer is proposed under Railway Avenue to convey runoff to a point near the intersection of Railway Avenue and 9th Street NE. From this point, a 72” RCP will be installed under the BNSF Railway to a structure upstream of the pump station (STS 2). A 10’ x 8’ RCB will connect STS 2 to the pump station. There will also be a 72” RCP connection to the detention pond, and an 8’ x 8’ RCB connection to the gravity bypass (STS 1).

6.8.4.2 GRAVITY BYPASS

The invert of the collection system pipe south of the BNSF Railway is approximately 1535’. The low river elevation near the interior drainage facility is controlled by the USACE control structure at the Roosevelt

Park dead loop, which has an invert elevation of approximately 1540'. If a gravity discharge pipe were installed at the same elevation as the collection system, the outfall pipe as well as a large portion of the upstream storm sewer collection system would be below normal river elevation and would be perpetually inundated with sediment laden river water. To reduce potential sedimentation and maintenance requirements, the gravity pipe will be installed above the normal river elevation. Interior runoff will only be conveyed through the gravity bypass pipe when the water surface in the junction structure upstream of the pump station exceeds 1541.2'. Analysis indicates that a 6' x 4' RCB is sufficient to convey adequate flow to prevent interior flooding for a 100-year gravity event. However, the project sponsors have requested that the size of the pipe be increased to 8' x 8' RCB to allow the use of a skid steer for maintenance and cleaning.

6.8.4.3 4TH AVENUE NE STORMWATER PUMP STATION

Since gravity discharge will only occur when the upstream water surface exceeds 1541.2', the pump station will be relied upon to pump all low flows during gravity periods as well as all storm runoff during blocked gravity periods. The pump station invert will be approximately 1528'. This elevation will allow the stormwater pumps to drain the entire collection system upstream of the junction manhole.

6.8.4.4 4TH AVENUE NE PUMP STATION INLET DESIGN

Storm water will enter the pump station inlet via a 10' x 8' RCB. The inlet pipe is oversized to limit influent velocity to help provide uniform flow to the pump bays. Inlet velocity computations are provided in Appendix G2. A slide gate will be provided on the wetwell entrance wall to allow for closure of influent storm runoff for maintenance purposes.

A trash rack will be situated at the front of the intake. The trash rack is designed to limit flow through velocity to 2.5 feet/second in the event that the trash racks are 50% blocked by debris. The vertical openings will be 2" to prevent large debris from entering the pump station wet well. Trash rack velocity calculations are provided in Appendix G1. The trash rack will be split into 3 equal length sections. Each section will have a manually operated trash rake with an electric winch for debris removal. The trash rake will be lifted by a permanently fixed jib crane on top of the pump station inlet structure. Structural design of the trash rack is documented in Appendix F3.

Multiple vacuum connections will be permanently installed on each side of the trash rack and on the pump bay divider walls. The Owner will attach a vacuum truck to these permanent pipes to utilize them for sediment and debris removal.

6.8.4.5 4TH AVENUE NE PUMP STATION WETWELL DESIGN

The pump station has a rectangular intake and has been sized based on geometric recommendations from ANSI/HI 9.8 (2018) where applicable. ANSI/HI 9.8 provides guidance primarily for open intakes with uniform inflow from a water body and does not offer specific guidance for pump stations with concentrated influent from stormwater conduits. Through discussions with the USACE, it was suggested that the 1994 version of HI's *Centrifugal Pump Design and Application*, as well as *The Hydraulic Design of Pump Sumps and Intakes*, by M.J. Prosser be consulted to verify adequate spacing from the influent pipe to the pumps. Dimension recommendations provided in ANSI/HI 9.8 are also generally tailored for pipes with true inlet bells such as vertical turbine solids handling (VTSH) type pumps. The centrifugal submersible pumps selected for the 4th Ave NE pump station have a volute with a diameter nearly twice the inlet bell diameter of a VTSH pump with a similar capacity. Due to the geometry of the selected pumps, there is some variance between selected dimensions and those recommended by ANSI/HI 9.8.

Preliminary wetwell geometry is provided in Appendix G2 and is based on a Flygt (Xylem) model CP 3501/765 3~ 1430 and KSB model KRTK 500-634/9010XNG-S centrifugal submersible pumps. Product data sheets for the design pumps are provided in Appendix G1.

A building will be constructed on top of the wetwell of the pump station to provide protection for the process and electrical equipment from the elements and provide space for maintenance. At the request of the owner, the diesel generator will be located outside of the building. A bridge crane lifting system will be provided inside the building for submersible pump maintenance. The height of the building was set to provide enough clearance to lift the pumps a minimum of 4' from the pump bottom to the operating floor to allow the pumps to be loaded on a flatbed trailer.

6.8.4.6 PUMP DESIGN, SELECTION AND OPERATION

The design pump station capacity is 20,000 gpm. The total station capacity will be 30,000 gpm. The station capacity will be provided by 3 -10,000-gpm pumps. This configuration will allow the pump station to operate at 2/3 capacity if one pump fails, in accordance with the Project Design Guidelines for the MREFPP. Centrifugal Submersible pumps were selected based on project sponsor recommendation. Divider walls will be provided to separate the wetwell into 3 separate pump bays.

Each pump will be outfitted with a dedicated discharge pipe. The discharge pipes will have a diameter of 24" and will be constructed of ductile iron pipe. Discharge piping inside the wetwell will have flanged connections. The pipes will pass through steel wall sleeves imbedded in the concrete back wall of the wetwell. The buried piping will be fully restrained flexible joint pipe. A flexible expansion joint will be installed immediately outside of the wetwell wall. The flexible expansion joints will provide for expansion and flexibility if differential settlement of the structure and levee occurs. The flexible expansion joints will connect to the discharge piping with mechanical joints. An additional flexible expansion joint will be installed on each line near the penetration through the gatewell wall for the same purpose. Each discharge line will be approximately 160 LF, with 20 LF contained inside the structure and 140 LF in the yard and under the roadway separating the pump station and the gatewell. The discharge pipes will terminate with flange-mounted flap gates in the discharge chamber. The velocity in the discharge piping will be approximately 7 fps. Design information regarding the discharge piping, including velocity computations, are provided in Appendix G3.

The maximum hydraulic grade line elevation at the inlet to the pump station during the design flow rate was estimated to be 1547 feet. The pump off elevation is 1534.5 feet. The invert elevation of the discharge piping is 1562'. Over this range of head conditions, each of the pumps will operate in the range of 9,000 to 14,000 gpm. Head loss calculations for the system between the pump station and the discharge into the gatewell are provided in Appendix G1.

A small submersible dewatering pump will be provided in the wetwell for dewatering and maintenance purposes. The 10,000-gpm pumps will be set at an elevation that will minimize the demand of the submersible pump to only dewatering the wetwell and the first upstream segment of pipe. The pond and the rest of the collection system will be completely dewatered by the 10,000-gpm pumps. For design purposes, a Flygt (Xylem) model NP 3127 MT 3~ Adaptive 438 was utilized. The dewatering pump will have a capacity of approximately 700 gpm and will utilize 6" ductile iron pipe for discharge. A freeze protection system will be provided for the dewatering pump. The type of freeze protection system has not yet been selected, but will likely consist of an air bubbling system, a water recycling system, or a glycol or electric heat system.

The wetwell, inlet pipe, and STS 2 are utilized for pump cycle time storage and provide more than sufficient volume to prevent over cycling of the pumps. The pumps will be operated with constant speed drives. Details regarding operating point selection and cycle time evaluation are provided in Appendix G1.

6.8.4.7 GATEWELL

The gatewell will be constructed on the river side edge of the line of protection. Both the gravity bypass pipe and the 4 pumps will discharge into the gatewell. The gravity bypass pipe will enter into a separate chamber on the upstream portion of the gatewell. This chamber will be separated from the pump discharge chamber by a center dividing wall. A combination slide/flap gate will be installed on the river side of the dividing wall. This will allow for gravity operation during intermediate flood levels on the Mouse River. A slide gate will also be installed on the upstream side of the dividing wall to provide redundant protection during blocked-gravity periods. The pump discharge piping will be set above the 2011 design flood elevation and will have flange mounted flap gates for redundant protection against flooding. The pumps will discharge into the gatewell chamber downstream of the dividing wall so that pump discharge will still be achievable even if the redundant sluice gate is closed. A gravity pipe will convey storm water from the gatewell to an open channel and ultimately to the Souris River. The slide gates will be manually operated by electric actuators.

6.8.4.8 CIVIL SITE LAYOUT

A concept of the pump station site layout is shown on Construction Drawing C-401 in Appendix K. The following civil elements were evaluated during design of the pump station:

- Access – The pump station site will be accessed from the realigned 4th Avenue NE on top of the line of protection. Within the site, access will be provided to the pump station building, pump station trash racks, gatewell, outfall pipe, and the 4th Avenue NE Detention Pond. Access road widths and turning movements are based on the City of Minot's current vactor truck.
- 4th Avenue NE Pump Discharge Gatewell – The 4th Avenue NE Pump Discharge Gatewell is located on the river side edge of proposed line of protection. It will be accessible from the realigned 4th Avenue.
- Box Culvert/Pump Station Inlet – The location of the pump station inlet is based on hydraulic requirements of pump station influent storm water relative to conveyance from both the collection system and 4th Avenue NE Detention Pond.
- Site Grading – The pump station operating floor elevation was set at 1555.5', which is above the 100-year flood elevation from the effective FEMA FIS, as well as the anticipated 100-year flood elevation after full construction of all MREFPP phases. This elevation is also above the anticipated residual hazard area determined by the interior drainage analysis for the project.

6.8.5 STRUCTURAL DESIGN

6.8.5.1 STRUCTURAL CODES, STANDARDS, AND REFERENCES

This section documents the methodology, assumptions, criteria, and input information used in structural design of the Pump Station Building (superstructure) and emergency generator facility and demonstrates adherence to applicable codes and standards.

6.8.5.1.1 TECHNICAL GUIDANCE AND REFERENCE STANDARDS

2015 International Building Code

- ASCE 7-10, Minimum Design Loads for Buildings and Other Structures[17]
- ACI 318-14, Building Code Requirements for Structural Concrete and Commentary[9]
- ACI 530-13, Building Code Requirements for Masonry Structures[11]
- ACI 530.1-13, Specifications for Masonry Structures[11]
- AISC 360-10, Specifications for Structural Steel Buildings[14]
- ADM 1-10, Aluminum Design Manual[6]
- Stantec Design Quality Procedures

6.8.5.1.2 DESIGN METHODS AND ASSUMPTIONS

Structures have been designed in accordance with sound engineering principles based on the references and codes listed herein. Unless otherwise approved by the project's Lead Structural Engineer, the following criteria applies:

- The 2015 IBC has been adopted by the North Dakota State Building Code.
- Concrete design is in accordance with ACI 318-14[9]. Strength Design procedures have been used.
- Structural steel design is in accordance with AISC 360-10[14].
- Aluminum design is in accordance with ADM-1-10[6].
- Masonry design is in accordance with ACI 530-13[11] and ACI 530.1-13[11]

6.8.5.2 MATERIALS

This section summarizes material properties and assumptions that will be used for the structural design. Unit abbreviations are defined as follows: psi = pounds per square inch, pcf = pounds per cubic foot, ksi = kilopounds per square inch, and kcf = kilopounds per cubic foot.

6.8.5.2.1 CONCRETE

- The specified compressive strength for all structural concrete shall be 4500 psi.

6.8.5.2.2 MASONRY

- ASTM C90 – normal weight CMU, f'm = 1,500 psi
- ASTM C270 – Type S mortar, f'c = 1,800 psi
- ASTM C476 – Grout, f'c = 2,000 psi

6.8.5.2.3 REINFORCING STEEL

- The minimum yield strength shall be 60 ksi.
- Reinforcing steel shall conform to ASTM A615 - Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement.
- Welded reinforcement shall conform to ASTM A706 – Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement

6.8.5.2.4 STRUCTURAL STEEL

- ASTM A992 – Wide Flange Shapes.
- ASTM A500, Grade B – Hollow Structural Shapes.
- ASTM A36 – Other Standard Shapes.
- ASTM A36 – Plates, bars and sheets.
- ASTM AF3125, Grade A325 – Structural Bolts

6.8.5.2.5 STAINLESS STEEL

- Type 316/316L – Submerged or corrosive applications.
- Type 304/304L – All other areas

6.8.5.2.6 ALUMINUM

- 6061-T6 – All applications, except as noted.
- 6063 – Railing

6.8.5.2.7 SOIL

The material properties of the soil and associated design recommendations utilize the design criteria listed within the geotechnical analysis in Section 2.

6.8.5.3 DESIGN LOADS

Structures designed under this project have been designed to meet the requirements of Risk Category IV – Essential Facilities (see Section 7.6.1). The design loads contained herein have been used for the design of the Pump Station Building (superstructure), Generator Foundation and frost protected equipment slab. Design calculations for these structures are provided in Appendix G6.

6.8.5.3.1 DEAD LOADS

Structural material dead loads have been based on the material unit weights indicated herein. All other dead loads have been based on the actual weight of the material and components.

- Concrete – 150 pcf
- Steel – 490 pcf
- Aluminum – 170 pcf

6.8.5.3.2 LIVE LOADS

The live loads listed below are based on Chapter 4 of ASCE 7-10[17] and Stantec standards.

- Roof – 20 psf

6.8.5.3.3 SNOW LOADS

Snow Loads have been determined in accordance with ASCE 7-10[17], Chapter 7.

- Ground Snow Load (P_g) - 50 psf (Section 7.6.8)
- Exposure Factor (Ce) - 0.9 (Table 7-2, Terrain Category C, Fully Exposed)
- Thermal Factor (Ct) - 1.0 (Table 7-3, Heated)
- Importance Factor (Is) - 1.2 (Table 1.5-2, Risk Category IV)
- Flat-roof Snow Load (Pf) - 38 psf
- Design for unbalanced roof snow loads and drift loads

6.8.5.3.4 RAIN LOADS

Rain Loads have been determined during design development based on the building and roof geometry and arrangement. The requirements of ASCE 7-10[17], Chapter 8 have been used to determine the applied loads.

6.8.5.3.5 SEISMIC LOADS

Seismic design has been in accordance with ASCE 7-10[17] and the design criteria provided in the geotechnical investigation. Building structures will comply with Chapter 12. Non-Structural components will comply with Chapter 13. Non-building structures will comply with Chapter 15.

- Short period spectral response acceleration, Ss – 0.062g
- 1-second period spectral response acceleration, S1 – 0.022g
- Site Class – E
- Seismic Importance Factor – 1.5 (Table 1.5-2 and 13.1.3, Risk Category IV)
- Design Spectral Response Acceleration at Short Period (Sds) – 0.103
- Design Spectral Response Acceleration at 1-second Period (Sd1) – 0.0513
- Seismic Design Category – A

6.8.5.3.6 WIND LOADS

Structures and components above grade will have wind loads applied conforming to Chapter 26 of ASCE 7-10[17] with the criteria listed below.

- Ultimate Design Wind Speed, Vult – 120 mph for Risk Category III and IV Buildings and Other Structures (Figure 26.5-1B)
- ASD Design Wind Speed – 90 mph
- Importance Factor – 1.00 (Table 1.5-2, Risk Category IV)
- Exposure Category – C (26.7.3)

6.8.5.3.7 FLOOD LOADS

As proposed, the Pump Station building has been elevated above the 100-year flood elevation based on the latest version of the Preliminary Ward County FIS models, and in addition, will be protected from flooding once the MREFPP is completed. Therefore, the buildings will not be designed for flood loads.

6.8.5.3.8 LOAD COMBINATIONS

Load combinations using strength design have been in accordance with 2015 IBC[56], 1605.2. Load combinations using allowable stress design have been in accordance with 2015 IBC[56], 1605.3.

6.8.5.4 STRUCTURE DESCRIPTIONS

6.8.5.4.1 PUMP STATION SUPERSTRUCTURE

A building, 30' x 42'-8", inside dimensions, will be constructed on top of the pump station wetwell. Exterior walls will be reinforced concrete masonry with brick and stone veneer. The roof structural system has been designed to support snow loads, as applicable, based on roof configuration and materials. A 4-ton capacity bridge crane will be supported by runway beams attached to the underside of interior steel support beams. Vertical and lateral loads from the building will be supported by the concrete sub-structure.

6.8.5.4.2 GENERATOR FOUNDATION

An onsite permanent generator will be provided to power the pump station in the event of primary power failure. A generator with an outdoor sound attenuated enclosure will be installed outside the pump station building, supported by a frost protected cast-in-place concrete foundation.

6.8.6 HVAC AND PLUMBING MECHANICAL DESIGN

The pump station building will consist of a single room above the operating floor. The pump station will consist of two major areas: the operating room and the wet well. HVAC will be provided for the operating room and the wet well. HVAC system for each area will be designed to overcome specific design conditions and to comply with the Standard requirements described herein.

Plumbing will be provided for water supply for exterior hose bibbs and a hose connection in the pump station as a maintenance feature for manually flushing sediment from the wetwell.

6.8.6.1 HVAC DESIGN BASIS

The following are the minimum applicable codes and standards that will be used for the design of the HVAC systems:

- U.S. Army Corps of Engineers, Engineer Manual 1110-2-3105[109] (changes 1 and 2)
- Latest applicable version of North Dakota State Building Code with City of Minot Ordinances and Amendments.
- 2014 NFPA 70, National Electrical Code with City Amendments
- 2012 International Fire Code with City Amendments
- 2012 International Mechanical Code with City Amendments
- 2012 International Fuel Gas Code with City Amendments
- 2012 International Energy Conservation Code with City Amendments
- 2012 International Plumbing Code with City Amendments
- Latest applicable version of ASHRAE Fundamentals Handbook
- Latest applicable version of ASHRAE Std. 62.1-Ventilation for Acceptable Indoor Air Quality
- Latest applicable version of ASHRAE Std. 90.1- Energy Standard for Buildings Except Low-Rise Residential Buildings
- Latest applicable version of SMACNA Duct Construction Standards, Metal and Flexible
- Latest applicable version of SMACNA Duct Construction Standards, Fibrous Glass Duct
- Latest applicable version of NFPA 13 Fire Sprinkler Systems, Installation
- Latest applicable version of NFPA 90A Air Conditioning and Ventilating Systems
- 2016 NFPA 820 Fire Protection in Wastewater Treatment and Collection Facilities
- Latest applicable version of OSHA - Organization Safety and Health Administration

From ASHRAE Fundamentals Handbook 2013, Minot Intl, ND USA (WMO#727676), the following climatic design information has been collected to determine heating, ventilating and air conditioning requirements for the facility. This climatic design information represents 99.6% heating and 0.4% cooling design conditions; it does not take into consideration extreme climatic conditions.

Summer	91.2 degrees F DB (dry bulb),
Summer	68.5 degrees F MCWB (mean coincident wet bulb)
Winter	-19.1 degrees F DB (dry bulb)
Elevation	1555 feet above sea level

The pump station building and wet well are normally unoccupied; therefore, ventilation, heating and air conditioning (HVAC) are provided mainly to satisfy standards requirements, prevent overheating and freezing, and mitigate concentrations of explosive gases. Design conditions for the HVAC equipment will be sized and operated to maintain for each space as listed below.

Indoor design temperatures:

Operating Room

- Summer dry-bulb temperature: 104 degrees F
- Winter dry-bulb temperature: 55 degrees F

Wet Well

- Ambient temperature. There is no heat or cooling of this space in the HVAC design.

6.8.6.2 OPERATING ROOM HVAC SUMMARY

Indoor equipment motor, indoor-outdoor temperature differential and solar radiation will generate the cooling load. Indoor and outdoor temperature differential, ventilation, and infiltration will generate the heating load.

The operating room will be provided with one make-up air unit with gas-fired heat, electric heaters and ventilation fan. The HVAC equipment will be sized and controlled to maintain the operating room inside design temperatures as specified above.

U.S. Army Corps of Engineers, Engineer Manual 1110-2-3105[109], requires at least 6 ACH during non-pumping periods for an operating area. Moreover, 2016 Edition of NFPA 820, Table 4.2, Row 5, Line b, indicates that the operating room is considered unclassified when continuous ventilation at 6 ACH or greater is provided. Therefore, HVAC units will be sized to provide continuous ventilation of 6 ACH for the operating room to satisfy both standards. An official letter received from the local jurisdiction, City of Minot Inspection Department, from Mitch Flanagan, dated 12-16-2015, regarding the review of electrical equipment for the 4th Ave. Floodwall stated that classification as a hazardous area requiring the proposed electrical equipment to meet Class I Div. II will not be applied to the Lift Station. Based on this hazardous classification exception letter, the Owner translated that the operating room area should be unclassified regardless of the ventilation rate, and there might be an intent not to operate the ventilation in winter. The ventilation system will be manually controlled by on/off switch(es) and provided with temperature adjustable set points.

The heating requirements for the building is normally provided by the make-up air. Supplemental electrical heating for freeze protection is furnished, to provide redundancy when the make-up air unit is not operating.

The operating room HVAC equipment will be operated by a combination of automatic and manual controls. The room temperature will be automatically maintained by onboard controls systems and wall mounted thermostat. The temperature set points will be adjustable. The ventilation equipment will be manually controlled by on/off switch(es), normally in the on position.

6.8.6.3 WETWELL HVAC SUMMARY

U.S. Army Corps of Engineers, Engineer Manual 1110-2-3105[109], requires 15 ACH of outside air for a wet sump for personnel entry. The 2016 Edition of NFPA 820, Table 4.2, Row 4, classifies stormwater wetwells as NEC hazardous Class 1, Division 2. However, an official letter received from the local jurisdiction, City of Minot Inspection Department, from Mitch Flanagan, dated 12-16-2015, regarding the review of electrical equipment for the 4th Ave. Floodwall stated that the NFPA Classification requirements to meet Class 1 Division 2 need not to be applied to the facility, and instead should be considered as unclassified area. Based on this hazardous classification exception letter, the Owner translated that the wetwell should be considered as an unclassified area.

Ventilation equipment will be provided to satisfy the 15 ACH requirement per the USACE for personnel entry. The exhaust fan and corresponding makeup air unit for the wet well will only operate during occupied periods. Wet well ventilation will be manually controlled and normally off. The ventilation system will be designed to be interlocked with the wet well lighting and shall be energized a minimum of 10 minutes before personnel can enter the space. No heating will be provided for this system. Therefore, it is recommended that the ventilation system will only be operated at above freezing conditions.

6.8.7 ELECTRICAL DESIGN

6.8.7.1 DESIGN METHODS

The pump station building will consist of a single room above the operating floor. Preliminary electrical and control designs have been completed. Electrical design for the pump station is in accordance with the current National Electrical Code (NEC)[61] and applicable local codes. The following sections provide an overview of the design criteria that has been utilized in the design.

6.8.7.2 ELECTRICAL POWER DISTRIBUTION

The Electrical system at the 4th Avenue NE Pump Station is designed to provide reliable primary and backup power for the pump station.

Utility power from Xcel Energy will be extended via conductors to a transformer located on site which will be dedicated to the pump station. The transformer will step down the 13.8kv primary voltage on the Xcel Energy distribution system to 480V for service to the pump station.

The utility electric service will be backed up by an onsite engine generator sufficiently sized to power the full capacity pump station at full load. The recommended fuel source is diesel in a sub-unit double-walled tank to maximize fuel source availability. The fuel tank will have a capacity adequate to power the generator at full power for approximately 12hrs. The generator will include provisions for connecting a portable load bank for testing the generator at rated load without using the pumps. The generator will be installed outside the pump station building in a weatherproof, sound-attenuated enclosure.

The pump station main switchgear will include an automatic transfer control scheme to sense loss of utility power, automatically start the generator, and transfer the load. When the utility source returns, the transfer controls will automatically return to utility power.

The 2016 Edition of NFPA 820 “Standards for Fire Protection in Wastewater Treatment and Collection Facilities”, Table 4.2, Row 4, classifies stormwater wetwells as Class 1, Division 2. However, an official letter received from the local jurisdiction, City of Minot Building Official, stated that the NFPA Classification requirements to meet Class 1 Division 2 need not to be applied to the facility, and the wetwell should instead be considered an unclassified area. The selection of main equipment located in the wetwell, including pumps, isolation gates and trash racks, is independent of the area hazardous classification. Therefore, the wetwell electrical design will follow all rules for a wet but unclassified space.

All electrical equipment will be mounted not less than 6” above the established project 100-year river assuming all phases of the MREFPP are completed.

6.8.7.3 PUMP STATION AND CONTROL SYSTEM

The pump station controls shall conform to the following standards:

- ISA standards

- NEMA standards
- National Electrical Code

The pump station controls shall consist of field instruments such as wetwell-level transducers and a control panel in the operating room with programmable logic controller (PLC). The control panel cover will have a touch-screen human/machine interface (HMI). System control logic will reside in the PLC.

The control system will include the following design features:

- PLC processors for the pump station will be specified as Rockwell, CompactLogix.
- Monitoring of Power Metering and Generators will be via Modbus TCP/IP over Ethernet.
- The PLC-Based controls will be powered from an Uninterruptable Power Supply (UPS).
- Redundant Level Transmitters will be provided for pump station control.
- The following preliminary instruments selection will serve as the basis for the design.
  - Submersible Level Transmitters: KPSI 750 Series, Sigma Controls, Inc. Series 6000MP
  - Ultrasonic Level Transmitters: Siemens, Multi-Ranger, Rosemount
  - Mechanical Float Level Switches: Roto-Float, Siemens, Flygt, Magnetrol, Kari
  - Pressure Gauges: Ashcroft 1279, Ametek Solfrunt Series 1900

The pump station will operate automatically since it needs to pump both during normal storm events and flood events. Based on wet-well level, the PLC will include logic for starting and stopping pump(s). A pump start sequence algorithm will be implemented in the PLC to rotate the start sequence of the pumps, equalizing pump wear over time.

A float-switch backup-level sensing system will be utilized for equipment protection and in case the level transducers or PLC fail. A high alarm level will call the station to run at capacity. A low alarm will call the station to shut off all pumps.

The PLC will monitor all analog inputs and will generate the appropriate control response, process alarms, and signal fail alarms. The PLC will monitor digital inputs for equipment status and alarms and will control a portion of the equipment through digital outputs. Refer to control system design features listed above for equipment to be monitored and controlled via network communications. A telephone alarm dialer will be included to generate a call to pre-programmed numbers in the event of an alarm. Interface to the Minot's data acquisition (SCADA) system will also be implemented through the installation of, and interface to, a communications panel provided by others, meeting the City's Wi-Fi panel standards.

The HMI will provide local display of system status, including alarms, via screen-view on the front of the control panel. Graphic screens will be built to depict the operation of the pump station, including wet-well level, pump status, and other aspects of the pump station operation.

Motorized electric operators will be installed on all gates in the pump station and gatewell. Gates will be operated manually.

6.8.7.4 4TH AVENUE NE PUMP DISCHARGE GATEWELL

The electrical design for the 4th Avenue NE Pump Discharge Gatewell will include only outdoor lighting, electrical receptacles, and two powered gate actuators.

6.8.8 ARCHITECTURAL DESIGN

The 4th Avenue NE Pump Station will include architectural features designed to fit within the context of the surrounding neighborhood as well as to be complimentary to other Pump Station designs in the area. The building will consist of concrete masonry units (CMU), cast stone veneer, and architectural precast concrete.

6.8.8.1 DESIGN BASIS

The architectural design of the Pump Station has been based on City of Minot, ND, Code Requirements, which references the 2017 North Dakota State Building Code and consists of:

- International Building Code (IBC) and Amendments 2015
- International Energy Conservation Code (IECC) 2009
- International Mechanical Code (IMC) 2012
- North Dakota State Amendments to IBC and IMC
- Life Safety Code (NFPA 101)
- Occupational Safety and Health Administration (OSHA)
- American's with Disabilities Act (ADA)
- Accessible and Usable Buildings and Facilities IICC A117.1-2009

The Pump Station is approximately 1,280 square feet and will be classified as a Group F-2 (Low Hazard factory industrial) with a construction type identified as Type IIB. The roof system is required to be an IIB Class 'C' based on these classifications, and does not require an automatic fire suppression system. The allowable height and area for this type of occupancy and construction type is 55 feet and 23,000 square feet maximum. The occupant load factor is 100 gross for industrial areas, and with a square footage of 1,280, the total occupant load is 13. Based on the F-2 occupancy and occupant load of 13, there is one exit required, and the common path of egress travel distance (without sprinkler system) is 75 feet.

The Pump Station is exempt from Americans with Disabilities Act (ADA) based on 2010 ADA Standards for Accessible Design, Section 203: "General Exceptions; Machinery Spaces, Spaces frequented only by service personnel for maintenance, repair or occasional monitoring of equipment shall not be required to comply with these requirements or to be on an accessible route. Machinery spaces include, but are not limited to, elevator pits or elevator penthouses; mechanical, electrical or communications equipment rooms; piping or equipment catwalks; water or sewage treatment pump rooms and stations; electric substations and transformer vaults; and highway and tunnel utility facilities." Except for regular monitoring and maintenance of equipment, the above Pump Station is not intended for human occupancy for extended periods of time.

6.8.8.2 PUMP STATION BUILDING

The Pump Station will be approximately 33'-4" x 46'-0" and will be designed to accommodate an operating room with adequate space for the pump and electrical equipment installation and maintenance. A bridge crane system and floor hatches will be provided for pump removal and maintenance and operator access. The operating room will be enclosed as a controlled space with the inside temperature and humidity controlled by an HVAC system.

The exterior walls of the Pump Station will be a cavity wall system consisting of a cast stone veneer, a 4-inch-wide cavity with 2-inch thick rigid insulation, and 12-inch by 8-inch by 16-inch concrete masonry unit (cmu) backup wall. The exterior wall design will feature precast concrete banding on the lower and upper portions of the walls, around the glass block windows, and above the entrance door. The veneer of the wall will be cast stone veneer with one color at the base of the building below a precast concrete band located 3'-4" above finished floor and the second color for the veneer above the lower banding to the roof soffit. The corner columns and pilasters will consist of 6" precast concrete panels, and the north elevation archway will consist of 4” precast concrete panels. Masonry and stone veneer exposed to the exterior shall receive a spray-applied clear penetrating sealer.

Exterior doors and frames will be flush galvanized steel with painted finish. Overhead coiling doors will be motor-operated, insulated galvanized steel doors with weather stripping. Six-inch diameter, concrete-filled, painted bollards will be provided to protect service entries and any other mounted equipment. Windows will be glass block and located at 10’-8” above the finished floor to allow light into the building while providing a solid wall surface below for mounting equipment. Louvers will be aluminum with painted finish and will be provided as required.

Building insulation will be provided for the roof and exterior walls, including walls of air-conditioned rooms, and conform to the energy code requirements based on Chapter 4 of the International Energy Conservation Code as referenced by Chapters 13 and 14 of the International Building Code. A vapor barrier will be provided along the inside face of the insulation.

Caulking, sealing, and moisture protection will be provided for weather tight construction for all buildings. A plastic vapor retarder will be placed over the backfill and under any new concrete floor slabs.

The roof system for the building will consist of a sloped roof with metal shingles, on rigid insulation, on structural roof deck. The roof system will meet UL Class A rating and satisfy wind uplift requirements for this area. The insulation will conform to the energy code requirements.

Interior finishes in the building will be as follows:

- Flooring for the pump station will be concrete slab with a steel trowel finish.
- Walls will be painted concrete masonry units.
- Signage will be provided per the appropriate authorities, agencies, and the building code for applicable areas. Fire protection will be provided by fire extinguishers based on the requirements of Chapter 9 of the building code and NFPA 10.

6.9 SLOPE EROSION PROTECTION

Erosion protection will be required along portions of the Mouse River channel bank, levee, and bridges through the Phase MI-5 project limits to minimize erosion and scour potential from flood flows. Additional details on the proposed slope erosion protection is included in Appendix E3.

6.9.1 ALTERNATIVES ANALYSIS

Alternative protection methods were evaluated which include riprap, turf-covered riprap, Turf-Reinforcement Mat (TRM), cellular confinement mats, cable-concrete mats, and bio-engineering methods. Each method was assessed based on performance, cost, constructability, aesthetics, and environmental considerations. A streambank stabilization memorandum from BARR Engineering to the SRJB providing information on the methods described above is included in Appendix E3.1.

6.9.2 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, EM 1110-2-1601 [93], and “Technical Supplement 14B in Stream Restoration Design” of the National Engineering Handbook Part 654 [67]. The primary focus of the erosion protection design was to armor the flood risk management system; however, channel bank erosion and erosion protection at bridge structures were also included.

A combination of riprap and TRM was used within the Phase MI-5 project limits to protect areas with high velocities using a robust design approach, while also incorporating more aesthetically pleasing products that still meet erosion protection requirements. The erosion protection design included an evaluation of river geometry, soil conditions, scour potential, and consideration of various design velocities. A conceptual layout is shown in Figure 6-12 and in the Construction Drawings (Appendix K). Additional erosion protection design details and computations are provided in Appendix E3.2.

Much of the flood risk management system within MI-5 is set back from the river channel, with exception to the eastern end where the levee and the 4th Avenue road embankment become one, at which point the levee is on the river bank itself. Natural grassed vegetation provides sufficient erosion protection for the majority of the flood mitigation features in the overbank areas since the velocities are lower, but combinations of riprap and TRM are planned for an area on the eastern end of the project where channel velocities range from 8 to 10 feet per second (fps). Riprap was also designed for the railroad bridge on the western end of MI-5 and the 4th Avenue NE Pump Station outfall. Erosion protection designs for the specific locations are described in the following sections.

Figure 6-12 Slope Erosion Protection Plan

6.9.2.1 LEVEE EROSION PROTECTION

Erosion protection measures for the levees were designed using velocities from the 100-year event and the 2011 event with consideration given to both overland velocities as well as channel velocities. Due to the levee's proximity to the channel between river station 11822+00 and 11831+00, the modeled velocities in the channel were used for the levee erosion protection measure design instead of overbank velocities. Typical sections of the levee erosion protection can be found in Appendix K.

Table 6-4 summarizes the erosion protection planned for the MI-5 reach.

- River Station 11822+00 to 11823+50: The levee, at the end of the MI-5 design reach, is a transition segment from an existing levee system to the full height (2011 design). Since the downstream reaches are expected to be raised in the future, this reach was protected using TRM instead of riprap. (R270 riprap will line the channel bank, as described in the following section).
- River Station 11823+50 to 11824+25 This is a bench segment where the levee is set back from the river bank. Here, TRM will extend from the river bank to the levee toe, throughout the bench segment, and R140 riprap will extend up the slope of the levee to the 2011 event elevation (~1561.5), at which point TRM will be extended to the top of levee. (R270 riprap will line the channel bank, as described in the following section).
- River Station 11824+25 to 11826+50: Through this segment, the levee slope extends down to the river bank, with a small, varied bench segment. Here, the R270 bank riprap will extend from the toe of the channel bank to the 2011 event elevation (~1561.5) at which point TRM will be extended to the top of levee.
- River Station 11826+50 to 11829+50: This is a bench segment where the levee is set back from the river bank. Here, TRM will extend from the river bank, to the levee toe throughout the bench segment, and R140 riprap will extend up the slope of the levee to the 2011 event elevation (~1561.5), at which point TRM will be extended to the top of levee. (R270 riprap will line the channel bank, as described in the following section). Figure 6-13 identifies this typical section.
- River Station 11829+50 to 11831+00: TRM will be extended from the top of the channel bank to the top of the levee. (R270 riprap will line the channel bank, as described in the following section).

Table 6-4 Levee Erosion Protection Summary

Souris River Station	Erosion Protection Type	Type	Thickness (inches)
11822+00 to 11823+50	Turf-Reinforcement Mat	VMAX P550	N/A
11823+50 to 11824+25	Riprap/TRM	R140/ VMAX P550	30
11824+25 to 11826+50	Riprap	R270	30
11826+50 to 11829+50	Riprap/TRM	R140/ VMAX P550	30
11829+50 to 11831+00	Turf-Reinforcement Mat	VMAX P550	N/A

Riprap revetment along the levee is shown in Figure 6-13.

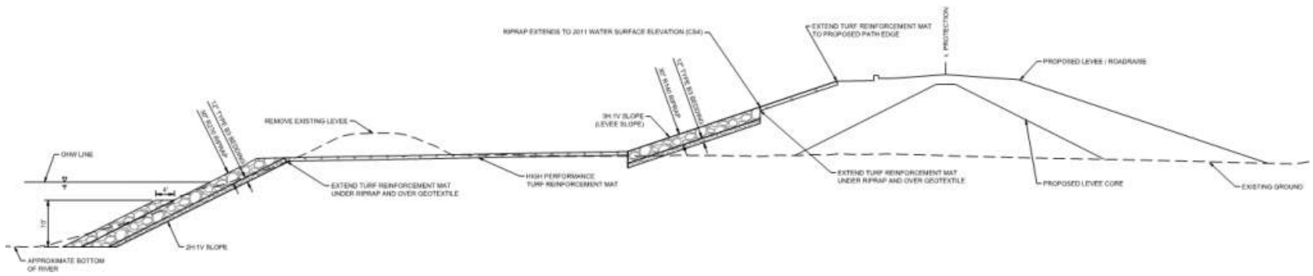


Figure 6-13 Typical Levee Erosion Protection Section

6.9.2.2 CHANNEL BANK EROSION PROTECTION

The Mouse River channel banks generally have little vegetation resulting in minimal natural erosion protection. Therefore, MI-5 channel bank armoring has been planned where erosive velocities have the potential to adversely affect critical infrastructure, such as through bridges and immediately adjacent to the primary line of flood protection. However, in an attempt to save costs and maintain a more natural river corridor, riprap will not be placed in non-critical areas. Here, the channel will be allowed to meander naturally while monitoring bank erosion extents to ensure it doesn't adversely affect critical infrastructure in the future. Channel bank erosion protection included in the MI-5 Project is shown in Figure 6-12 as well as in the Construction Drawings (Appendix K). As presented above, erosion protection designs for the levees referenced the higher velocities from either the 100-year event or the 2011 event. However, the channel bank erosion protection was derived using the 100-year event, while also considering more robust riprap designs required for adjacent levees and bridges. Table 6-5 summarizes the channel bank erosion protection for the MI-5 reach.

- River Station 11860+90 to 11861+90: Channel bank riprap through this reach of the river, immediately upstream from the BNSF Railroad bridge is driven by the riprap design from Phase MI-1. The riprap to be installed as part of MI-5 will match that of MI-1.
- River Station 11822+00 to 11831+00: Scour calculations for the channel bank itself did not demand extensive riprap sizes, however due to uncertainties in the 1-dimensional modeling and the sharp change in direction at this levee/bank segment, the same R270 levee riprap was extended down through the channel bank to produce a uniform bed of riprap throughout the reach. The R270 channel bank riprap extends from the toe of the channel bank through the top of bank as described in the typical sections on Construction Drawing C-352. As previously described, segments of levee slope protection are extended to the 2011 event elevation with either riprap or TRM.

Table 6-5 Channel Bank Erosion Protection

Souris River Station	Erosion Protection Type	Riprap Type	Thickness (inches)
11860+90 to 11861+90	Riprap	R470	36
11822+00 to 11831+00	Riprap	R270	30

Configuration of launchable riprap and sizing were completed in accordance with Section 3-11, Method D of EM 1110-2-1601 [93] and is shown in Figure 6-14.

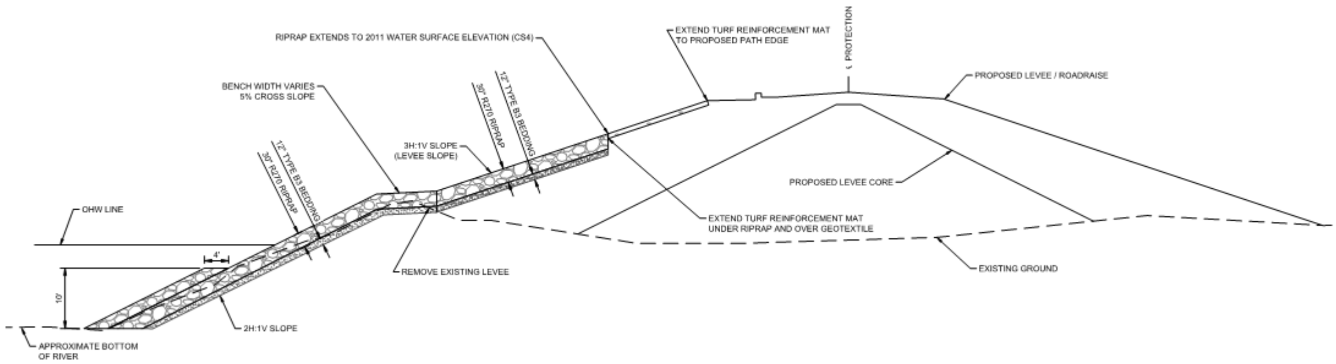


Figure 6-14 Typical Bank Erosion Protection Section

6.9.2.3 BRIDGE EROSION PROTECTION

MI-5 erosion protection design has been extended through the BNSF Railroad bridge. However, the 7th Street NE and 1st Avenue NE bridges were not included in the MI-5 design because the Construction Stage 1.5 design (including MI-5) does not adversely increase velocities when compared to existing conditions. However, when Construction Stage 3 becomes implemented, the velocities will have increased such that additional erosion protection should be implemented for these two additional bridges. The BNSF Railroad bridge riprap design is based on methodologies in HEC-23 for bridge abutments and piers. The bridge riprap size is summarized in Table 6-6, with additional details in Appendix E3.2.

Table 6-6 Bridge Erosion Protection Summary

Location	Souris River Station	Erosion Protection Type	Riprap Type	Thickness (inches)
Railroad	11858+75 to 11860+25	Riprap	R270	30
Railroad	11860+25 to 11860+90	Riprap	R270	30
Railroad	11860+90 to 11861+90	Riprap	R470	30

Riprap revetment along a typical bridge abutment is shown in Figure 6-15.

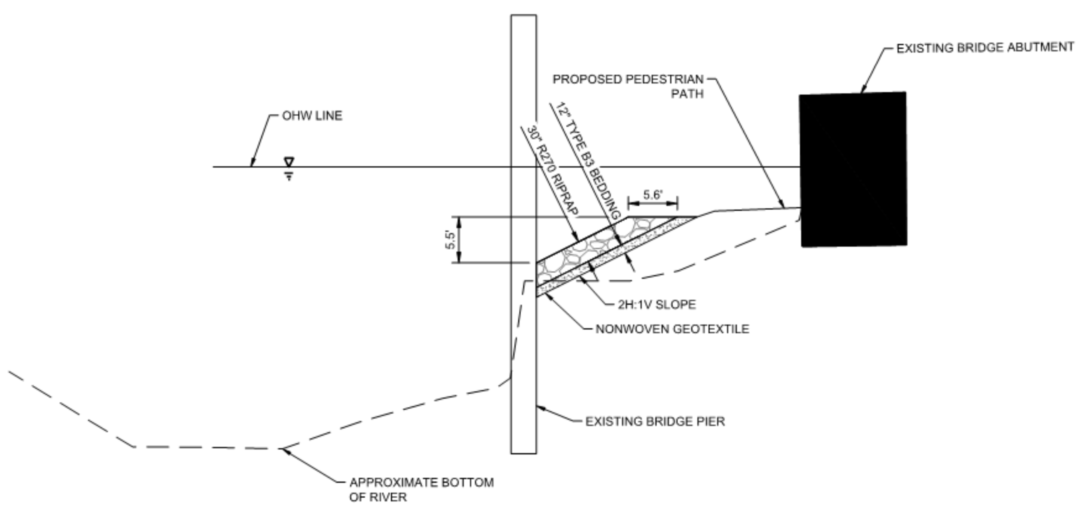


Figure 6-15 Typical Bridge Abutment Erosion Protection Section

6.9.2.4 RIPRAP THICKNESS

Riprap thicknesses were determined using EM 1110-2-1601[93] design guidance, generally to be no thinner than the diameter of W_{100} or 1.5 x W_{50} stone size. At a minimum, the R140 and R270 riprap mats are to be 30-inches, and the R470 mat is to be 36-inches thick.

Portions of the channel bank riprap will be placed below normal water levels. Typical design guidance suggests that riprap volumes are increased by up to 50% (FS = 1.5) to account for uncertainties in construction practices. This practice would result in an added cost and would also require additional mitigation because the extra thickness would further encroach on the hydraulic conveyance of the river. To avoid this, it has been assumed that underwater bathymetric survey will be conducted during construction by a Resident Project Representative to validate the in-place riprap thickness.

6.9.2.5 BEDDING DESIGN

Riprap erosion protection for levees, channel bank, and at bridge structures require geotextile fabric and granular bedding to be installed on the finished ground surface prior to placing the riprap. Typical sections describing the erosion protection are included in Construction Drawings C-352 and C-353 (Appendix K). The granular bedding size is based on the gradation of riprap. Bedding material and bedding thickness is specified in accordance with standard gradations provided in USACE St. Paul District's Standard Riprap Design document [121]. For the MI-5 riprap design, a 12-inch layer of Type B3 should be used for levee and bank sections requiring R140, R270, and R470 riprap.

6.9.2.6 CHANNEL BANK TOE PROTECTION

Toe scour is a frequent cause of riprap revetment failure along channel banks. There are two general methods of construction for toe scour protection, depending on the water levels in the river. Due to typical summer flows in the river and downstream grade control structures, is anticipated that portions of the MI-5 channel construction will be completed “in the wet”. To provide adequate toe scour protection, a launchable riprap design is to be implemented. Here, additional loose riprap will be placed at the bank toe such that as the bank is eroded and the loose stone is undermined, it will roll/slide down into position and prevent future bank erosion. Design guidance also suggests that this method provides a built-in scour gage providing an opportunity to see what scour is occurring under the water because the riprap at the toe will launch down the bank. This allows additional stone to be placed during emergency conditions, if needed. As further described in Appendix E3.2, an additional block of riprap will be placed along the toe of the levee throughout the R270 channel riprap segment. The dimensions of the launchable section will be 10 feet high (22.3 feet on the diagonal) and 4 feet wide (2.3 feet perpendicular to the slope).

6.9.3 PROTECTION AT STRUCTURES

Erosion protection at structures and outlets is required to prevent erosion from concentrated flows. The riverbank and levee adjacent to the 4th Avenue NE Pump Station and 4th Avenue NE Pump Discharge Gatewell is expected to be armored with riprap to protect against high river velocities during flooding events. An armored swale will also be constructed to carry concentrated flows from the 4th Avenue NE Pump Discharge Gatewell to the river.

6.10 MUNICIPAL ROADWAY MODIFICATIONS

Flood protection improvements will impact several existing roadways. Modifications to these streets are required to maintain access and accommodate the flood protection and utility modifications. These streets include 7th Street NE, 4th Avenue NE, Railway Avenue NE, 6th Avenue NE, 7th Street NE, 8th Street NE, 9th Street NE, 10th Street NE, 11th Street NE, 12th Street NE and 13th Street NE as shown in the Construction Drawings in Appendix K.

All roadway modifications are designed to meet applicable federal, state, and City of Minot specifications for urban roadways. Horizontal and vertical geometric features, as well as sight triangle distances, are designed per AASHTO design guidelines.

4th Ave NE and 7th St NE will be realigned to accommodate levee construction between the Mouse River channel and the BNSF Railroad. All other roadways south of the BNSF Railroad and north of the Mouse River will be removed from 3rd St NE to 14th St NE. The realigned, combined 4th Ave NE and 7th St NE roadway will be constructed on top of the levee. The elevation of the roadway will be set to maintain the full levee height at the top of the roadway subgrade. A typical section for the roadway above the levee is shown in Figure 6-16.

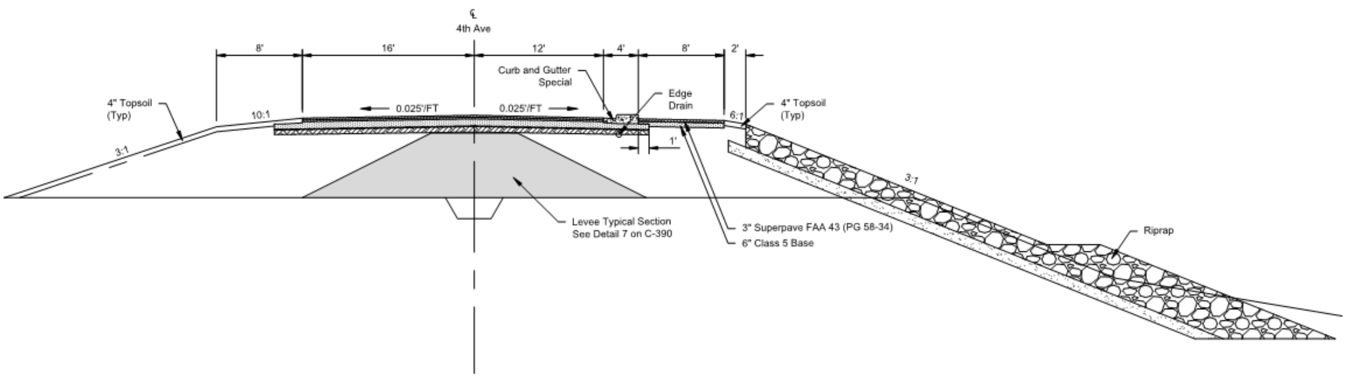


Figure 6-16 Typical Roadway above Levee

Railway Ave NE will be raised to pass above the north/south levee crossing east of 13th St NE on the north side of the BNSF railroad. The elevation of the roadway was set to maintain the top of subgrade above the top of flood protection. To provide the separation between the top of flood protection and top of subgrade, the roadway will be raised approximately 14.5 inches. All vertical curves are designed to meet a design speed of 40 mph per AASHTO.

All remaining street modifications are related to intersection and utility modifications. Sidewalk revisions and improvements are also included as part of the street modifications. These modifications include reconstruction of Railway Ave from 6th St NE to 13th St NE, as well as 8th St NE from Railway Ave north to 7th Ave NE. All other intersections and approach roadways will be reconstructed through the radii to allow for required utility modifications.

6.10.1 TRAFFIC CONTROL

Detailed vehicular and pedestrian traffic control plans are included in the Construction Drawings in Appendix K. All work zone traffic control devices and signing shall be in accordance with the NDDOT Standards for Road Construction[71], NDDOT Standard Drawings and the Manual on Uniform Traffic Control Devices, MUTCD[127].

6.11 RAILROAD MODIFICATIONS

Phase MI-5 of the MREFPP includes significant modifications to the BNSF railroad in order to install a removable closure structure on the west end and sheetpile cutoff on the east end. An overview of the modifications, including temporary facilities, permanent facility modifications and preliminary operations during flooding is included below.

6.11.1 OVERVIEW

6.11.1.1 PROPOSED MODIFICATION – 90% DESIGN

The west flood protection crossing (Removable Closure Structure) was developed to allow for rail traffic to remain operational as long as possible. The current 90% design of the removable closure has the southernmost mainline track higher than the remaining tracks through the yard and across the closure threshold. This design was selected to provide BNSF the ability to use the southern mainline while the other tracks would have the floodwall planks installed during times of flooding. The closure structure has a recessed rail though the closure structure, with the bottom of concrete sill elevation being 1556.84 for the siding lines and northern mainline; and 1558.40 for the southern mainline. The estimated frequency of required closure installation for the siding and northern mainline is approximately a 150-year event, while the south mainline is greater than a 200-year event. Additional information for the removable closure structure can be found in Section 6.5.5.

The sheetpile cutoff wall proposed to cross on the easterly end of the project areas will be below final BNSF grade. The elevation the levee protects to is the design water surface elevation plus hydraulic uncertainty and settlement (where sheetpile is not to be installed). The sheetpile cutoff is proposed to be installed from elevation 1558.75 to 1530.00 and will require the raising of some of the siding tracks to keep the top of the sheet pile out of the railroad ballast. Additional information for the sheetpile cutoff can be found in Section 6.5.6.

6.11.1.2 OPTIMIZATION FOR RAILROAD IMPACT

In October of 2018, a memo was prepared that outlined the efforts to minimize the impacts to BNSF and is included in Appendix E6. As explained in this memo, the current design allows for optimal function of the railroad as it provides BNSF a flood protection threshold that is higher than what is experienced today while minimizing potential downtime in the event the flood protection closure is installed.

6.11.2 TEMPORARY FACILITIES

Temporary tracks (shooflies) are proposed to be constructed in order to construct the closure structure and a storm sewer utility crossing. A combination of shooflies and project staging will minimize impacts to BNSF due to construction of the Project.

Additional details for the shooflies and proposed project staging are included in the Construction Drawings in Appendix K.

6.11.2.1 SHOOFLY

Mainline 1 and Mainline 2 will be temporarily relocated during the construction of the closure structure and storm sewer utility crossing. Multiple layouts for shooflies were developed, which included the option to only have one shoofly, as well as the preferred option of shooflies for both mainlines to the south of the existing track infrastructure. Considering the frequency of trains travelling on the mainline tracks, it was determined to be a better design to keep both mainline tracks in service during construction. These tracks will be located far enough away from the closure structure construction to ensure safety for both the railroad and the contractors building the closure structure.

6.11.2.2 STAGING

Information regarding railroad staging can be found in Section 6.18.2 – Phase 2.

6.11.3 PERMANENT FACILITY MODIFICATIONS

When complete, several permanent modifications will be made to the BNSF railroad, including track realignments and grading, installation of the sheetpile cutoff wall and installation of the removable closure structure footing. Detailed plan and profile sheets of the railroad facility modifications are included in the Construction Drawings in Appendix K.

As part of these modifications, Mainline 1 is proposed to be realigned slightly south and will closely match the existing profile with proposed grades of approximately 0.11% or less in the area of the reconstructed track. Mainline 2 will be raised in the area of the removable closure footing with a maximum proposed grade of 0.33%. Tracks 5 and 8 will receive a ballast raise through this area of the removable closure.

Mainline 1, Mainline 2 and Tracks 3, 4, 5, 6, 7 and 8 will be removed and replaced as needed for the installation of the sheet pile cutoff during time of scheduled track outages. Tracks 3, 4, 5, 6, 7 and 8 will receive a ballast raise through this area in order to obtain the desired height.

Tracks 3, 4, 5, 6, 7 and 8 and Mainlines 1 and 2 will be removed and replaced as needed for the open cut installation of a 72” RCP as well. The Mainline 1 and 2 installation will take place while the shooflies are in operation.

6.11.4 FLOOD OPERATION

Based on information provided by BNSF representatives, BNSF is no longer capable of operating when the top of rail is inundated by 6 inches of water. The current low points are 1556.65 for Mainline 1 and 1555.84 for Mainline 2 just to the west of the existing BNSF bridge over the Mouse River.

The sill of the removable closure structure is the same elevation as the top of rails crossing the closure structure. As such, the rails are recessed down into the stem. Before installation of the stoplogs, a bituminous filler material would be placed in the space surrounding the rails to seal around the rails. The removable closure has been designed to allow for the stoplogs to be installed in two separate stages to keep Mainline 1 operating as long as possible. The recessed sill elevation on the south side for Mainline 1 is 1558.40 (sill elevation 1559.03) (1.75-ft over low top of rail) and the recessed sill elevation on the north side for Mainline 2 and siding tracks 5 & 8 is 1556.84 (sill elevation 1557.47) (1-ft over low top of rail).

To minimize track outage time, the optimum operation plan would initiate the installation of stoplogs across Mainline 1 at elevation 1557.05 and across the siding tracks and Mainline 2 at elevation 1556.34. This plan would provide 1.25 feet of deployment time over the point at which BNSF must suspend operations on Mainline 1 and 0.5 feet of deployment time for the lower threshold (Tracks 5 and 8 and Mainline 2). However, since the majority of the length of the closure (110.2 ft) is only crossing siding tracks and the second mainline, it has been assumed that this section would be installed earlier than the optimum timeframe to ensure proper installation. By installing this earlier, the shorter portion (49.3 ft) at the higher threshold can be kept open longer to allow for operation of Mainline 1. By completing the installation in two phases, the time required for installation of the final portion to complete the closure will be significantly reduced.

Additional information will be presented in the future Operation & Maintenance Manual based on continued negotiations with BNSF.

6.12 RECREATIONAL FACILITIES

As shown in the Construction Drawings in Appendix K, the project will implement a 10’ wide shared-use path along the north side of the Mouse River from 3rd St NE to 7th St NE. An additional 10’ wide shared-use path will pass along the north side of the project, over the levee section generally parallel to the BNSF right-of-way. This path provides connectivity to the existing pedestrian bridge over the BNSF railroad near 8th St NE. South of 4th Ave, an 8’ wide path will be constructed from the Mouse River to east of 14th St NE.

The shared-use paths will provide for pedestrian access throughout the project site. Additionally, a reconstructed underpass beneath the BNSF railroad bridge will provide connectivity with the MI-1 project area.

North of the BNSF railroad, existing sidewalks will be removed and replaced as part of the utility modifications. All sidewalks and shared-use paths will be reconstructed meeting ADA requirements.

6.13 LANDSCAPE DESIGN

Future use of the area between the flood protection system and the Mouse River is unknown by the Minot Park District at this time. As a result, landscaping in this area will be limited to seeding of disturbed areas and preserving trees where possible and compatible with the flood control project. Additional details of the landscape design are included in Section 10.

6.14 RESTORATION

Areas that are disturbed because 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 shall be installed using a variety of methods in different locations, as specified. Additional details of the restoration design are included in Section 10 and the Construction Drawings in Appendix K.

6.15 BORROW AREA SELECTION AND DESIGN

The requirements for levee materials are identified and in the geotechnical section of this report. For Phase MI-5, a site called the MADC site was selected for use. A memorandum discussing the selected borrow area, the MADC site, which includes a location map, existing conditions, boring logs and laboratory test results, is included in Appendix E4. Several additional related borrow material and environmental reports related to the borrow area are also contained within Appendix E4, including cultural, environmental and geotechnical assessments. Location, grading plan, section and erosion control sheets for the proposed MADC site have also been completed and are included in the Construction Drawings in Appendix K. The following paragraphs describe the process for selection and design of borrow sources for levee material.

6.15.1 PER REVIEW

A desktop study identifying potential borrow sources was completed as part of the PER. The study concluded that suitable low hydraulic conductivity fill is located in the valley walls of the Mouse River Valley corridor. Additionally, the study identified several borrow pits that 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 approximately half the cost of obtaining material from one or two offsite borrow locations.

6.15.2 BORROW IDENTIFICATION

The selection of borrow areas was completed in compliance with EM-1110-2-1913, Chapter 4. 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. The borrow site constraints and requirements used in the selection of the site are as follows.

6.15.2.1 SUITABLE MATERIAL

The borrow location must contain a sufficient quantity of impermeable soils meeting the requirements for levee construction. 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.

6.15.2.2 MATERIAL PROXIMITY

The location of the borrow material must not create adverse impacts to the levee or surrounding structures and project features.

Preference was given to borrow locations near the project. If suitable, material from existing levees will be used first. The alternate offsite location should be based on proximity to the levee sections and haul truck accessibility.

6.15.2.3 PROPERTY OWNERSHIP

Project stakeholders who own and manage borrow areas, including Minot and the SRJB, received preference in selecting borrow material. Secondary preference was given to cooperative landowners who would 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.

6.15.2.4 ENVIRONMENTAL CONSTRAINTS

A desktop environmental study was done prior to the initial selection of borrow locations. Borrow source areas within a known cultural, archeological, or other environmental constraint area were not considered.

6.15.2.5 RESTORATION REQUIREMENTS

Selection criteria for a borrow source included minimal restoration requirements beyond replacing topsoil and re-establishing vegetation. Sites requiring construction and/or restoration of structures after material removal were given low consideration.

6.15.3 BORROW SOURCE DESIGN

Borrow source design is to be completed in conformance with Chapter 4, Paragraph 4-4 of EM 1110-2-1913 [96]; the offsite borrow area for MI-5 has been designed based on owner preference and existing constraints. Grading of the site shall be completed based on the proposed features of the borrow site.

6.15.4 BORROW SOURCE ALTERNATIVES

Based on the screening criteria above, a series of potential offsite borrow source locations were identified and considered. These included:

- Highway 2 Site (HW)
- Magic City Campus (MC)
- Mini Golf Course (MG)
- Alternate Borrow Area (ABA) (aka the Price Site in the EIS)
- Minot Area Development Corporation (MADC)

A geotechnical investigation was completed at each area identifying the type of material available and is provided in Appendix E4. Based on these analyses, each of these areas had the potential to provide suitable material for the construction of the levees.

6.15.5 BORROW SOURCE SELECTION

Primarily based on its proximity to Phase MI-5, the MADC site was the selected borrow site after screening the multiple sites presented in the previous section. The location of the selected borrow source is shown in Figure 6-17.

Proposed grading of the MADC site during borrow activities is included in the Construction Drawings in Appendix K. This grading has been developed to be consistent with and based on future owner planned features of the site. Since the current owner has been planning for significant future grading activities at the site for several years, extensive environmental review has already been accomplished and is included in Appendix E4.

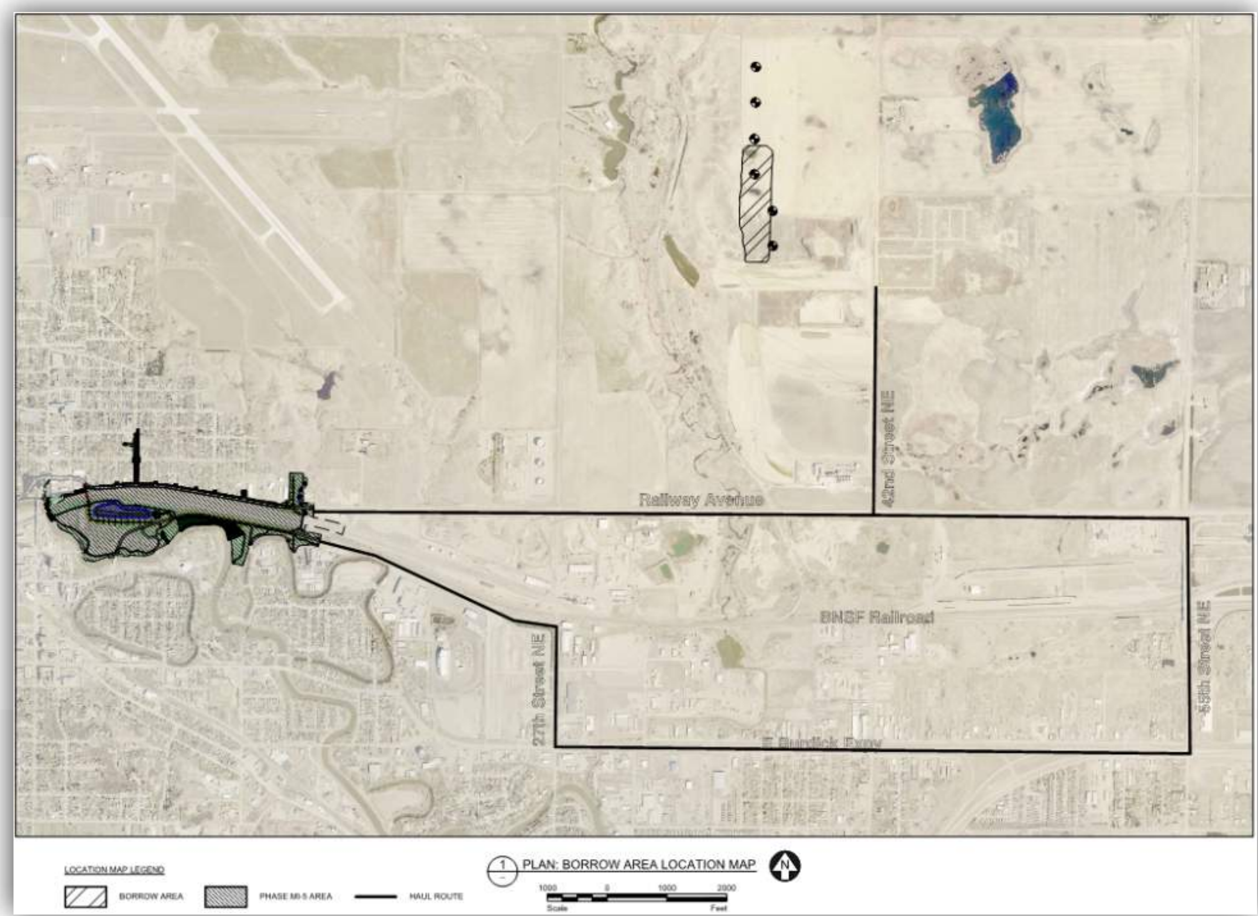


Figure 6-17 Selected Borrow Site Location

6.16 EARTHWORK BALANCE

The estimated volume of material required to construct the levees was calculated as described in the following sections.

6.16.1 MATERIAL ADJUSTMENT FACTOR

Chapter 4 of EM 1110-2-1913 [96] suggests that a shrinkage factor of at least 25% should be used to account for material shrinkage during placement and losses during excavation and hauling, however no material adjustment factor was used. Instead, quantities for the project are based on in-place volumes assuming the contractor will adjust embankment cost to account for shrinkage and swell as necessary in future bids.

6.16.2 METHODOLOGY FOR VOLUME CALCULATIONS

A three-dimensional surface was developed in AutoCAD Civil 3D (software) for the existing ground surface and the final proposed levee surface. A surface-to-surface composite volume calculation was completed in the software by comparing the series of prisms generated by the points that define each surface.

6.17 DISPOSAL OPTIONS

An excess of approximately 104,863 cy of in-place soil material unsuitable for levee fill is expected to be generated from within the construction limits. Work expected to generate unsuitable levee fill material includes the following:

- Installation of infrastructure such as a gatewell, pump station, storm drain structures, floodwalls, and utilities.
- Excavation of exploration trench and pond.
- Excavation of unsuitable material from beneath floodwalls and levees.

At the current 90% design level, all onsite excavations are to be removed from the project site.

6.17.1 DISPOSAL AREA

The assumed disposal area is the City of Minot Landfill. The following summarizes the potential disposal location at the 90% design level.

The Minot landfill is approximately 4.5 miles from the project site. Daily cover soil material is needed for landfill operations and would be accepted at no cost as long as it is suitable for cover. Transportation costs would need to be included in the estimate.

It is currently estimated that 104,863 cy of in-place excavated material will be transported to the Minot landfill for disposal.

6.18 STAGING

The proposed project is complex and thus will need to be phased in order to maintain operations for the surrounding areas. The preliminary construction staging plan for the project was broken down into the following four phases.

- Phase 1 – Pump Station Site
- Phase 2 – Work Completed on BNSF Right-of-Way
- Phase 3 – Underground Utilities and Roadway North of the BNSF Railyard
- Phase 4 – Flood Protection Features

The preliminary phases listed above are not also predicated on finishing one phase prior to moving on with another phase. The following sections break down the phases listed above into subcategories, which list criteria describing how the preliminary project phasing was developed. The proposed staging plan coordinates with the proposed traffic control plan for work areas, and each section described below assumes the traffic control plan is initiated as set forth in the Construction Plans in Appendix K. Additional drawings and details of the phases and sub-phases are shown on the preliminary Construction Drawings located in Appendix K.

6.18.1 PHASE 1

Work completed under Phase 1 generally consists of installing items associated with the proposed pump station site, associated piping, and adjacent levee/roadway embankment work. Additional details on each subphase is included below.

6.18.1.1 PHASE 1A

Construction of the proposed pump station, pump discharge chamber, STS 1 and 2, as well as adjacent associated underground piping installation occur in Phase 1A. Although the pump station will likely take the longest to construct, the critical path for Phase 1A is to install the 72” RCP pipe north of STS 2 to a location just south of the existing BNSF railroad mainlines. In this area, the 72” RCP pipe must be installed under the existing valley forcemain (sanitary sewer forcemain) and water transmission line, neither of which can be out of service during this phase of the project. Once the 72” RCP is installed and backfilled, Phase 2A can start and move forward concurrently with Phase 1A. It is anticipated that Phase 1A will be on-going through most of Phase 2.

A small portion of the pond excavation can also occur in Phase 1A, but an existing gravity sanitary sewer line currently runs through the proposed pond, as well as an existing watermain mainline which needs to stay in service until Phase 3C is completed.

Prior to beginning Phase 1A, temporary paving for a roadway bypass of the Phase 1A construction area will be completed as set forth in Traffic Control Plan in Appendix K.

6.18.1.2 PHASE 1B

Phase 1B consists of completing the outfall pipe and excavating the outfall channel from the proposed discharge chamber. This phase cannot start until the roadway leading over the pump discharge lines is in place, and ready for temporary traffic flow, as the temporary bypass for traffic that was installed prior to Phase 1A will be cutoff.

6.18.2 PHASE 2

All work completed on BNSF right-of-way is completed in Phase 2. As stated above in Phase 1A, Phase 2 cannot start until the 72” RCP line is installed near the existing BNSF mainlines, and each sub-phase must be completed prior to moving onto the next sub-phase as part of Phase 2. Additional details on each subphase are included below.

6.18.2.1 PHASE 2A

Phase 2A involves construction of two mainline shooflies that will allow for the construction of the closure structure and installation of the 72” RCP storm crossing. To construct the proposed BNSF mainline shooflies, the pedestrian bridge stairs must first be removed, and the bridge temporarily closed. The proposed shooflies shall be constructed and tied into the main lines as shown on the Construction Drawings in Appendix K. Once the shooflies are operational, Phase 2B can start.

6.18.2.2 PHASE 2B

There are three major features that are installed during Phase 2B: the removable floodwall closure, a 72” RCP, and the 4th Avenue NE Tieback Levee sheet pile cut-off wall and associated stormwater drainage. All these features shall be installed while the mainline shoofly is in operation. At the conclusion of Phase 2B, the temporary shoofly can be taken out of operation, and rail traffic can commence over the newly constructed removable closure structure footing. Additional details on each major feature are included below.

6.18.2.2.1 REMOVABLE CLOSURE

A section of the removable closure structure which will cross both existing mainline tracks will be constructed under Phase 2B. Track re-alignment and grading on the existing mainlines will also be completed during the mainline shoofly operation.

6.18.2.2.2 72" RCP

The 72" RCP will be continued from Phase 1A and installed to the north through the existing mainlines, which will be out of service due to the temporary shoofly. Siding tracks 3-5 will also need to be temporarily shut down with a track window to complete the installation of the proposed storm water pipe.

6.18.2.2.3 SHEETPILE AND TRACK GRADE RAISE

A track window will be required to install the sheetpile wall through the existing mainlines, as the proposed shoofly is west of the 4th Avenue NE Tieback Levee location. The remaining siding tracks 3-5 would also have the sheet pile installed, as well as the track grade raise during the time of the shutdown while installing the 72" RCP.

6.18.2.2.4 ASSOCIATED STORMWATER DRAINAGE

The existing underground drainage located in the existing railyard will also be connected to the interior drainage system as part of this phase. To accommodate this connection, a 30" pipe will be jack and bored under the existing mainlines and siding lines. A storm sewer structure will be installed between Mainline 2 and Siding Track 3, and should be installed during the track window in which the sheetpile is being installed.

Temporary pumping from the newly installed manhole located within the BNSF right-of-way as part of Phase 2B would need to be completed. This temporary pumping could be discharged east to existing infrastructure located within BNSF's right-of-way, thus not requiring it to cross any proposed or existing tracks. Note that this temporary pumping would need to be in service until phase 2C is complete and a gravity outlet is in-place.

6.18.2.3 PHASE 2C

Phase 2C complements Phase 2B, by completing generally the same operations on the final siding tracks north of Phase 2B. Upon the completion of Phase 2C, the railroad yard can resume full operation, with no anticipated shutdowns or construction operations impacting BNSF during the remaining phases. Additional details on each major feature are included below.

6.18.2.3.1 REMOVABLE CLOSURE

The second section of the removable closure is installed under Phase 2C. This continues north from Phase 2B to the north end of the removable closure structure. Also, the remaining railyard grading and rail raises on the siding lines are completed.

6.18.2.3.2 72" RCP

The 72" RCP will be continued from Phase 2B, and installed to the north through siding tracks 6-8. Siding tracks 6-8 will also need to be temporarily shut down with a track window to complete the installation of the proposed stormwater pipe.

6.18.2.3.3 SHEETPILE AND TRACK GRADE RAISE

In addition, the remaining siding tracks 6-8 would have the sheetpile installed below them, as well as the track grade raise during the time of the shutdown under Phase 2C. At the completion of the underground installation near the sheetpile wall, temporary pumping will need to be moved from the manhole located within BNSF property, to the new manhole located between Railway Avenue and BNSF. This will pump into an existing 11’x9’ RBC on the eastern end of the project until the completion of Phase 3B.

6.18.3 PHASE 3

Prior to Phase 3 commencing, all work under Phase 1 must be completed as well as the installation of the 72” pipe through the BNSF railyard. The pump station will need to be operational, or else adequate temporary pumping from STS 2 to STS 1 will need to be incorporated until the pump station is completed. Additional details on each subphase are included below.

6.18.3.1 PHASE 3A

Starting at 9th Street NE and moving east to near 13th Street NE, Phase 3A includes the installation of underground utilities and roadway reconstruction. The proposed sanitary sewer forcemain will also be installed along this stretch with no connections until the completion of Phase 3B/3C.

6.18.3.2 PHASE 3B

Phase 3B continues to the east from where Phase 3A ends, with underground utility installation and roadway reconstruction along Railway Avenue NE, to the eastern edges of the project. All proposed underground utility borings would be completed in this phase as well as the connection of the new sanitary sewer forcemain to the existing Valley Forcemain south of the existing BNSF railyard. The new forcemain will also be connected to the Roosevelt Forcemain under Phase 3B.

In addition, the 4th Avenue NE Tieback Levee (semi-permanent segment) and associated storm sewer drainage is also completed under this phase. This generally includes the installation of multiple large box culverts on both sides of the existing BNSF railyard through the adjacent roadways.

6.18.3.3 PHASE 3C

Prior to beginning Phase 3C, the temporary Sanitary Sewer Bypass Piping Plan will need to be implemented, which is shown in the Construction Drawings in Appendix K. Once the bypass piping plan is completed and operational, all underground utilities can be installed from the eastern edge of MI-1 (western edge of MI-5) to the tie-in location of Phase 3A. Once the sanitary sewer forcemain is connected, the temporary piping plan can be disconnected, and the permanent forcemain utilized.

In addition, with the completion of the Phase 3C activities described above, the existing water transmission line and gravity sanitary sewer line south of the BNSF railyard can be fully removed and/or abandoned, as shown in the Construction Drawings in Appendix K.

6.18.3.4 PHASE 3D

Phase 3D includes installation of underground utilities and roadway reconstruction north on 8th Street NE (where Phase 3C left off) to 6th Ave NE.

6.18.3.5 PHASE 3E

Phase 3E continues the underground utility and roadway reconstruction installed under Phase 3D to the north end of the project, near 7th Avenue NE.

6.18.4 PHASE 4

Although Phase 4 is listed last, there are several items within this phase that can happen concurrently with other phases of the project being constructed. Additional details on each subphase are included below.

6.18.4.1 PHASE 4A

Areas designated as within Phase 4A are areas in which there are few constraints to completing at any time. These include the construction of a portion of the stormwater pond, levees and pedestrian underpass grading, wetside grading, and existing levee removal.

6.18.4.2 PHASE 4B

The floodwall north of the railroad could potentially be completed earlier, but waiting until the railroad operations resume to normal is the direction the preliminary phasing plan recommends. The south floodwall tie-in, as well as the last section of the removable closure, would also be completed under this phase.

6.18.4.3 PHASE 4C

Phase 4C can commence as soon as the existing 16" water transmission line is taken out of service under Phase 3C. This phase will also involve excavating the remaining section of the pond as well as continuing east from Phase 1A with the levee and roadway construction.

6.18.4.4 PHASE 4D

Phase 4D could happen concurrently with Phase 4C, but the intent in this preliminary phasing plan is to keep traffic flowing to Lowe's Garden Center from the west as long as possible. This connection from the west will be accomplished from 14th Street NE until Phase 4D is initiated.

6.19 USACE INSPECTION ITEMS

The USACE performed an inspection of the existing levee system in September of 2017 and developed a *Routine Inspection Report*[80][119], dated August 9th, 2018. Numerous items along the Phase MI-5 reach have been identified as minimally acceptable or unacceptable and include items such as:

- Unwanted vegetation growth
- Encroachments
- Erosion/embankment excavation
- Corrections to culverts or discharge piping

Appendix E7 includes maps and descriptions from the inspection report which outline the deficiencies and required work items. The construction drawings show inspection items on sheet C-102 which will be corrected as part of the project. Table 6-7 below summarizes the corrective actions. Additional design and detail surrounding correction of each individual deficiency is provided in the Construction Drawings in Appendix K.

Table 6-7 USACE Inspection Items

USACE Inspection Deficiency ID (2017)	Remarks (2017)	USACE Recommended Correction	Proposed Correction Phase MI-5
MINL_2017_a_0069	Power pole on levee, vegetation in riprap.	Verify levee easement, relocate encroachments outside of levee easement, unless approved by Corps. Remove unwanted vegetation from vegetation-free zone. Ensure environmental compliance with all appropriate agencies prior to removal.	Existing levee will be removed as part of project.
MINL_2017_a_0070	A drainage ditch has been cut through levee prism. Levee appeared to be constructed out of pervious fill material.	Backfill erosion to the design grade, compact in lifts, and reseed with grass. Verify the type of material used to construct the levee. Impervious material should be used.	Existing levee will be removed as part of project.
MINL_2017_a_0071	Vegetation on landside levee slope and toe.	Remove unwanted vegetation from vegetation-free zone, up to the levee easement. Remove root ball, backfill, compact in lifts, and reseed with grass. Ensure environmental compliance with all appropriate agencies prior to removal.	Existing levee will be removed as part of project.
MINL_2017_a_0072	Encroachments include buildings, tires, material storage, and fences.	Verify levee easement. Relocate encroachments/debris outside of levee easement, unless approved by Corps. Verify approval was received from Corps to perform excavations.	Existing levee will be removed as part of project.
MINL_2017_a_0073	PVC pipe through levee. 2017 NOTE: Vegetation obstructs view of pipe.	Verify levee easement; Relocate encroachments/debris outside of levee easement, unless approved by Corps;	Pipe will be removed as part of project.
MINL_2017_a_0074	Large tree within the vegetation-free zone.	Remove large tree from vegetation-free zone, up to the levee easement. Remove root ball, backfill, compact in lifts, and reseed with grass. Ensure environmental compliance with all appropriate agencies prior to removal.	Existing levee will be removed as part of project.

USACE Inspection Deficiency ID (2017)	Remarks (2017)	USACE Recommended Correction	Proposed Correction Phase MI-5
MINL_2017_a_0075	Guard rail and power pole on levee.	Verify levee easement; Relocate encroachments outside of levee easement.	Existing levee will be removed as part of project.
MINL_2017_a_0076	Large holes dug into levee crown and landside slope, likely for use as fire pits.	Backfill and compact holes. Discourage these activities.	Existing levee will be removed as part of project.
MINL_2017_a_0077	Removed debris from Grade Control Structure.	Resolved	N/A
MINL_2017_a_0078	Pipe has not been televised within the past 5 years.	Televise and verify acceptability.	Will be removed as part of project.
MINL_2017_a_0079	Trees and brush on channel bank and levee riverside slope.	Remove unwanted vegetation from vegetation-free zone, up to the levee easement. Remove root ball, backfill, compact in lifts, and reseed with grass. Ensure environmental compliance with all appropriate agencies prior to removal.	Existing levee will be removed as part of project.
MINL_2017_a_0080	A utility pole is located in the levee prism.	Verify levee easement. Relocate utility pole outside of levee easement, unless approved by Corps.	Existing levee will be removed as part of project.
MINL_2017_a_0081	Hole in emergency levee to expose fire hydrant. 2017 NOTE: Could not locate.	Relocate fire hydrant. Backfill erosion to the design grade, compact in lifts, and reseed with grass.	Existing levee and fire hydrant will be removed as part of project.
MINL_2017_a_0082	Pipe has not been televised within the past 5 years.	Televise and verify acceptability.	Pipe will be removed and storm outfall plugged as part of project.
MINL_2017_a_0084	A fence is located at the landside toe of the levee embankment.	Verify levee easement. Relocate fence encroachment outside of levee easement, unless approved by Corps.	Fence will be removed as part of project.

USACE Inspection Deficiency ID (2017)	Remarks (2017)	USACE Recommended Correction	Proposed Correction Phase MI-5
MINL_2017_a_0085	Trees (> 2 inches in diameter) and long vegetation located on both levee slopes.	Remove unwanted vegetation from vegetation-free zone, up to the levee easement; Remove root ball, backfill, compact in lifts, and reseed with grass; Ensure environmental compliance with all appropriate agencies prior to removal.	Existing levee will be removed as part of project.

7 STRUCTURAL DESIGN

7.1 INTRODUCTION

This section provides the structural design basis for the Project that will apply to the following structures:

- Concrete Floodwall
- Railroad Removable Closure
- 4th Avenue NE Pump Station Substructure
- 4th Avenue NE Pump Discharge Gatewell
- STS 1 Storm Manhole
- STS 2 Storm Manhole
- Cast-in-Place Box Culvert

Roadway features and other non-hydraulic appurtenant structures, as well as culverts and other drainage structures associated only with roads, access roads, and railroads, will be designed according to applicable local, state, and national design criteria and codes and are not addressed in this section of the report.

Non-hydraulic structures, such as the pump station superstructure building, will be designed based on the references listed in Section 18 as well as any other applicable local, state, or federal design criteria and codes.

7.2 TECHNICAL GUIDANCE AND REFERENCE STANDARDS

Engineering design for federal flood risk reduction projects is governed by United States Army Corps of Engineers' (USACE) engineering regulations (ERs), engineering manuals (EMs), engineering technical letters (TLs) and engineering circulars (ECs). See Section 18 - References for a complete list.

7.3 STRUCTURAL FEATURES

7.3.1 CONCRETE FLOODWALL

The reinforced concrete floodwall will be designed to provide a continuous line of protection between the east end of the proposed Phase MI-1 floodwall and future phases of the MREFPP. The top of floodwall elevation was set at a minimum of the modeled 2011 flood level plus hydraulic uncertainty and superiority. The elevation of the top of the floodwall matches the top elevation from the adjacent MI-1 project.

Floodwall will be designed according to EM 1110-2-2100[97], EM 1110-2-2502[101], and ECB 2017-3[87]. The typical cross-section and dimensions of the floodwall are shown in Figure 7-1.

Preliminary stability analysis calculations associated with the floodwall are included in Appendix F1.

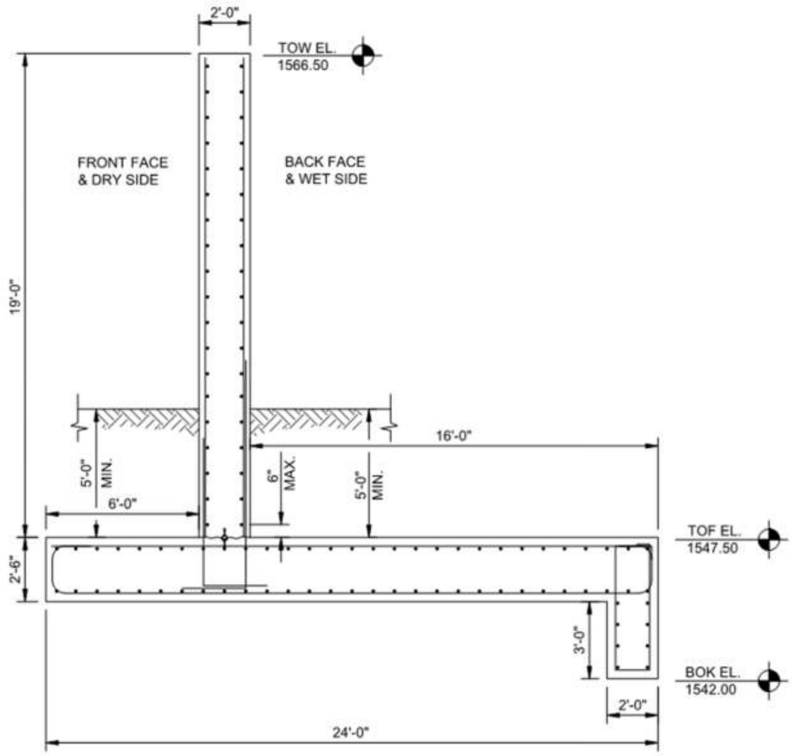


Figure 7-1 Flood Wall Segment A

7.3.2 RAILROAD REMOVABLE CLOSURE

One continuous removable closure will be installed across 4 BNSF rail lines just east of Railway Avenue. Closure structures are defined as closure gates and/or stoplogs where roadways, railroads, or pedestrian paths pass through levees and floodwalls during non-flood conditions.

The railroad removable closure will be a removable aluminum stoplog closure. Designs for the aluminum panels, connections, seals, and miscellaneous features will be submitted by the supplier in accordance with ETL 1110-2-584[114], and the Aluminum Design Manual[6], based on performance standards that are included in the plans and specifications. The closure will be designed to the same events/loadings set forth in the remainder of this report. The closure structure will have a reinforced concrete T-type floodwall on both sides of the railroad opening.

Closure structures (stoplogs) are assumed to be pre-engineered, manufactured aluminum frames covered with a skin plate. The closure consists of stoplogs stacked vertically to a minimum elevation of 1566.50 (the same elevation as the permanent floodwall). The stoplogs are supported by vertical removable posts and kickback support links (if required) placed at a 45^0 angle. The end columns connect to the stoplogs through a support channel in the face of the column. The vertical removable posts are supported by a 3'-6" wide continuous concrete stem, which is part of the footing.

The footing will be designed and included in the Construction Drawings in Appendix K using the provisions set forth in Section 7.8 of this report, ETL 1110-2-584[114], and assumptions on the removable floodwall arrangement and dimensions. However, final design of the stem and columns will be completed after a removable floodwall supplier has been selected and the arrangement and dimensions of the removable floodwall are provided in shop drawings. The design of the footing may also be adjusted at this time to account for differences between the assumed removable floodwall dimensions and those listed in the supplier-provided shop drawings.

The closure structure will be designed with hydrostatic loading to the elevation provided in Table 7-11 for the 750-year event on the river side and with no hydrostatic load on the land side for the unusual event and designed with a hydrostatic load to the top of the wall for the extreme event. The top of the closure structure was set at the modeled 2011 flood level plus hydraulic uncertainty and superiority. The base or sill of the closure structure will consist of a reinforced-concrete foundation supported on soil below frost depth. The closure structure and foundations will be designed for the load cases listed under Section 7.8.5. The sill of the closure includes a step to allow one of the rails to stay open for a longer time period than the other three rails. This reasoning behind the step is further discussed in Section 6.11.4. The footing of the removable closure structure shall be designed to accommodate a potential future raise of the threshold to elevation 1561.10, if the railroad is raised at some time in the future.

A typical cross-section and elevation view of the closure structure foundation are shown in Figure 7-2 and Figure 7-3 respectively.

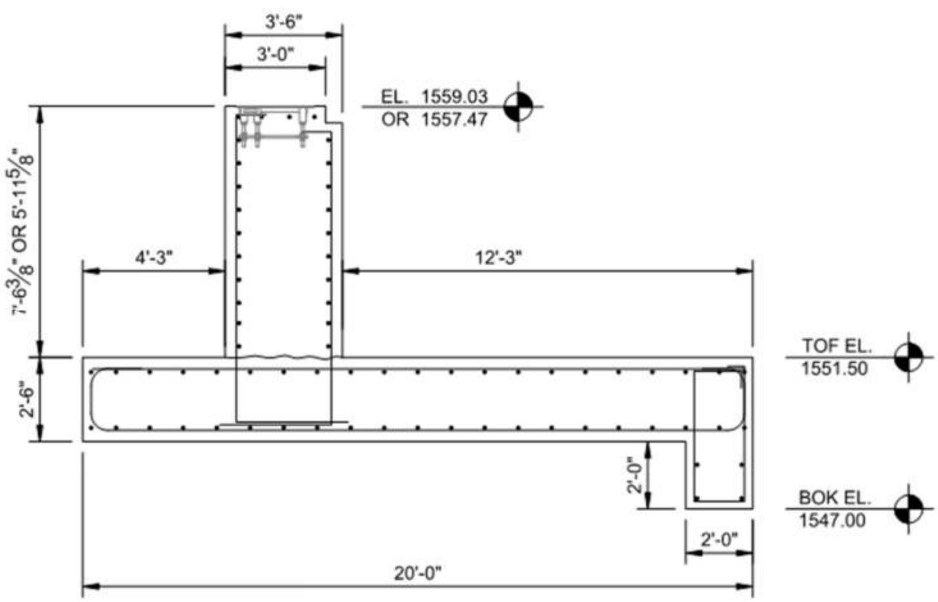


Figure 7-2 Removable Closure Footing

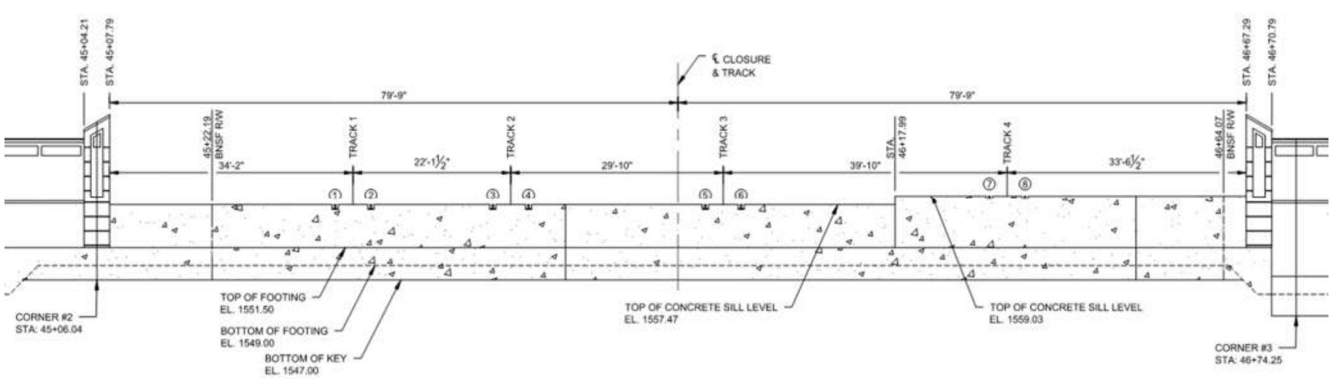


Figure 7-3 Removable Closure Elevation

7.3.3 4TH AVENUE NE PUMP STATION SUBSTRUCTURE

Due to the elimination of the existing gravity outlets of the MI-5 watershed because of the flood protection alignment, a new interior drainage facility will be constructed south of the BNSF Railway and north of the proposed line of protection near River Station (unsteady) 11834+50 (Levee Station 61+00). The 4th Avenue NE Pump Station will be designed as a cast-in-place, reinforced concrete station with a 2,300-square-foot footprint. The pump station has a design pumping capacity of 20,000 gallons per minute and a total station capacity of 30,000 gallons per minute. The pump station was sized to allow for adequate operation of the interior drainage facility during times of gravity outfall conditions and blocked gravity outfall conditions. For more information on hydraulic sizing and interior drainage analysis, see Section 4 of this report.

The reinforced concrete box culvert will route storm water from the improved storm water network through the STS 2 storm manhole and into the pump station. From there the storm water will pass through the trash racks and will enter the wetwell of the pump station where three 10,000-gpm submersible pumps and a sump pump will pump the interior storm water back to the Mouse River via discharge piping to the 4th Avenue NE Pump Discharge Gatewell.

The pump station substructure will be below the final grade, constructed with cast-in-place reinforced concrete, and will be designed to be supported on a reinforced concrete mat foundation. It will contain a masonry building on top of the concrete substructure that houses the pumping, electrical, instrumentation control, and maintenance equipment. Final sizing and structural design of the above ground building has been completed. The above ground masonry building is discussed in detail in Section 6.8 of this report. Detailed design computations associated with the 4th Avenue NE Pump Station Substructure are included in Appendix F3.

7.3.4 4TH AVENUE NE PUMP DISCHARGE GATEWELL

The 4th Avenue NE Pump Discharge Gatewell is located near River Station (unsteady) 11834+00 (Levee Station 61+50) and is approximately 115’ southeast of the 4th Avenue NE Pump Station. The structure will be constructed in the river side of the levee embankment and has a footprint of approximately 770 square feet. The 4th Avenue NE Pump Discharge Gatewell is a multi-level reinforced concrete structure that will collect the storm water discharge flows from the pump station and drain to the Mouse River via an 8’ x 8’ cast-in-place box culvert. The 4th Avenue NE Pump Discharge Gatewell also serves as a gravity overflow bypass for the 4th Avenue NE Pump Station during high flow events.

All pump discharge lines will have a flange mounted flap gates to prevent back flow into the 4th Avenue NE Pump Station. Hinged aluminum access panels will be placed in the top slab above the gates to allow for future maintenance or replacement. Detailed design computations associated with the 4th Avenue NE Pump Discharge Gatewell are included in Appendix F4.

7.3.5 STS 1 STORM MANHOLE

The STS 1 Storm Manholes is a 20’-8” x 20’-8” cast-in-place, single-cell reinforced concrete structure. The structure was oversized to allow for skid steer turn around during storm sewer cleaning operations. The structure serves as a storm junction box for the overflow gravity discharge around the 4th Avenue NE Pump Station. Detailed design computations associated with the STS 1 Storm Manhole are included in Appendix F5.

7.3.6 STS 2 STORM MANHOLE

The STS 2 Storm Manhole is a 22’-4” x 22’-4” cast-in-place, single-cell reinforced concrete structure. The structure serves as a storm sewer junction box that routes the influent water from the north to the 4th Avenue NE Pump Station and the storm water detention pond to the west. During high flow events, the structure routes water to the east through the overflow gravity discharge line through STS 1 and the 4th Avenue NE Pump Discharge Gatewell, where it discharges into the Mouse River. Due to the high invert elevation of the 8’x8’ cast-in-place box culvert, a bollard was designed to hinder forward progress from maintenance equipment and staff during the cleanout process. Detailed design computations associated with the STS 2 Storm Manhole are included in Appendix F6.

7.3.7 CAST-IN-PLACE BOX CULVERT

The US Army Corps of Engineers requires cast-in-place concrete box culvert through the levee prism. As a result, the gravity outfall into and out of the 4th Avenue NE Pump Discharge Gatewell are required to be cast-in-place. For constructability, the short segment of box culvert between the STS 1 and STS 2 Storm Manholes will also be cast-in-place. The 8’ x 8’ box culvert was oversized to allow a skid steer to be used for storm sewer cleanout operations. Data for each section of cast-in-place box culvert can be found in Table 7-1. Detailed design computations associated with the cast-in-place box culvert sections are included in Appendix F7.

Table 7-1 Cast-in-Place Box Culvert Summary

Box Culvert Segment	Box Culvert Location	Box Culvert Size	Maximum Cover (ft)	Minimum Cover (ft)	Length
Segment A	Gatewell to Outfall	Single 8'x8'	21.75	2.50	115.00
Segment B	STS1 to Gatewell	Single 8'x8'	21.55	2.50	204.58
Segment C	STS 2 to STS 1	Single 8'x8'	4.50	3.00	22.83

7.4 PERFORMANCE OBJECTIVES

Guidelines for the design of hydraulic structures exist in numerous USACE manuals with various publication dates.

Performance objectives for this Project will be based on EC 1110-2-6066[84], developed after Hurricane Katrina and the subsequent review of the New Orleans flood risk reduction system.

While EC 1110-2-6066[84] provides guidance specific to I-walls, the performance requirements are applicable to all USACE flood risk reduction structures. The goal for performance is as follows:

- **Normal load events (Usual):** structure is expected to perform in the linearly elastic range with no damage or repairs expected.
- **Less frequent events (Unusual):** minor nonlinear behavior is acceptable, but any necessary repairs are expected to be minor.
- **Low-probability events (Extreme):** structural damage which partially impairs the operational functions are expected and major rehabilitation or replacement of the structure might be necessary, but the structure is expected to accommodate extreme loads without experiencing catastrophic failure.

7.4.1 LOAD CATEGORIES

Table 7-2 is adopted from Table 1 of ECB 2017-2[86] for critical structures. This table generally follows the intent of guidance in EM 1110-2-2502[101] and EM 1110-2-2607[104]. This table should be used in place of Table 3-1 of EM 1110-2-2100[97]. In addition, when selecting criteria requirements from EM 1110-2-2100[97] and ECB 2017-2[86], all hydraulic structures for this Project were considered critical structures with Ordinary site information.

Table 7-2 Load Categories to Satisfy Performance Requirements

Load Condition Categories	Return Period	Annual Exceedance Probability
Usual	10-Year Event	10%
Unusual	10- to 750-Year Event	0.133%
Extreme	Top of Structure	<0.133%

7.4.2 STABILITY CRITERIA

Stability criteria used for structures was in accordance with EM 1110-2-3104[108] and EM 1110-2-2502[101]. The minimum factors of safety for stability of Critical structures with Ordinary site information are listed in Table 7-3 and are in accordance with those listed in Table 8-2 of the MREFPP Design Guidelines[49]. Stability categories are described in more detail in the following sections.

Table 7-3 Required Factors of Safety

Category	Usual	Unusual	Extreme
Sliding	2	1.5	1.1
Bearing	3.5	3.0	2.0
Overturning	100% of Base in Comp.	75% of Base in Comp.	Resultant Within Base
Flotation	1.3	1.2	1.1

7.4.2.1 SLIDING STABILITY

Sliding along a horizontal plane is caused by a differential in hydrostatic elevation and/or soil elevation on each side of the structure. Sliding forces are resisted by shear-friction forces between the potential sliding surfaces and passive soil pressure. The shear-friction forces are developed between the vertical load (caused by gravity of the material) and the shear interface resistance between the horizontal plane of the concrete slab and soil. The factor of safety against sliding is the ratio of the total resisting force to the forces that cause sliding. This factor of safety is determined in accordance with EM 1110-2-2502[101].

7.4.2.2 BEARING STABILITY

Bearing calculations for the floodwall will be completed using the USACE methodology outlined in EM 1110-2-2502[101]. For all other structures the bearing pressure will be calculated as a sum of all applicable vertical loads assumed to act equally over the area of the structure footing. Service loads will be used when calculation bearing pressure exerted by the footing of the structures.

In their 90% Geotechnical Evaluation Report dated December 21, 2018, Braun Intertec provided a net allowable bearing capacity of the soil of 3,000 pounds per square foot for all structures founded below frost depth. It was noted in the report that the allowable bearing pressure was provided with a factor of safety of three and could be increased by 1/3 for occasional transient loads.

7.4.2.3 OVERTURNING STABILITY

Overturning of the structure is checked by limiting the eccentricity of the resultant force with respect to the analyzed surface(s). The overturning forces are the horizontal resultant of lateral loads from differential hydrostatic and soil pressures and uplift forces. Resisting forces are the horizontal resultants of vertical self-weight loads that are multiplied by the friction angle factor with respect to the sliding plane. The at-rest lateral earth pressure coefficient will be used in overturning stability analysis. The overturning stability and bearing pressures of the structures will be checked in accordance with EM 1110-2-3104[108] and EM1110-2-2100[97].

7.4.2.4 FLOTATION STABILITY

Flotation of the structure is due to the uplift pressure on the base slab caused from the hydrostatic pressure from the water elevation. Under balanced water conditions, the uplift pressure is uniform and rectangular. The unbalanced water condition causes a linearly varying pressure dependent on the water elevation. The factor of safety against flotation is the ratio of total downward to upward forces.

7.5 MATERIALS

Below is a summary of the materials used for the structural components. Additional details will be included in later submittals in the structural appendices designated for each structure.

7.5.1 STRUCTURAL STEEL

All structural steel within the structural components is per the specifications of the American Institute of Steel Construction (AISC) Manual of Construction[12]. The minimum yield strength for structural steel is listed in Table 7-4.

Table 7-4 Structural 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
HSS (ASTM A500)(4)	46	58	AISC Table 2-4
Plates (ASTM A36)(3)	36	58	AISC Table 2-5
Bolts (ASTM 325)(5)	N/A	105	AISC Table 2-6

(1) AISC Manual of Steel Construction, 14th Edition[14]
(2) ASTM A992 – Standard Specification for Structural Steel Shapes[28]
(3) ASTM A36 – Standard Specification for Carbon Structural Steel[23]
(4) ASTM A500 – Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes[24]
(5) ASTM A325 – Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength[27]

7.5.2 CONCRETE

Because ground water is anticipated to vary across the project site and many structures will be exposed to moisture and/or groundwater, the class of concrete is set to F2, S0, P1, C1 in accordance with Chapter 4, Durability Requirements, ACI 318[9]. The minimum 28-day compressive strength for reinforced concrete in all structural components is 4,500 pounds per square inch (psi). Concrete mix design requirements (per Chapter 4, ACI 318[9] and ACI 350[10]) are listed in Table 7-5.

Table 7-5 Reinforced - Concrete Material Properties

Component	Designation	Reference
Exposure category and class	F2 (severe)	ACI 318[9], Table 19.3.1.1
Maximum water-to-cement ratio	0.42	ACI 350[10], Table 4.2.2
Minimum 28-day compressive strength	4,500 psi	ACI 318[9], Table 19.3.2.1 ACI 350[10], Table 4.2.2
Nominal maximum aggregate size	¾ inch	ACI 318[9], Table 19.3.3.1
Air content	6% ± 1.5%	ACI 318[9], Table 19.3.3.1

7.5.3 REINFORCING STEEL

All reinforcing steel is per ASTM A615[25] Grade 60, deformed, and uncoated unless otherwise noted on the plans.

7.5.3.1 CLEAR COVER

As per EM 1110-2-2104[99] and ACI 318[9] minimum concrete clear cover is listed in Table 7-6. Concrete clear cover requirements are dependent on location.

Table 7-6 Minimum Concrete Clear Cover

Concrete Location	Minimum Clear Cover (inches)	Reference
Surfaces subject to cavitation or abrasion erosion	6	EM 1110-2-2104[99], Table 2-1
Unformed surfaces in contact with foundation	4	EM 1110-2-2104[99], Table 2-1
Formed and screeded surfaces:		
Equal to or greater than 24 inches in thickness	4	EM 1110-2-2104[99], Table 2-1
Greater than 12 inches and less than 24 inches in thickness	3	EM 1110-2-2104[99], Table 2-1
Equal to or less than 12 inches in thickness	2	ACI 318[9], Table 20.6.1.3.1

7.5.3.2 MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Minimum shrinkage and temperature reinforcing for hydraulic structures will be in accordance with EM 1110-2-2104[99]. It states that the area of reinforcement should be 0.003 times the gross cross-sectional area, half in each face, with a maximum area equivalent to No. 9 bars at 12 inches for smaller joint spacing. As seen in Section 7.5.3.3, the area of reinforcement will increase with larger joint spacing. As joint spacing exceeds 30 feet, the values in Table 7-7 would be used rather than 0.003 listed above.

7.5.3.3 CONTRACTION JOINTS, EXPANSION JOINTS, AND WATERSTOPS

Contraction joints for shallow-founded structures will be placed 20 to 40 feet apart depending on structure height with spacing between expansion joints not exceeding 80 feet. Contraction joints will not be detailed in footings. Expansion joints will also be provided at changes in alignment and offset from the point of intersection a minimum of 5 feet per Section 8.3.1.4 of the MREFPP Design Guidelines[49]. Dowels will be used across expansion joints to prevent undesirable lateral or vertical movement of concrete elements. Waterstops will be embedded in the monolith joints of the floodwalls to stop the passage of water through the joint. Where permanent, frequent, and/or long-term head differential is expected on construction joints, waterstops will be installed. Non-metallic waterstops in accordance with EM 1110-2-2102[98] will be used.

Additional considerations will be given when longer monolith lengths are required to provide a practical design (e.g. road closures, pump stations, gate monoliths, long walls, etc.). Monolith length and joint spacing may dictate the requirements for more shrinkage and temperature reinforcement than the specified minimum. Table 7-7 below provides minimum shrinkage and temperature reinforcement ratios for longer joint spacing of floodwalls.

Table 7-7 Joint Spacing vs. Minimum Reinforcement

Length between Control Joints (ft)	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

7.5.3.4 LAP SPLICES AND DEVELOPMENT LENGTHS

Lap splices and development lengths in reinforcing bars conform to ACI 318[9]. For lap splice and development lengths, refer to Table 7-8. Values are based on ACI 318[9] requirements for f'_c = 4,500 psi concrete and Grade 60 reinforcement.

Table 7-8 Splice and Development Lengths

Splice and Development Lengths					
Walls and Slabs					
Bar Size	Lengths of Lapped Splices for Reinforcement (inches)		Lengths of End Anchorage for Development of Reinforcement (inches)		
#	Top Bars*	Others	Top Bars*	Others	90° Hooks
3	23	18	18	14	6
4	31	24	24	18	8
5	38	30	30	23	10
6	46	36	36	27	12
7	67	52	52	40	14
8	76	59	59	45	15
9	86	66	66	51	18
10	97	75	75	58	21
11	108	83	83	64	23
*Top bars are horizontal bars so placed that more than 12” of concrete is cast in the member below the bar. Horizontal bars in walls need not be provided with lap lengths as required for top bars.					
Note: Values above are for uncoated reinforcing. Lap splice and development lengths shall be multiplied by 1.2 for epoxy-coated reinforcement.					

7.5.4 SOIL PARAMETERS

The following soil parameters were provided by Braun Intertec in the 90% Geotechnical Evaluation Report and were used for all structural design. Soil properties for the clay foundations and backfills used for structural design are shown below:

- Moist Unit Weight (γ_s) 132 pcf

- Buoyant Unit Weight (γ_b) 69.5 pcf
- Friction Angle (θ) 30.0 degrees drained
  0 degrees undrained
- Cohesion (c) 0 psf drained
  2000 psf undrained

As recommended by Braun Intertec, a moist unit weight of 122 pounds per cubic foot was used beneath the floodwall footing for the calculation of allowable bearing as shown in Appendix F1 of this Report.

7.6 DESIGN LOADS

7.6.1 RISK CATEGORY

All structures are considered risk/occupancy Category IV, per ASCE/SEI 7[17], because the structures are considered essential facilities and could pose substantial hazard to the community.

7.6.2 DEAD LOADS

Dead-load unit weights for materials are in Table 7-9. The soil self-weight properties are taken directly from the geotechnical data detailed in Appendix B. Additional dead load information specific to individual structures can be found in the Appendices.

Table 7-9 Dead Load Unit Weights

Dead Load	Unit Weight (pcf)
Reinforced concrete self-weight	150
Non-reinforced structural grout	130
Structural steel self-weight	490
Water self-weight	62.5
Moist Soil	125
Saturated Soil	132
Buoyant Soil	69.5

7.6.3 HYDROSTATIC LOADING

Hydrostatic loading is linear and increases with the fluid depth. Hydrostatic pressure is applied perpendicular to all surfaces regardless of orientation. For the structures in this system, hydrostatic pressures occur laterally on vertical walls or vertically on base slabs. Each is described in the following sections. The design fluid depth is a function of the structure's location relative to the free water surfaces on each side of the line of protection and the load case considered. Hydrostatic lateral and vertical pressures were applied to all structures based on the assumed water level for each load case at a magnitude of 62.5 psf per foot depth.

- Loading to 10-yr event = Usual
- Loading to 100-yr event = Unusual
- Loading to 750-yr event = Unusual (Top of wall/structure is higher than 750-yr event)
- Loading to top of wall/structure = Extreme (750-yr event is lower than the top of wall)

7.6.4 UPLIFT

Uplift pressure may be caused by hydrostatic or seepage pressure. The amount of seepage pressure present is dependent on the duration of the event creating head differential, the permeability of the soil (including cracking in the soil and potential for soil settlement beneath the structure), and the effectiveness of the sheet pile cut-off (if applicable). High-flow conditions will generate differential head for limited durations. In these cases, potential uplift conditions were bracketed around possible high and low uplift loads (no seepage and steady-state seepage). Uplift pressures were calculated in accordance with the methodology outlined in Section 3-3 of EM 1110-2-2200[100]. Analysis showed drains were not needed for structures benefitting from uplift pressures caused by high water levels and reduced loads. The uplift caused by seepage causes a more unfavorable event for the floodwall and therefore, uplift pressures were used. Uplift forces were used in global stability analysis of each structure (sliding, bearing, overturning, and flotation) as well as concrete and reinforcement design of each structure. Service loads were used in the global stability analysis of the structure.

7.6.5 LIVE LOADS

Live loads for the structures were evaluated. These include minimum floor loads for both base and top slabs, along with moving live loads created by vehicular traffic (where applicable). The section below summarizes the live loads used for the evaluation of hydraulic structures.

Structures that will be accessed by vehicles were designed for maximum anticipated vehicle loads. Design surcharge load on soil next to structures related to construction and heavy truck loads was taken as 250 psf except in instances where higher loading is anticipated. Live loading for this project was analyzed in accordance with Section 8.7.2.2 of the MREFPP Design Guidelines[49] and EM 1110-2-3104[108].

The removable closure footing will be designed for Cooper E-80 vehicle loading in accordance with Chapter 8 of AREMA[16].

7.6.5.1 MINIMUM FLOOR LOADS

The minimum floor live loads were determined based on the North Dakota State Building Code for 2014[69], International Building Code Amendments[56], and EM 1110-2-3104[108]. Table 7-10 lists the minimum floor live load values, dependent on floor classification.

Table 7-10 Minimum Floor Loads

Description	Live Load	References
Pump station operating floor	300 psf or HS-20 vehicle	EM 1110-2-3104[108], Table 4-1
Pump station exterior deck	300 psf	EM 1110-2-3104[108], Table 4-1
Buried structures beneath roadway	300 psf	EM 1110-2-3104[108], Table 4-1
Gatewell and minor structure top slabs	200 psf	EM 1110-2-3104[108], Table 4-1

7.6.5.2 MOVING LIVE LOADS

Vehicular traffic may occur adjacent to the vertical walls of the pump station and other structures. A vehicular surcharge load equivalent to an HS-20 vehicle of 300 pounds per square foot (psf), per AASHTO[7] and EM 1110-2-3104[108], will be used. Live load surcharge was applied in accordance with Appendix J of EM 1110-2-2502[101].

Portions of the service floor slab of the pump station will allow vehicular traffic for maintenance. A vehicular load equivalent to an HS-20 vehicle will apply either as a 300-psf uniform load or as point loads per AASHTO[7]. The City of Minot requested the top slab inside the building be designed to resist maximum loading from their vac truck. Final analysis was performed using AASHTO[7] loading; however, additional design checks were also done using the heavier axle loads of the vac truck in anticipated locations to be provided by the City of Minot, and the governing loading was used in the final design. The top slab of the other structures will be located far enough above the surrounding grade to prevent vehicular access and were not considered. Any necessary additional live loading information has been included for each structure in its designated Appendix.

7.6.6 EARTH LOADS

The soil parameters used for stability and capacity were derived from the geotechnical analysis detailed in Appendix B and are listed in Section 7.5.4.

The structures will be surrounded by soils exhibiting both cohesive and cohesionless properties. The soil acts more cohesively when undrained and less cohesively when drained. Both soil states will be conservatively assessed, assuming θ equals 0 for the cohesive (undrained) condition and c equals 0 for the cohesionless (drained) condition.

Lateral and vertical soil loads were computed and applied in accordance with EM 1110-2-2502[101] for shallow or pile founded concrete structures. Because minimal movement or rotation is anticipated, at-rest pressures were applied to the structures per EM 1110-2-2100[97]. However, in sliding analysis of the floodwall footing, in accordance with EM 1110-2-2502[101] and further USACE guidance, passive pressures were used on the resisting side of the footing. As stated in Section 7.6.5.2 construction load surcharge was applied in accordance with Appendix J of EM 1110-2-2502[101] which uses both active and passive pressures.

The following formulas were used to calculate K_a, K_0, and K_P:

$$K_a = \frac{1-\sin\theta}{1+\sin\theta} \qquad K_0 = 1-\sin\theta \qquad K_P = \frac{1+\sin\theta}{1-\sin\theta}$$

Where θ = friction angle of soil

7.6.7 WIND LOADS

Information regarding wind loads is given in Section 6.8.5.3.

7.6.8 SNOW LOADS

Information regarding snow loads is given in Section 6.8.5.3.

7.6.9 ICE, DEBRIS AND IMPACT LOADS

Impact loads include thermally expanding ice and impact loads from floating debris and ice. Thermally expanding ice will be considered for all structures adjacent to static water pools. Impact load values for floating debris and ice and thermally expanding ice can be obtained directly from engineering manuals specific to the structure or computed according to EM 1110-2-1612[94] for all structures subject to moving water flow. For the 100-year event levels, impact loads of 500 lbs/foot will be used per the MREFPP Design Guidelines[49]. No ice loading will be applied for unusual or extreme events higher than the 100-year event. Expansive ice forces of 5,000 lbs/foot will be used for flexible structures and 10,000 lbs/foot for rigid structures at the usual event. Impact loads will be applied uniformly across exposed members. Ice and debris load will be applied at or above the low-water elevation for Usual or Unusual load conditions to produce the maximum effects.

The load factors applied to each ice-load condition are discussed in Section 7.8.3. Ice loads will be considered for all structures.

7.6.10 EARTHQUAKE LOADS

The mapped spectral accelerations based on USGS Seismic Maps for Minot, North Dakota, in zip code 58701 are as follows:

- Short Period, S_s = 0.062g
- 1-second Period, S_1 = 0.022g

Minot does not experience major seismic activity. Lateral seismic forces would be very small and normal water levels are low; therefore, hydrodynamic forces are small and seismic loading would be negligible for design. Consequently, specific seismic design was not required for non-building structures per the MREFPP Design Guidelines[49].

The building structure over the pump station wetwell was designed for the seismic requirements laid out in state and local building codes.

7.7 ADDITIONAL DESIGN CONSIDERATION

7.7.1 WALL THICKNESS

The wall thicknesses are sufficient to allow proper placement and consolidation of concrete. Also, the wall thicknesses were designed in multiples of 2 inches to simplify form tie systems. A minimum wall thickness of 16 inches with two mats of steel was used in walls where lift height exceeded 8 feet.

7.7.2 FROST

The foundations of all hydraulic structures are founded below the design frost depth. This applies to both shallow- and pile-founded structures. The minimum frost depth for foundations is 6 feet below the ground surface for non-heated structures. In order to make sure this requirement is met, Houston Engineering assumed 5' of fill over the top of the footing with a minimum footing thickness of 18". For this reason, frost heave was not included in analysis.

7.7.3 SETTLEMENT

In Braun Intertec's 90% Geotechnical Evaluation Report, settlements of around 1 inch for the floodwall and other minor structures were predicted. Settlements for the cast-in-place box culverts are anticipated to differ between 1 ½ - 3 inches. STS 1 and STS 2 are anticipated to be in the order of 1-2 inches. The 4th Avenue NE Pump Discharge Gatewell is anticipated to settle 2 ½ inches. The 4th Avenue NE Pump Station is anticipated to settle 2 inches if the site is pre-consolidated or settle as much as 6-9 inches without.

7.7.4 SAFETY & CONSTRUCTABILITY

All structures were designed to provide operator and public safety in conformance with EM 385-1-1[88] and applicable codes. The top of the floodwall surface was designed to prevent the public from easy access.

Structures were designed considering constructability issues – both how to make the structure readily constructible as well as how construction procedures may affect the performance of the structure.

7.8 STRUCTURAL EVALUATION AND CAPACITY

7.8.1 DESIGN SOFTWARE

The following software programs were utilized during the final design of the floodwalls, pump station, and other structures:

- IES Visual Analysis 18.0 (finite element analysis software)
- Microsoft Excel
- CTWall
- IES QuickRWall 5.0
- ETCulvert
- PTC MathCAD Prime 3.1

7.8.2 DESIGN HYDRAULIC ELEVATIONS

Due to the size and complexity of the project, design hydraulic elevations for individual structures vary based on location. Table 7-11 below shows various flood design elevations covering the full extents of the project. Detailed discussion on the design hydraulic elevations for each structure can be found in the individual structural appendices.

Table 7-11 Flood Design Elevations

Location	Stationing		Q10 Elevation	CS4 Q100 Elevation	2011 Flood Q750 Elevation
	Project	FIS (Unsteady)			
Start of Floodwall	41+28	19767+50		1554.36	
		(11861+50)	(1545.62)	(1554.19)	(1561.83)
Center of Closure	46+87	19766+62		1554.16	
		(11860+63)	(1545.57)	(1554.25)	(1561.95)
End of Floodwall	46+96	19764+84		1553.91	
		(11858+83)	(1545.52)	(1554.20)	(1561.81)
Gatewell	61+50	19740+50		1552.73	
		(11834+43)	(1543.83)	(1553.54)	(1561.68)

7.8.3 LOAD FACTORS AND LOAD COMBINATIONS

Reinforced concrete was designed for serviceability and strength limit states in accordance with EM 1110-2-2104[99] and the MREFPP Design Guidelines[49]. Serviceability and strength limit states were analyzed using the load factors listed in Table 7-12.

Table 7-12 Concrete Design Load Factors

Limit State		Serviceability		Strength
Load Category		**Usual (U)**	**Unusual (N)**	**Extreme (X)**
Permanent Loads, Lp		γ_U	γ_N	γ_X
Dead	D	2.2[5]	1.6[5]	1.2[1],0.9[2]
Vertical Earth	EV	2.2[5]	1.6[5]	1.35[1], 1.0[2]
Lateral Earth	EH	2.2[5]	1.6[5]	See Note 3
Hydrostatic (Companion Load)	H_S	2.2[5]	1.6[5]	1.0
Gravity (Mud/Ice)	G	2.2[5]	1.6[5]	1.6[1], 0[2]
Temporary Loads, Lt		γ_U	γ_N	γ_X
Peak Hydrostatic – Flood, Drought, Surge, Maintenance (Principal Load)	H_S	2.2[5]	1.6[5]	1.0 or 1.3[4]
Thermal Expansion of Ice	IX	NA	1.6[5]	1.0 or 1.3[4]
Soil Surcharge	ES	2.2[5]	1.6[5]	1.0 or 1.3[4]
Operating Equipment	Q	2.2[5]	1.6[5]	1.0 or 1.3[4]
Live Load (Vertical)	L	2.2[5]	1.6[5]	ASCE 7[7]
Self-Straining	T	2.2[5]	1.6[5]	ACI 318[7]
Vehicle Live Loads	V	2.2[5]	1.6[5]	AASHTO[7]
Dynamic Loads, Ld		γ_U	γ_N	γ_X
Hydrodynamic (Except Seismic)	Hd	2.2[5]	1.6[5]	1.0 or 1.3[4]
Wave	Hw	2.2[5]	1.6[5]	1.0 or 1.3[4]
Wind	W	NA	1.6[5]	ASCE 7[7]
Debris/Floating Ice Impact	I	2.2[5]	1.6[5]	1.0 or 1.3[4]

Table 7-12 Notes:

1. Applied when loads add to the predominant load effect.
2. Applied when loads subtract from the predominant load effect.
3. Load Factors for Lateral Earth Pressure:
   - Structures using at-rest pressure for design
     - Driving Pressure = 1.35; Resisting Pressure = 0.9.
   - All other structures
     - Driving (Active) Pressure = 1.5; Resisting (Passive) Pressure = 0.5
     - Dynamic analysis (response spectra and time history) of earthquake (at-rest pressure) = 1.0
4. Temporary and dynamic Extreme loads shall be designed with:
   - Load factor = 1.3
     - Loads that are physically limited with return periods lower than 3,000 years for normal structures or 10,000 years for critical structures.
     - Loads for which return periods cannot be determined.
   - Load factor = 1.0
     - Loads that are not limited, for which return period can be determined, with design with return periods great than or equal to 3,000 years for normal structures or 10,000 years for critical structures.
5. For members in direct tension (net tension across the entire cross section): Usual load factor = 2.8, Unusual load factor = 2.0
6. Load factors for serviceability limit states are intended to provide designs with stresses in the concrete and reinforcing steel that limit cracking under service loads. The load factors are not reliability based.
7. Where other standards are referenced, load cases and load factors from those standards will be used for design when those loads are primary loads. See load description details in EM 1110-2-2104[99].
8. Load categories not applicable to the Mouse River project were omitted. See Table 3-1 of EM 1110-2-2104[99] for complete list of load categories.

The load factors listed in Table 7-12 were applied within the load combinations shown in the following sections when determining the required nominal strength for all combinations of axial, moment, and shear.

7.8.4 PUMP STATION AND HYDRAULIC CONTROL STRUCTURES LOAD CASES

Load cases for design and stability analyses of the pump station and all other concrete hydraulic control structures were in accordance with the MREFPP Design Guidelines[49] and are listed Table 7-13. Additional guidance was provided in the notes following Table 7-13 for instances where hydraulic structures act as the line of protection and are not embedded in the levee on all sides. These load cases were considered for the 4th Avenue NE Pump Discharge Gatewell but were found not to apply.

Brief descriptions of each load case from EM 1110-2-3104[108] were also provided following Table 7-13. Applicability of each load case is discussed in more detail in the individual structural appendices. Certain load cases were omitted from analysis for individual structures.

Table 7-13 Pump Station Load Combination

Load Combinations	Category
1. Construction	Unusual
2. Normal Operating[1]	Usual
3. Start-up Condition	Usual
4. Pump Stop Condition	Usual
5. High Head Condition	Usual
6. Maximum Design Water Level[2]	Unusual
7. Maintenance	Unusual
8. Rapid Drawdown	Unusual
9. Blocked Trash Rack	Unusual
10. Inundated[3]	Extreme

[1]Assume a Normal Flood Elevation acting on the exterior of the structure (where applicable) taken as the Q100 WSE.

[2]Assume an Infrequent Flood Elevation acting on the exterior of the structure (where applicable). Design shall consider Load Case 5 through 8 from Table 7-14 using Unusual load factors for Load Cases 5 through 7 and Extreme load factors for Load Case 8.

[3]Assume Maximum Design Flood Elevation acting on the exterior of the structure (where applicable)

- Load Case 1: Construction (Unusual)
  Pump station complete with and without fill in place, no water loads inside.
- Load Case 2: Normal Operating (Usual)
  Plant operating to discharge routine local floods over a range of exterior flood levels for which the pumps are operating at approximately 100% efficiency.
- Load Case 3: Start-up Condition (Usual)
  Station empty with water at pump start elevation or maximum pump level.
- Load Case 4: Pump Stop Condition (Usual)
  Water below pump start elevation on intake side, levee design flood on discharge side.
- Load Case 5: High Head Condition (Usual)
  Maximum design water level outside protection line, minimum pumping level inside.

- Load Case 6: Maximum Design Water Level (Unusual)
  Maximum operating floods both inside and outside protection line, maximum pump thrust.
- Load Case 7: Maintenance (Unusual)
  Maximum design water level inside with one, more, or all intake bays unwatered.
- Load Case 8: Rapid Drawdown (Unusual)
  Water at pump stop elevation, sumps dewatered. (Applies to stations inside protection line only.)
- Load Case 9: Blocked Trash Rack (Unusual)
  Five-foot head differential across trashracks.
- Load Case 10: Inundated (Extreme)
- Maximum flood levels inside and outside protection line, pumping station inoperative, foundation drains inoperative, protection line intact.

7.8.5 FLOODWALL LOAD CASES

Load cases for design and stability analyses of the floodwall and closure structure was in accordance with the MREFPP Design Guidelines[49] and listed in Table 7-14. Additional notes have been added to clarify some of the load case requirements. Appendix F1 contains final floodwall stability analysis calculations. Appendix F2 discusses applicability of each load case in more detail for the closure structure. Certain load cases may be omitted from analysis.

Table 7-14 Floodwall Load Combinations

Load Combinations	Type
1. Construction[1]	Unusual
2. Construction + Wind[2]	Unusual
3. Normal Water (10-yr)	Usual
4. Normal Water (10-yr) + Wind	Usual
5. Infrequent Flood	Unusual
6. Infrequent Flood +Wind	Unusual
7. Infrequent Flood + Debris[3]	Unusual
8. Infrequent Flood + Debris + Impact[4]	Unusual
9. MDF	Unusual

1. Evaluated with soil in place and with surcharge loading of 250 psf.
2. Evaluated without soil in place with a wind load of 30 psf.
3. Debris is taken as 0.5 kip/ft uniformly distributed across the floodwall.
4. Impact load is taken as 5-kip load resulting in maximum effect at the waterline.

7.8.6 FINITE ELEMENT ANALYSIS

Visual Analysis 18.0 was used to model and analyze the cast-in-place concrete structures and will be used to design the removable closure footing once a supplier has been selected. Visual Analysis models structures using 2-dimensional plates and member elements with associated material properties and thicknesses or cross-sections.

The Global Coordinate System (X,Y,Z) for each structure is shown in each model view included in the calculations. This is the coordinate notation used to report the location of each node in the model. As the members and plates are modeled, each element is assigned a separate local coordinate system (x,y,z). Plate forces are then reported based on each element's local axes. To clarify in each model analysis, the local coordinate system is indicated.

Visual Analysis utilizes a Load Case Manager, which allows the user to choose from preset factored and service load combinations or create custom combinations. Each structure required careful planning of the service load cases and factored combinations. Descriptions of the load cases and combinations are included in detail in the individual structural appendices.

7.8.7 RESISTANCE FACTORS

For the design of both concrete and steel items, the calculated capacity of the section must be reduced by the associated resistance factor. Strength reduction (resistance) factors from Appendix C of ACI318[9] (corresponding to load factors from the older ACI codes) should be used in the design. Table 7-15 provides a summary of the typical resistance factors used for both concrete and steel per ACI 318[9] and the AISC Steel Construction Manual[12], respectively.

For hydraulic structural steel items, the calculated capacity is reduced by the reliability factor listed in ETL 1110-2-584[114]. All steel components used in this project are assumed to be exposed for inspections and not subject to brackish water. Therefore, the AISC resistance factors are multiplied by 0.9, resulting in a resistance factor of 0.81. This is reflected in Table 7-15.

Table 7-15 Resistance Factors

Material	Design	Resistance	Reference
Concrete	Axial – tension	0.90	ACI 318[9], Table 21.2.2
	Axial – compression	0.65	ACI 318[9], Table 21.2.2
	Shear and torsion	0.75	ACI 318[9], Table 21.2.1
	Flexure – tension	0.90	ACI 318[9], Table 21.2.2
	Flexure – compression	0.65	ACI 318[9], Table 21.2.2
Steel	Axial – tension	0.81	AISC, Chapter D[12]
	Axial – compression	0.81	AISC, Chapter E[12]
	Flexure	0.81	AISC, Chapter F[12]
	Shear	0.81	AISC, Chapter G[12]

7.9 GLOBAL STABILITY

7.9.1 FLOODWALL

The Global Stability results for each load case analyzed is listed in Table 7-16. The factors of safety presented in each table are from the internal floodwall design spreadsheets developed by HEI and checked vs CTWall for accuracy. The design spreadsheets, CTWall models, and QuickRWall models for each load case are provided in Appendix F1. In some cases, when using passive pressures of soil for sliding, the loading from the dry side got large enough where there was more force applied from the dry side due to passive pressures. In these situations, the design spreadsheet produced a negative factor of safety in which case the factors of safety from CTWall were reported. This is the case for all wall sections for the unusual construction case, the usual saturated soil case, and the unusual 100-yr event plus ice for the removable closure. In all of these cases, CTWall produced a factor of safety that was over 100. The factors of safety for both overturning and bearing are still given from the internal design spreadsheet. For removable closure footing sections, the additional footing for kickbacks could not accurately be modeled in CTWall. In the cases where a negative sliding factor of safety value was reported in the internal design spreadsheet, an asterisk is placed meaning the loading from the wet side is not enough to induce passive pressures from the dry side soil. In these cases, we can conclude the factor of safety is significantly higher than that required.

Table 7-16 Global Stability Factors of Safety

Floodwall Extreme Loading Results (Water to top of Wall)			
	Required	Wall Section A	Removable Closure
Sliding FOS	1.10	1.13	1.74
Overturning Base in Compression	10%	87	100
Bearing FOS	2.0	2.07	2.93

Floodwall Unusual Loading Results (750-yr. Event)			
	Required	Wall Section A	Removable Closure
Sliding FOS	1.50	1.85	10.89
Overturning Base in Compression	75%	100	100
Bearing FOS	3.0	3.18	5.06

Floodwall Unusual Loading Results (100-yr. Event + Ice)			
	Required	Wall Section A	Removable Closure
Sliding FOS	1.50	88.9	*
Overturning Base in Compression	75%	100	100
Bearing FOS	3.0	6.07	6.63

Floodwall Unusual Loading Results (Construction)			
	Required	Wall Section A	Removable Closure
Sliding FOS	1.50	174.36	*
Overturning Base in Compression	75%	100	100
Bearing FOS	3.0	5.59	5.69

Floodwall Usual Loading Results (10-yr. Event)			
	Required	Wall Section A	Removable Closure
Sliding FOS	2.0	164.00	*
Overturning Base in Compression	75%	100	100
Bearing FOS	3.5	6.92	3.97

7.9.2 CAST-IN-PLACE CONCRETE STRUCTURES

Table 7-17 through Table 7-20 summarize the global stability results for the cast-in-place concrete structures discussed in this section. The calculations deriving the factors of safety can be found in the Appendix for each individual structure. The values in parenthesis are the required factors of safety as outlined in Table 7-3. Because of the uniformity of fill around the cast-in-place structures, analysis for overturning and sliding were not required.

Table 7-17 4th Avenue NE Pump Station Global Stability Results

	Buoyancy	Bearing
Usual	2.18 (1.30)	3.82 (3.50)
Unusual	1.41 (1.20)	3.38 (3.00)
Extreme	1.75 (1.10)	3.22 (2.00)

Table 7-18 4th Avenue NE Pump Discharge Gatewell Global Stability Results

	Buoyancy	Bearing
Usual	5.05 (1.30)	3.57 (3.50)
Unusual	1.58 (1.20)	3.12 (3.00)
Extreme	1.14 (1.10)	2.92 (2.00)

Table 7-19 STS-1 Storm Manhole Global Stability Results

	Buoyancy	Bearing
Usual	4.85 (1.30)	7.68 (3.50)
Unusual	1.21 (1.20)	5.76 (3.00)

Table 7-20 STS-2 Storm Manhole Global Stability Results

	Buoyancy	Bearing
Usual	2.18 (1.30)	4.50 (3.50)
Unusual	1.23 (1.20)	4.20 (3.00)

7.9.3 CAST-IN-PLACE BOX CULVERTS

Table 7-21 summarizes the global stability results for each segment of the cast-in-place concrete box culverts discussed in this section. The values in parenthesis are the required factors of safety as outlined in Table 7-3. The calculations deriving the factors of safety can be found in Appendix F7. Because of the uniformity of fill around the box culvert sections, analysis for overturning and sliding were not required.

Note: ɫ Denotes load cases that did not control the design, therefore, analysis was not required.

Table 7-21 Cast-In-Place Box Culvert Global Stability Results

		Buoyancy	Bearing
Segments A&B SNGL 8'x8'	Usual	ɫ (1.30)	3.55 (3.50)
	Unusual	1.36 (1.20)	ɫ (3.00)
Segment C SNGL 8'x8'	Usual	ɫ (1.30)	5.75 (3.50)
	Unusual	1.25 (1.20)	ɫ (3.00)

7.10 STATUS

This submittal contains 90% design documentation for all structures included as part of the project. Detailed design information is located in Appendix F of this report. As noted in Section 7.3.2 the final design of the removable closure is subject to change during construction once a final closure system supplier has been accepted and approved. Additionally, the Jib Crane Pad will be designed during construction and will be the responsibility of the Contractor.

8 MECHANICAL DESIGN

8.1 OVERVIEW

Section 6.8 provides mechanical details for all structures located within the 4th Avenue NE Pump Station site, including the 4th Avenue NE Pump Station and the 4^{th} Avenue NE Pump Discharge Gatewell. Appendix G also provides detailed process-mechanical design documentation for the 4^{th} Avenue NE Pump Station. HVAC design documentation is provided in Appendix G also.

9 ELECTRICAL DESIGN

9.1 DESIGN METHODS

The design of the Project is based on the latest National Electrical Code and other codes, regulations, and design manuals as applicable, including those referenced in Appendix G.

9.2 4TH AVENUE NE PUMP STATION SITE

Information regarding electrical design within the 4th Avenue NE Pump Station Site is discussed in Section 6.8 of this report. Detailed design documentation is also provided in Appendix G. Electrical drawings for the site are included in Appendix K.

9.3 STREET LIGHTS

Street lighting will be required along the relocated portion of 4th Avenue NE, 7th Street NE and the shared-use path. This lighting has been designed in accordance with local standards adopted by the City of Minot. Roadway lighting types are shown in Appendix E5.3. Lighting locations are included in the Construction Drawings in Appendix K.

9.4 FLOODWALL LIGHTING

Lighting will be installed on all of the columns of the floodwall. This will be consistent with lighting provided for Phase MI-1. Details related to the floodwall lighting are included in Appendix E8.1. Column lighting details are located in the Construction Drawings in Appendix K.

10 ARCHITECTURAL AND LANDSCAPE ARCHITECTURAL DESIGN

10.1 OVERVIEW & INTRODUCTION

The architectural features will generally involve the building and related features at the 4th Avenue NE Pump Station location. The 4th Avenue NE Pump Station structure is discussed further in Section 6.8. The landscape architectural features include those at the 4th Avenue NE Pump Station site, flood protection aesthetics, the removable closure and shared-use paths. These architectural and landscape architectural features are described as follows.

10.1.1 4TH AVENUE NE PUMP STATION SITE

Information regarding architectural design within the 4th Avenue NE Pump Station site is discussed in Section 6.8. Detailed design documentation is provided in Appendix G of this report.

10.1.2 FLOOD PROTECTION AESTHETICS

Early in the development of Phase MI-1 of the project, a series of superficial treatments of the floodwalls were developed and presented to the City of Minot and SRJB. These concepts took into account the proposed Broadway Bridge replacement aesthetics and the architectural look of the surrounding neighborhood and downtown areas. Due to proximity, the Phase MI-5 floodwall has been designed to have the same aesthetic treatments as in Phase MI-1. A copy of the final proposed concept and supporting documentation is provided in Appendix E8.1.

Concrete form-liners will be utilized during the construction, followed by staining and sealing of the walls to provide desired aesthetics and graffiti protection. Lighting will also be incorporated into the floodwall design, accentuating the columns. The lighting concepts are shown in Appendix E8.1. These features are incorporated into the Construction Drawings in Appendix K.

10.1.3 STRUCTURE AESTHETICS

During the construction of the STS-1, STS-2, and 4th Avenue NE Pump Discharge Gatewell structures, concrete form-liners will be utilized as well. Staining and sealing of the structures will also be completed to ensure proper aesthetics and graffiti protection. These features are shown in the Construction Drawings in Appendix K.

10.1.4 REMOVABLE CLOSURE

In order to provide BNSF railroad access, a removable closure structure will be required on the west end of the project. Aesthetic treatments will be incorporated on the floodwalls on each end. A copy of the concept is provided in Appendix E8.2. This is intended to be consistent with the aesthetics incorporated in the Broadway Park closure in Phase MI-1. A design layout of the removable closure aesthetics has been incorporated into the Construction Drawings in Appendix K.

10.1.5 SHARED-USE PATHS

The project will include shared-use paths to provide public recreation access and allow for maintenance vehicles to access the wetside of the flood protection features. Paths crossing the line of protection are included just south of the floodwall closure and east of the proposed lift station. Proposed layouts for these paths are shown in Appendix K.

10.1.6 RESTORATION

Future use of the area between the flood protection system and the Mouse River is unknown by the Minot Park District at this time. As a result, landscaping in this area will be limited to seeding of disturbed areas and preserving trees where possible and compatible with the flood control project. The seeding provided will be consistent with the guidelines of ETL 1110-2-583. This will include providing a vegetation-free zone surrounding all levees, floodwalls, and critical appurtenant structures in the flood damage reduction system.

Additional details of the seeding areas can be found in the Construction Plans in Appendix K.

10.2 TECHNICAL GUIDANCE & REFERENCE STANDARDS

The architectural designs of these projects are based on International Building Code 2015 [56] as well as codes mentioned in Section 7 and Section 9 of this document.

11 PERMITTING AND REGULATORY

The proposed project is subject to federal, state, and local jurisdiction and regulations. These permits and required approvals are summarized below and are described in more detail in the following sections.

Table 11-1 Potentially Required Permits/Approvals

Agency	Permit/Approval
Federal Permits/Approvals	
U.S. Army Corps of Engineers	National Environmental Policy Act Compliance – Addendum
U.S. Army Corps of Engineers	Section 14 (also known as Section 408)
U.S. Army Corps of Engineers	Clean Water Act – Section 404 - Revision
U.S. Fish and Wildlife Service	Section 7 Concurrence(1)
U.S. Fish and Wildlife Service	Fish and Wildlife Coordination Act Compliance(1)
Natural Resources Conservation Service	Farmland Protection Policy(1)
Federal Emergency Management	Conditional Letter of Map Revision
State Permits/Approvals	
State Historical Society of North Dakota	Section 106 Concurrence(1)
North Dakota State Water Commission	Sovereign Lands Permit
North Dakota State Water Commission	Construction Permit
North Dakota State Water Commission	Drainage Permit
North Dakota Department of Health	Construction General Permit NDPDES
North Dakota Department of Health	Section 401 Water Quality Certification(1)
North Dakota Department of Health	Asbestos Notification of Demolition and Renovation (if applicable)
North Dakota Department of Health	Water and Sanitary Sewer Permits
North Dakota Game & Fish Department	Fish and Wildlife Coordination Act Compliance
North Dakota Department of Transportation	Driveway Permit, project review and approval.
North Dakota Office of the State Engineer	Regulatory Floodways Review Century Code 61-16.2-14
Local Permits/Approvals	
Burlington Northern Santa Fe Railroad	Permission to work in railroad rights-of-way
City of Minot	Floodplain Development Permit
City of Minot	Project Approval
City of Minot – Planning and Zoning Department and Minot Park District	Project review
Ward County Highway Department	General Approval/Coordination – Construction at Roadway Crossings
Ward County Water Resource Districts	General Permit

(1) Completed as part of USACE 404 or 408 process.

11.1 FEDERAL

11.1.1 US ARMY CORPS OF ENGINEERS

11.1.1.1 SECTION 408/NEPA COMPLIANCE

Section 14 of the Rivers and Harbors Act, (Section 408) stipulates that any proposed modification of an existing USACE project must obtain permission from the Secretary of the Army by demonstrating that such proposed alteration or permanent use and occupation of the federal flood control project is “not injurious to the public interest and will not impair the usefulness of such work.” Section 408 requires strict adherence to a variety of both engineering and regulatory requirements, including USACE design standards and design review, and environmental protection compliance in accordance with the National Environmental Policy Act (NEPA). The 408 process allows for the consideration of a nonfederal entity to modify a federal project. The MREFPP proposed features will modify areas within the existing federal project by modifying the existing levees, alignments, altering channel conveyance and the construction of larger levees, or floodwalls.

Agency representatives from the USACE attended agency coordination meetings on several occasions during development. In addition, periodic status calls were held with USACE staff and the 30% design, 60% design, 90% design and 100% design will undergo Agency Technical Review (ATR). The plans will also undergo Independent External Peer Review (IEPR). The SRJB representatives also co-developed the Environmental Impact Study (EIS), and a Record of Decision was issued for the MREFPP at the program level and for Construction Stage 1.5 on December 19th, 2017. Due to project alignment changes for Phase MI-5 and borrow site selection, an addendum to the EIS is being developed and will be included in the 100% submittal. The St. Paul District – St. Paul Office has been the lead for the 408 process and was responsible as the lead federal agency for NEPA. The USACE Regulatory office in Bismarck cooperated in the development of the NEPA document to assure that it is compliant with their needs in permitting under 404.

11.1.1.2 CLEAN WATER ACT – SECTION 404

Section 404 of the Clean Water Act requires approval prior to discharging dredged or fill material into waters of the United States, including jurisdictional wetlands. The term “waters of the United States” has been broadly defined by statute, regulation, and judicial interpretation to include all waters that were, are, or could be used in interstate commerce, such as interstate lakes, rivers, streams (including ephemeral streams), mudflats, wetlands, sloughs, prairie potholes, playa lakes, and ponds. At the MREFPP site, Section 404 jurisdiction will include the Mouse River channel as well as any adjacent wetland areas or tributaries. Temporary or permanent impacts to these resources will require Section 404 authorization and associated mitigation.

The North Dakota Department of Health requires quarterly discharge monitoring reports for permitted activities. The North Dakota Department of Health, which also holds water quality certification authority under Section 401 of the Clean Water Act, will coordinate with the USACE to certify the 404 permit, if required. This process is combined with the USACE permit process. The Mouse River within the project limits is not listed as impaired.

Based on consultation with the USACE as part of the permitting process for Phases MI-1 through MI-3, it was recommended that permitting for these three phases be handled in a combined nature as part of what is referred to as Proposed Construction Stage 1.5 by the SRJB. When completed, Proposed Construction Stage 1.5 would allow for FEMA accreditation and removal of properties lying north of the river in this area from the regulatory floodplain. The features in Proposed Construction Stage 1.5 are only a subset of the project features of the MREFPP and were included in the Final Programmatic Environmental Impact Statement developed for the USACE 404 and 408 approval process.

Proposed Construction Stage 1.5 includes the following features;

- Terracita Vallejo
- Highway 83 Bypass Bridge Replacement (to be permitted separately by NDDOT)
- Phase MI-2 – Napa Valley
- Phase MI-3 – Forest Road
- Minot Water Treatment Plant (Permitted previously under Section 404)
- Phase MI-4 – Maple Avenue High-Flow Diversion
- Broadway Bridge Replacement (to be permitted separately by NDDOT)
- Phase MI-1 – 4th Avenue NE
- Phase MI-5 – 4th Avenue NE Tieback Levee

Both temporary and permanent impacts are anticipated from the construction of these features. Project impacts are described in the original US Army Corps of Engineers Section 404 permit application submitted August 16th, 2017 and approved on February 16th, 2018. Impacts were estimated at 1.29 acres of temporary impacts (0.46 for wetlands and 0.83 for waters) and 9.56 acres of permanent impacts (1.21 for wetlands and 8.35 for waters). However, due to updates in the updated MI-5 alignment (as discussed in Section 1.6 of this report) since the original application and 404 approval, an addendum to the USACE Section 404 permit submission for Proposed Construction Stage 1.5 is being developed and will be included in the 100% submittal. Estimated 90% level of design wetland and OHWL impacts are described in more detail in Section 5 and are included in Table 5-1 and Table 5-2. Maps of the wetland and OHWL impacts anticipated at 90% are included in Appendix H3.2. Final impacts will be included in the final permit application documents and in the 100% submittal. Maps of the wetland and OHWL impacts anticipated at 90% are included in Appendix H3.2. Final impacts will be included in the final permit application documents and in the 100% submittal.

11.1.2 US FISH AND WILDLIFE SERVICE – SECTION 7 OF THE ENDANGERED SPECIES ACT

Under Section 7 of the Endangered Species Act (ESA) of 1973, Federal agencies are required to ensure that agency actions are not likely to jeopardize the continued existence of any listed species or result in the destruction of adverse modification of critical habitat (16 U.S.C. § 1536(a)(2)). Listed species include endangered and threatened species. According to the USFWS, an endangered species is one that is in danger of extinction throughout all or a significant portion of its range, and a threatened species is one that is likely to become endangered in the foreseeable future. A candidate species is a plant or animal on which the USFWS has sufficient information to propose it as threatened or endangered under the ESA, but which they have not yet proposed because other listing activities take precedence. While candidate species are not legally protected under the ESA, it is consistent with the intent of the ESA to consider that these species have significant value and merit protection.

As part of the Final Programmatic EIS dated July 2017, Section 7 consultation was completed with the USFWS to discuss potential impacts to the species, in accordance with the Endangered Species Act of 1973. The Dakota skipper and the northern long-eared bat were identified as two federally-listed (threatened) species with potential habitat in the project area covered in the Programmatic EIS. However, based on the project design included in the EIS, a “no effect” determination for federal-listed species was made; therefore, no further consultation was required.

Additional review of the revised current MI-5 alignment and proposed borrow source will be completed as part of the EIS addendum described in Section 11.1.1.1. This is expected to include a supplemental raptor nesting site inventory study in the Spring of 2019 to address comments provided by the USFWS and North Dakota Game and Fish Department as part of the EIS.

No separate application will be needed from the SRJB.

11.1.3 US FISH AND WILDLIFE SERVICE – FISH AND WILDLIFE COORDINATION ACT COMPLIANCE

Coordination with the U.S. Fish and Wildlife Service is also required for the Fish and Wildlife Coordination Act Compliance. This will ensure protection and conservation of fish and wildlife resources. This coordination will be completed as part of the 404/NEPA processes, as applicable.

No separate application will be needed from the SRJB.

11.1.4 NATURAL RESOURCES CONSERVATION SERVICE – PRIME FARMLAND

Prime Farmland has a formal definition set by the US Congress related to soil properties impacting crop production. Farmland classification identifies map units as prime farmland, farmland of statewide importance, farmland of local importance, or unique farmland. It identifies the location and extent of the soils that are best suited to food, feed, fiber, forage, and oilseed crops. NRCS policy and procedures on prime and unique farmlands are published in the "Federal Register," Vol. 43, No. 21, January 31, 1978.

Many areas within the project boundary of the overall MREFPP contain soil units that are identified as prime farmland.

Although prime farmland soils are present within Phase MI-5, croplands are not present, thus impact to prime farmland is not anticipated. Coordination on this will be completed as part of the 404/NEPA processes and thus no separate application from the SRJB will be needed.

11.1.5 FEDERAL EMERGENCY MANAGEMENT AGENCY – CLOMR/FLOODPLAIN DEVELOPMENT PERMIT

FEMA has defined a regulatory floodplain and floodway for the Mouse River in the Effective Flood Insurance Study (FIS), dated February 15, 2002. However, currently, the NDSWC and FEMA are working on a revision to the Ward County FIS. The preliminary version of this updated Ward County FIS would change the discharge frequency curve such that the 1% annual chance discharge would go from 5,000 cfs to 10,000 cfs.

The SRJB coordinated with FEMA to prepare a CLOMR for work identified as Construction Stage 1.5 including Phase MI-5. This CLOMR was approved on October 16, 2017. A copy of the FEMA approval is included in Appendix H2.1. An addendum will be included in the 100% submittal due to project alignment changes associated with MI-5.

Similarly, a Floodplain Development Permit was also submitted and approved for Construction Stage 1.5 on January 2, 2018. Additional details related to the Floodplain Development Permit are included in Section 11.3.2.1.

11.2 STATE

11.2.1 NATIONAL HISTORIC PRESERVATION ACT (NHPA) – SECTION 106

Regulation 36 CFR Section 800.2(c)(2)(ii)(A) requires that the Federal agency "ensure that consultation in the section 106 process provides Native American tribes… a reasonable opportunity to identify its concerns about historic properties, advise on the identification and evaluation of historic properties, including those of traditional religious and cultural importance, articulate its views on the undertaking's effects on such properties, and participate in the resolution of adverse effects. It is the responsibility of the agency official to make a reasonable and good faith effort to identify tribes… that shall be consulted in the section 106 process."

As the lead federal agency, the USACE therefore needs to make a good faith effort to consult with tribes that have ancestral ties to the geographic area the project is located in, including those tribes that no longer reside in the area, to determine if any properties of traditional religious and cultural importance to a tribe which meet the National Register criteria are located in the project's area of potential effect and to resolve potential adverse effects to them by the project. A Programmatic Agreement was executed between the USACE, ND SHPO, and the Souris River Joint Water Resource Board to cover effects that cannot be fully determined in advance of the undertaking.

Representatives from the ND SHPO participated in agency coordination meetings on January 29th, 2015, and May 27th, 2015. ND SHPO staff will review reports and provide guidance. In May 2015, a Class I cultural resources survey identified all known archaeological and historic structures within one mile of the Mouse River corridor from Minot to Burlington. Additionally, in May 2015 a Class III Standing Structures Survey and a Class III Archaeological Survey were completed for the construction footprint of Phases MI-1, MI-2/3 and a portion of the MI-5 footprint. Additional Class III surveys covering the remaining area of MI-5 have been completed, were submitted to the USACE for review, and have been submitted to ND SHPO. The survey titled *Mouse River Enhanced Flood Protection Project Construction Stage 1.5 Class III Archaeological Investigation*[2] dated August 2017 is with SHPO, as well as the survey titled *Mouse River Enhanced Flood Protection Project Construction Stage 1.5 Class III Architectural History Inventory*[3] dated August 2017.

A Previous Class III Cultural Resource Investigation at the proposed MADC borrow site was completed in December of 2016 by Ackerman-Estvold and is included in Appendix E4.2.1. This report recommended that a finding of *"No Historic Properties Affected"* be determined. This was reviewed by SHPO and a concurrence letter was issued by SHPO on February 6, 2017. This letter is included in Appendix E4.2.2.

More information related to the Cultural Reviews are also included in Section 5.

Coordination related to Section 106 will be completed as part of the 404/NEPA processes and thus no separate application from the SRJB will be needed.

11.2.2 NORTH DAKOTA STATE WATER COMMISSION

11.2.2.1 SOVEREIGN LANDS PERMIT

The beds of navigable lakes and streams are owned by the public, and the ND State Engineer is statutorily charged with the responsibility of managing those lands and regulating activities that impact those lands. As included by the ND State Water Commission on the state navigable waters list, the Mouse River has been determined to be navigable and thus any projects impacting the bed of the river below the Ordinary High-Water Level will require authorization from the State Engineer.

Furthermore, representatives from the ND State Water Commission participated in agency coordination meetings on January 29th, 2015, and May 27th, 2015. Input from these meetings indicated that the work within the bed of the Mouse River would require a ND Sovereign Lands Permit. Field studies documenting the Sovereign Lands boundary will not be required.

A permit application will be submitted to the NDSWC for Sovereign Lands approval. A copy of the application will be included in Appendix H in the 100% submittal.

11.2.2.2 CONSTRUCTION PERMIT

A construction permit is required from the North Dakota State Engineer for the construction or modification of any dam, dike, or other device with a diverting capacity greater than 50 acre-feet of water. The diverting capacity of a dike is calculated based upon the area protected as measured from the effective top of dike; if the absence of the dike could result in more than 50 acre-feet of water inundating the protected area, a permit is required. The application process involves submitting the application form provided by the State Engineer as well as preliminary plans.

Representatives from the NDSWC participated in agency coordination meetings on January 29th, 2015, and May 27th, 2015. Input from these meetings indicated that the construction of levees and floodwalls would require a NDSWC Construction Permit. A permit application will be submitted to the NDSWC for a construction permit. A copy of the application will be included in Appendix H in the 100% submittal.

11.2.2.3 DRAINAGE PERMIT

Water Resource Districts (WRD) and the Office of the State Engineer (OSE) are responsible for regulating drainage in North Dakota as authorized under North Dakota Century Code title 61.

North Dakota Century Code (NDCC) differentiates between permitting processes for surface drainage and subsurface water management systems (a.k.a. drain tile systems).

A permit is required before draining a pond, slough, lake or sheetwater, or any series thereof, that has a watershed area (i.e., drainage area) of 80 acres or more. A permit is also required for the installation of a subsurface water management system comprising 80 acres of land area or more.

No actions in Phase MI-5 will exceed these thresholds and thus no Drainage Permit will be required.

11.2.3 NORTH DAKOTA DEPARTMENT OF HEALTH

11.2.3.1 CONSTRUCTION GENERAL PERMIT (NDPDES)

Administered through the NDDH, construction general permits under the North Dakota Pollutant Discharge Elimination System (NDPDES) are issued based on an application that can characterize adequately the project impacts and areas of discharge along with protective measures.

The construction of Phase MI-5 proposed features would result in the disturbance of over one acre of land triggering the requirement for a NDPDES Permit. An application will be submitted once the contractor for the project has been determined.

11.2.3.2 SECTION 401 WATER QUALITY CERTIFICATION

The Section 401 Water Quality Certification, as a component of the Section 404 Permit application, ensures compliance with Section 401 of the Clean Water Act. Specifically, an activity that may result in any discharge into waters of the United States is required to obtain a certification from the NDDH.

Section 401 certification will occur as part of the USACE Section 404 permit and thus no separate application will be required from the SRJB.

11.2.3.3 ASBESTOS NOTIFICATION OF DEMOLITION AND RENOVATION

The North Dakota Department of Health Asbestos Control Program is regulated by the Division of Air Quality. This division enforces the Emission Standards for Asbestos under the North Dakota Air Pollution Control Rules.

As part of this program, regulations require that all affected parts of a facility being renovated or demolished must be inspected by a state-certified inspector for the presence of asbestos-containing materials prior to beginning a renovation or demolition project. In addition, all regulated asbestos-containing material that will be disturbed as part of a renovation or demolition must be properly removed by state-certified individuals before beginning the project. All asbestos-containing waste material must be properly disposed of in an approved landfill and the Notification of Demolition and Renovation (SFN17987) form must be submitted to the Department ten days prior to beginning any demolition activity.

Currently all demolition of structures is being completed by others in advance of this project and thus no application is anticipated from the SRJB.

11.2.3.4 WATER AND SANITARY SEWER PERMITS

The NDDH conducts the Drinking Water Program in the state of North Dakota to implement and regulate the standards for water quality set by the US Environmental Protection Agency (EPA) under the Safe Drinking Water Act (SDWA). The Drinking Water Program works with all public water systems in North Dakota to ensure that they provide safe drinking water. One of the ways this is accomplished is plans and specifications review of water and sanitary improvements. Plans and specifications review ensures that all new or modified public water system facilities meet established state design criteria prior to construction.

A copy of the final plans and specifications will be provided to the NDDH for their review.

11.2.4 NORTH DAKOTA GAME AND FISH DEPARTMENT

Coordination with the North Dakota Game and Fish Department will be completed as part of the Fish and Wildlife Coordination Act Compliance. This will ensure protection and conservation of fish and wildlife resources. This coordination will be completed as part of the 404/NEPA processes, as applicable.

Aquatic Nuisance Species (ANS) rules were also enacted by North Dakota Game and Fish Department in 2008. These regulations are to prevent the introduction of undesirable species of plant and animals. The contractor will be required to provide the Department a reasonable opportunity to inspect any equipment that will be used in the Mouse River.

No separate application will be needed from the SRJB.

11.2.5 NORTH DAKOTA DEPARTMENT OF TRANSPORTATION

No North Dakota Department of Transportation permit is currently anticipated to be required for this project since no state roadways will be impacted.

11.2.6 NORTH DAKOTA OFFICE OF THE STATE ENGINEER

11.2.6.1 REGULATORY FLOODWAYS REVIEW

N.D.C.C. § 61-16.2-14 requires that the community responsible for permitting or authorizing a use in a regulatory floodway must notify the State Engineer of the proposed use before issuing the permit or authorization. As a result, the City of Minot Floodplain Administrator submitted an Application for the State Engineer's Floodway Review before issuance of the Floodplain Development Permit for Construction Stage 1.5. An amended approval from the State Engineer will be required by the City of Minot before issuing an amendment to the Floodplain Development Permit to address changes to the alignment of MI-5.

This approval will be obtained by the City of Minot Floodplain Administrator and thus no separate application will be required by the SRJB.

11.3 LOCAL

11.3.1 BNSF RAILWAY

Due to the proximity of flood control features to areas near/on BNSF Railway jurisdiction, coordination and/or approval by this entity will be required.

The construction of the project will result in impacts within the BNSF Railroad right-of-way. Coordination and/or approval with BNSF will be required. Due to proposed work within the right-of-way, contractors will need to obtain all permits and insurance required by the railroad to perform work within the railroad right-of-way. Due to permanent impacts, easements or land acquisitions will need to be coordinated with the railroad.

11.3.2 CITY OF MINOT

11.3.2.1 FLOODPLAIN DEVELOPMENT PERMIT

A Floodplain Development Permit is required from the City of Minot's Engineering Department (Floodplain Administrator), for construction within the regulatory floodplain as defined in prior sections. A permit application for Construction Stage 1.5 was submitted and approved in January of 2018. A copy is included in Appendix H6.1. However, due to project alignment changes, an addendum for Phase MI-5 is required and will be included in the 100% submittal.

11.3.2.2 PROJECT APPROVAL – ENGINEERING DEPARTMENT

Phase MI-5 will involve the replacement of public and franchise utilities as well as the realignment and reconstruction of city streets. Approval from the City Engineering Department would be necessary for this work.

11.3.3 CITY OF MINOT PLANNING AND ZONING DEPARTMENT AND MINOT PARK DISTRICT

Recreational features in Phase MI-5 include multiple shared-use paths.

The Phase MI-5 footprint is within a future regional greenway system which could result in significant benefits by providing more recreational opportunities through increased access and connections with trails, parks, and river activities. The project plans have been presented to the Minot Park District, and the

90% submittal will be provided to the Minot Planning and Zoning Department to ensure consistency with their programs.

11.3.4 WARD COUNTY

11.3.4.1 WARD COUNTY HIGHWAY DEPARTMENT

Phase MI-5 will involve modifications to Railway Avenue on the east end of the project north of the existing BNSF railroad. These improvements will extend outside the city limits of Minot and thus approval for the proposed modifications will be required from the Ward County Highway Department.

11.3.4.2 WARD COUNTY WATER RESOURCE DISTRICT

Phase MI-5 will involve project features that will extend outside the city limits of Minot and thus plan review and approval for the proposed modifications may be required from the Ward County Water Resource District.

12 REAL ESTATE

This section describes the real estate requirements for construction of the Project.

12.1 PARCEL ACQUISITIONS

The City of Minot is currently in the process of acquiring property needed for construction, operation, and maintenance of the flood risk management system. Figure 12-1 indicates the status of parcel buyouts and anticipated acquisitions as of October 1st, 2018. Additionally, several areas require temporary and/or permanent easements and are shown on the real estate drawings in the Construction Drawings located in Appendix K.

12.2 EXISTING PROPERTY INFORMATION

To determine legal property boundaries, property surveys were completed through the Phase MI-5 area. Property corners were recovered along the reach, and property lines and parcel boundaries established by North Dakota professional land surveyors in accordance with generally accepted practice and state law.

Property ownership data was developed using a GIS database supplied by the City of Minot. This information shows approximate property boundaries and corresponding property owner information. Property ownership will be verified as part of the final acquisition process.

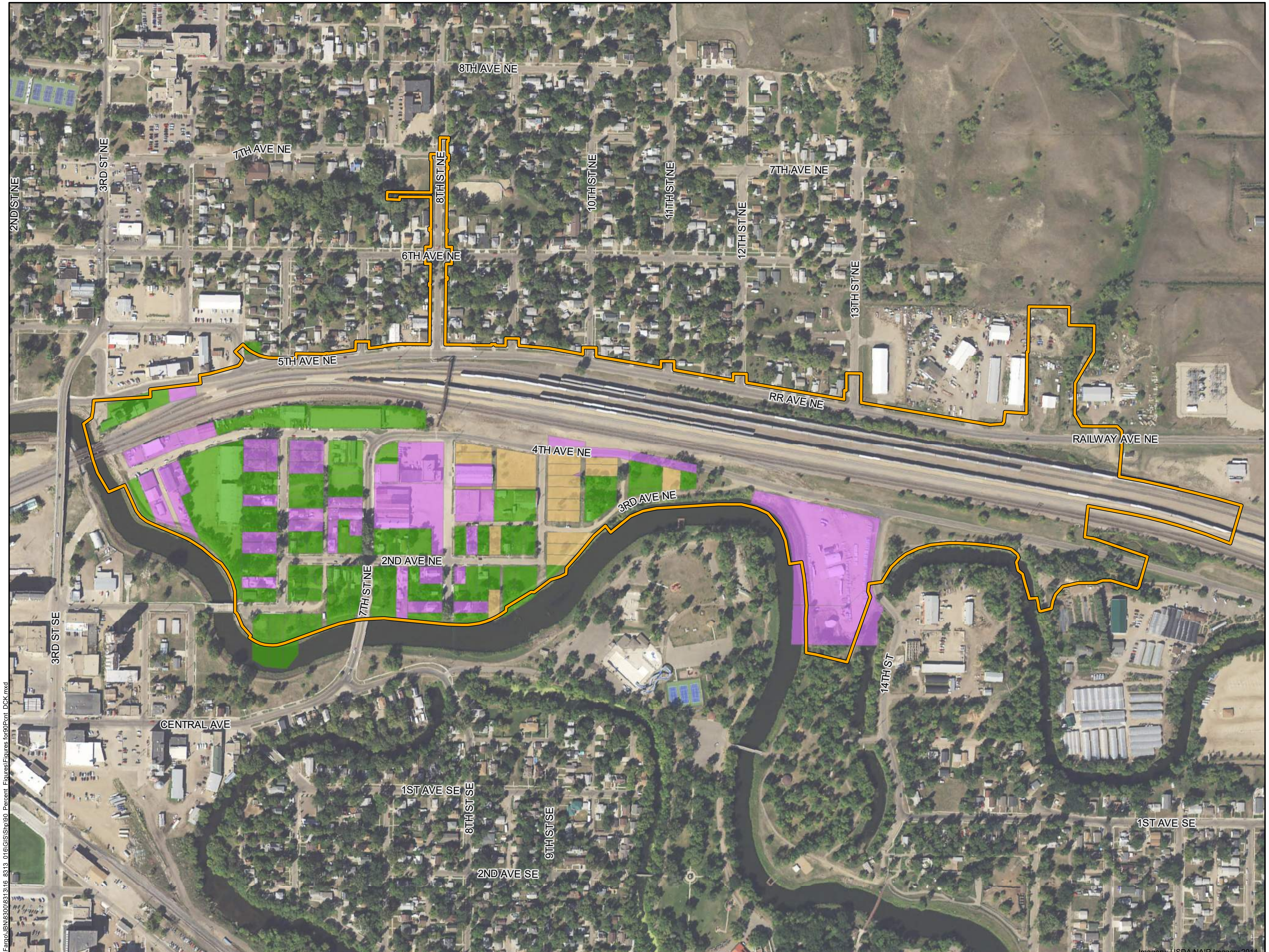
The horizontal datum for the Project has been established as North Dakota State Plane, North Zone, North American Datum of 1983, with U.S. survey feet as the unit of measure. This datum and unit of measure is consistent with the information currently being used by the City of Minot, Ward County, and the USACE.

Parcel and property information on construction drawings is shown in the project coordinate system.

12.3 USACE RIGHT-OF-WAY

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

- **Floodwalls:** Real estate surrounding floodwalls to provide access for operation and maintenance.
- **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.
- **Closure Structure and Sheetpile Cutoff Wall:** Real estate surrounding the closure structure and sheetpile cutoff wall to provide access for operation and maintenance of these features.
- **Pump Station and related structures:** Real estate surrounding the pump station and related structures (pond, STS 1 and STS 2) to provide access for operation and maintenance of these features.
- **Gatewell**: Real estate surrounding the gatewell to provide access for operation and maintenance of this feature.



Buyout Parcels (10/1/2018)

BOUGHT

MINOT PARK DISTRICT

NEED TO OFFER

Phase MI-5

0 205 410 820
Feet

Houston Engineering Inc.

Figure 12-1

CITY OF MINOT
BUYOUT PARCELS
Revised for 90%
Basis of Design Report
Mouse River Enhanced Flood
Protection Project
Phase MI-5
Minot, North Dakota

The extents of the proposed flood protection right-of-way are included in the Real Estate drawings in Appendix K.

12.4 MUNICIPAL RIGHT-OF-WAY

Several municipal city streets and utilities, along with corresponding public right-of-way, will be modified as part of the flood risk management project. It is anticipated that new public right-of-way will be dedicated for the long-term operation and maintenance of these features. This platting of new public right-of-way will be completed by the City in the future, however, appropriate real estate for the features will be obtained as part of this project.

12.5 PERMANENT UTILITY EASEMENTS

As a part of the utility modifications associated with this project, portions of the existing utility networks will be relocated onto private property. Permanent utility easements or fee titles will be acquired 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, storage and transport of materials, and clearance for construction activities. Temporary easements will be in effect until final acceptance of the work. Locations of required Temporary Construction Easements at the project site are included in the Real Estate Drawings in Appendix K.

12.7 REAL ESTATE REQUIREMENT TABULATION

The USACE Real Estate Division requires tabulation of real estate requirements for the Project. Based on the current design configuration, the real estate requirements are presented in Table 12-1. Additional information is presented in Appendix I1 as part of the Real Estate Summary. Real Estate Drawings are in Appendix K. The SRJB and the City of Minot are currently acquiring all necessary property in fee title or easements prior to construction.

Table 12-1 Real Estate Requirements

Real Estate Description	Estimated Area
Construction Temporary Easement	3.347 Acres
Fee Title	0.000 Acres
BNSF Temporary Construction Easement	15.953 Acres
Required BNSF Easements	7.155 Acres
USACE Project Permanent ROW	19.683 Acres

13 OPINION OF PROBABLE CONSTRUCTION COSTS

This opinion of probable cost (OPC) is intended to provide information for consideration during decision-making and financial planning at this 90% submittal design stage for Phase MI-5 of the Mouse River Enhanced Flood Protection Project (MREFPP). 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, based on the alignment and investigations available at this time. Information from the recent Phase MI-1 and Phase MI-2/3 bids, along with North Dakota and Minnesota Departments of Transportation unit prices, was utilized in the development of this OPC.

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

13.2 OPINION OF COST BREAKDOWN

Preliminary cost estimates were developed for the following primary elements of the Project, similar to the PER from 2012 and OPCs completed for previous phases such as Phase MI-1 of the MREFPP.

- Removals
- Flood Mitigation
- Pavement
- Watermain
- Sanitary Sewer
- Storm Sewer
- Franchise Utilities
- Ecological Mitigation
- Cultural Resources
- Hazardous, Toxic, and Radioactive Waste (HTRW)
- Miscellaneous

The format of the cost estimate tables changed at 60%. For the 60% and 90% Draft estimates, the format reflects a proposed bid tabulation table for the project, which is different from the categories listed above and the table format presented in the 30% deliverable. The categorical items listed above are included in the 90% OPC but are no longer summarized by work category as shown above.

Review performed since the 60% OPC resulted in design modifications that have been incorporated into the 90% OPC. The more significant changes to the OPC from 60% to 90% are listed below.

- Realignment of the closure structure across BNSF.
- Added grade and alignment modifications to BNSF railroad.
- Added Temporary RR facilities to allow for project staging while maintaining RR operations.
- East end semi-permanent tieback levee system raised from providing 100-yr to 2011 flood level protection.
- Added a royalty fee of $1.00 per cubic yard assumed for material from the borrow site.
- Added additional multi-use paths.
- Added maintenance access path and loading facility.

- Updated franchise utility relocation costs.
- Watermain crossing the Mouse River and BNSF railroad.
- Excluded Planning, Engineering, and Design assumed equal to 12-percent of construction cost.

13.3 OPINION OF PROBABLE COST SUMMARY

The OPC is summarized in Table 13-1.

Table 13-1 Opinion of Probable Cost Summary

Item	OPC Anticipated Accuracy Range, Low (-5%)	OPC Phase MI-5 (Point Estimate)	OPC Anticipated Accuracy Range, High (+10%)
Estimated Construction Cost (1)(2)(3)(4)	$45.02 Million	$47.39 Million	$52.13 Million
Lands and Easements		TBD	
Planning, Engineering, and Design (PED) (Assume 12%)		Not Included	
Construction Management (CM) (Assume 7%)		Not Included	
Total Opinion of Cost	$45.02 Million	$47.39 Million	$52.13 Million

(1) Includes 10% contingency.
(2) Does not include temporal escalation costs, O&M costs, relocations, or betterments.
(3) Does not include acquisition of lands and easements.
(4) Does not include planning, engineering, design or construction management.
(5) Numbers rounded to the nearest $0.01 million.

13.4 OPINION OF PROBABLE COST CONSIDERATION

The OPC was developed based on detailed designs, unit prices that are benchmarked against MI-1, MI-2/3, and other regional prices for similar construction scopes, and engineering judgment. The OPC is based on 90% design alignments, quantities, and unit prices. Costs will change with further design. A contingency of 10% for construction costs has been used based on referenced projects and published references. Time value of money escalation costs are not included.

Operation and maintenance costs are not included. The OPC is a point estimate ($47.39 million) within an estimated accuracy range. The estimated accuracy range for the total project cost, as Phase MI-5 is defined, is -5% to +10%, or between $45.02 million and $52.13 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 MI-5 as currently scoped or risk contingency. A two-year construction duration is assumed. As design progresses, estimated costs will change. Due to the magnitude of mechanical and electrical work on the project it will require separate bids as dictated by section 48-01.2-02 of the ND Century Code.

The OPC is considered a construction bid estimate and has been developed on the basis of similar projects and the HEI team's experience and qualifications. The estimate represents our best judgement as experienced and qualified professionals familiar with Phase MI-5, based on Phase MI-5-related information available, current information about probable future costs, and a 90% draft development of design for Phase MI-5. 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.

With limited discussion with BNSF Railway, the quantities established based on the current 90% Design have not been commented on by BNSF, and significant changes to railroad design could be forthcoming. The design as shown in the Construction Drawings in Appendix K is currently designed to BNSF standards, but future discussions may warrant changes.

Since the HEI 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 HEI team cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from the OPC.

14 DRAWINGS AND TECHNICAL SPECIFICATIONS

Construction drawings for Phase MI-5 of the Project are included in Appendix K under a separate cover. The drawings are at a 90% level of completion and will be modified pending sponsor, BNSF, IEPR, and USACE comments, permitting, and agency reviews.

Draft technical specifications have been developed and are in Appendix L. These specifications were prepared in general accordance with Construction Specifications Institute (CSI) 2004 MasterFormat guidelines using a six-digit numbering system to organize the specifications sections. Front-end specifications (Division 00), which typically include procurement and contracting requirements, will be developed by the SRJB and will be included as part of the final bid documents. Front-end documents will generally be 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 the primary sources used for technical information, guidelines, and reference specifications:

- City of Minot Standard Specifications and Details (2013)
- North Dakota Department of Transportation Standard Specifications for Road and Bridge Construction (2014) NDDOT_2014 Standard Specifications
- 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 OPERATION AND MAINTENANCE MANUAL

An addendum to the original Operations and Maintenance (O&M) Manual will be completed as part of Phase MI-5 of the Project prior to operation of the project. 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 the City of Minot. Although final development of the O&M Manual is still pending, the content of the manual is anticipated to include the following sections:

- Part 1 – General Information
  - 1.1 Authority
  - 1.2 Additional Manuals
  - 1.3 Datum
  - 1.4 Location
  - 1.5 Historic Flooding
  - 1.6 Datum and United States Geological Survey (USGS) Gage
  - 1.7 Safe and Reliable Water Supply Purpose
  - 1.8 Maximizing Project Performance
  - 1.9 Project Features
  - 1.10 Construction History
  - 1.11 USACE Section 408 Approval
  - 1.12 General Regulations and Inspection and Reporting Procedures
  - 1.13 Notifications
  - 1.14 World Wide Web References
  - 1.15 Improvements and Project Modifications
  - 1.16 Encroachment or Trespass on Right of Way
  - 1.17 Reports
  - 1.18 Government Inspections
- Part 2 – Normal Operations, Maintenance and Inspections
  - 2.1 General
  - 2.2 Repair, Replacement, and Rehabilitation
  - 2.3 Construction Warranty
  - 2.4 Floodplain Management
  - 2.5 Emergency Action Plan
  - 2.6 Quarterly Inspection
  - 2.7 Five-Year Periodic Inspection
  - 2.8 Normal Operation, Maintenance, and Inspection Procedures
  - 2.9 Surveillance
  - 2.10 Semiannual Inspection Report Worksheets
- Part 3 – Emergency Operations and Post-Flood Recovery
  - 3.1 Notification of Distress
  - 3.2 General
  - 3.3 Advance Preparations
  - 3.4 Flood Alert and Flood Emergency
  - 3.5 Equipment and Supplies
  - 3.6 Flood Alert Conditions

   - 3.7 Flooding Conditions
   - 3.8 Flood Emergency Conditions
   - 3.9 Post-Flood Recovery
   - 3.10 Post-Flood Report
- Part 4 – References

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 reviewed and approved by the USACE, FEMA and Project sponsor upon completion of Phase MI-5.

16 PROJECT DESIGN GUIDELINES

The Project Design Guidelines represent procedures, guidelines, and formats to be used in the design of the Mouse River Enhanced Flood Protection Project (MREFPP) (Project). 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 MI-5. 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.

Version 2.0 of the Project Design Guidelines is included in Appendix N.

17 QA/QC

Quality is a priority of critical importance, and therefore a Quality Management Plan (QMP) was developed and utilized throughout the design and permitting of Phase MI-5 of the MREFPP. The QMP defines the parameters and provides the framework for achieving the goal of meeting the client's needs by efficiently providing deliverables that:

- Meet all project requirements defined in the Scope of Services, including those related to cost and schedule.
- Are technically accurate and free from significant errors.
- Effectively communicate their intended meaning.
- Are professional in appearance and tone.

Quality is controlled (QC) by thoroughly checking and reviewing the work products. Quality is assured (QA) by adequately defining the quality parameters to be followed on the project and ensuring that they are implemented. The Quality Management Plan and copies of the QA/QC review forms are given in Appendix Q.

18 REFERENCES

[1] A. E. Kehew, *Geology and Geotechnical Conditions of the Minot Area*, North Dakota: North Dakota Geological Survey, 1983.

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[4] Ackerman-Estvold, Storm Water Management Plan – City of Minot: Volume I – III, 2015.

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[10] American Concrete Institute, ACI 350 Code Requirements for Environmental Engineering Concrete Structures and Commentary, 2006.

[11] American Concrete Institute, ACI 530/530.1-13 Building Code Requirements and Specifications for Masonry Structures, 2013.

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[16] American Railway Engineering and Maintenance-of-Way Association (AREMA) – Manual for Railway Engineering Chapter 8, *Concrete Structures and Foundations*, 2018.

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