RED RIVER DIVERSION

FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4

APPENDIX F – HYDRAULIC STRUCTURES

Report for the US Army Corps of Engineers, and the cities of Fargo, ND & Moorhead, MN

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TABLE OF CONTENTS

F1.0 INTRODUCTION 11
F2.0 HYDRAULICS AND ENVIRONMENTAL CONSIDERATIONS 12
F2.1 Federally Comparable Plan (FCP) 13
F2.1.1 Control Structure on Red River of the North 13
F2.1.2 Fish Passage on Red River of the North 17
F2.1.3 Inlet Weir on Red River of the North 19
F2.1.4 Outlet to Red River of the North 20
F2.2 Locally Preferred Plan (LPP) 20
F2.2.1 Control Structure on Red River of the North 21
F2.2.2 Fish Passage on Red River of the North 27
F2.2.3 Diversion Inlet Structure 29
F2.2.4 Control Structure on Wild Rice River 30
F2.2.5 Fish Passage on Wild Rice River 32
F2.2.6 Diversion Channel Transition and Aqueduct at the Sheyenne River 33
F2.2.7 Spillway at Sheyenne River 36
F2.2.8 Diversion Channel Transition and Aqueduct at Maple River 37
F2.2.9 Spillway at Maple River 39
F2.2.10 Drop Structure at Lower Rush River 40
F2.2.11 Fish Passage on Lower Rush River 41
F2.2.12 Drop Structure at Rush River 42
F2.2.13 Fish Passage on Rush River 43
F2.2.14 Outlet Structure to Red River of the North 43
F2.2.15 Storage Area 1 44
F2.2.16 Wolverton Creek 45

F2.2.17 Local Drains....45
F2.2.18 Standard Project Flood (SPF) Analysis....46
F3.0 GEOTECHNICAL ENGINEERING.... 47
F3.1 Review of Geotechnical Data.... 47
F3.2 Seepage Analysis.... 48
F3.3 Slope Stability.... 48
F3.3.1 Methodology....49
F3.3.2 Storage Area 1 Levees....50
F3.3.3 Approach Channels....50
F3.3.4 Hydraulic Structures at Sheyenne and Maple Rivers....51
F3.3.5 Road and Rail Bridges....52
F3.4 Pile Capacity.... 52
F3.4.1 Methodology....52
F3.4.2 Design Capacity Summary....54
F3.4.3 Recommendations for Final Design....54
F4.0 STRUCTURAL DESIGN.... 55
F4.1 Structural Design Criteria.... 55
F4.1.1 Loads....55
F4.1.2 Load Combinations....56
F4.1.3 Reinforced Concrete Design....56
F4.1.4 Pile Design....56
F4.1.5 Sheet Pile....58
F4.2 Control Structures.... 58
F4.2.1 Load Combinations....59
F4.2.2 Assumptions....60
F4.3 Aqueduct Structures.... 61
F4.3.1 Load Combinations....62
F4.3.2 Assumptions....63
F4.4 Sheetpile Weirs/ Stepped Spillways/Ogee Spillways.... 65
F4.4.1 Sheetpile Weirs....65
F4.4.2 Stepped Spillways....65
F4.4.3 Ogee Spillways....67
F5.0 CIVIL DESIGN AT HYDRAULIC STRUCTURES.... 68
F5.1 Existing Conditions at Hydraulic Structures.... 68
F5.2 Civil Design at Gated Control Structures (Red River Control Structure, Wild Rice River Control Structure).... 69
F5.2.1 Hydraulic Structure Micro-Siting (Red River Control Structure, Wild Rice River Control Structure)....69
F5.2.2 Hydraulic Structure Civil Site Design (Red River Control Structure, Wild Rice River Control Structure)....71
F5.3 Civil Design at Aqueduct Structures (Sheyenne River Structure, Maple River Structure).... 74

F5.3.1 Hydraulic Structure Micro-Siting (Sheyenne River Structure, Maple River Structure)....75
F5.3.2 Hydraulic Structure Civil Site Design (Sheyenne River Structure, Maple River Structure)....77
F5.4 Civil Design at Tributary Drop Structures (Lower Rush River Structure, Rush River Structure) 79
F5.4.1 Hydraulic Structure Micro-Siting (Lower Rush River Structure, Rush River Structure)....79
F5.4.2 Hydraulic Structure Civil Site Design (Lower Rush River Structure, Rush River Structure)....80
F5.5 Civil Design at Diversion Drop Structures (Connecting Channel Weir, Diversion Inlet Structure, Outlet Structure to the Red River of the North).... 83
F5.5.1 Hydraulic Structure Micro-Siting (Connecting Channel Weir) 83
F5.5.2 Hydraulic Structure Micro-Siting (Diversion Inlet Structure, Outlet Structure to the Red River of the North)....83
F5.5.3 Hydraulic Structure Civil Site Design (Diversion Inlet Structure, Outlet Structure to the Red River of the North)...85
F5.6 Civil Design at Closure/Drainage Structures (Storage Area 1 Closure/Drainage Structure North, Storage Area 1 Closure/Drainage Structure East, Wolverton Creek Closure/Drainage Structure, Drain 14 Drop Structure) 90
F5.6.1 Hydraulic Structure Micro-Siting 90
F5.6.2 Hydraulic Structure Civil Site Design....91
F5.7 Civil Design at Storage Area 1 93
F5.7.1 Storage Area 1 Micro-Siting....93
F5.7.2 Storage Area 1 Civil Site Design 94

FIGURES

Figure F01 Project Overview Map
Figure F02 Longitudinal Profile of Diversion Channel
Figure F03 Storage Elevation Curves for Upstream Staging Area and Storage Area 1
Figure F04 Typical Cross Sections of Diversion Channel
Figure F05 Flows and Water Surface Elevations at Main Project Features for 0.2-percent Chance Event in Red River of the North (and Coincidental Event in ND Tributaries)
Figure F06 Flows and Water Surface Elevations at Main Project Features for 1-percent Chance Event in Red River of the North (and Coincidental Event in ND Tributaries)
Figure F07 Flows and Water Surface Elevations at Main Project Features for 2-percent Chance Event in Red River of the North (and Coincidental Event in ND Tributaries)

Figure F08 Flows and Water Surface Elevations at Main Project Features for 10-percent Chance Event in Red River of the North (and Coincidental Event in ND Tributaries)

Figure F09 Flows and Water Surface Elevations at Main Project Features for 0.2-percent Chance Event in ND Tributaries (and Coincidental Event in Red River of the North)

Figure F10 Flows and Water Surface Elevations at Main Project Features for 1-percent Chance Event in ND Tributaries (and Coincidental Event in Red River of the North))

Figure F11 Flows and Water Surface Elevations at Main Project Features for 2-percent Chance Event in ND Tributaries (and Coincidental Event in Red River of the North)

Figure F12 Flows and Water Surface Elevations at Main Project Features for 10-percent Chance Event in ND Tributaries (and Coincidental Event in Red River of the North)

Figure F13 Flows and Water Surface Elevations at Main Project Features for the 1997 Flood

Figure F14 Flows and Water Surface Elevations at Main Project Features for the 2006 Flood

Figure F15 Flows and Water Surface Elevations at Main Project Features for the 2009 Flood

Figure F16 Flows and Water Surface Elevations at Main Project Features for the 2010 Flood

Figure F17 FCP at the Red River of the North – Flows for Peaks in the Red River of the North

Figure F18 LPP at the Red River of the North and Wild Rice River – Flows for Peaks in the Red River of the North

Figure F19 Aerial of Red River Control Structure – LPP Flow Scenario 1: Average Flow

Figure F20 Aerial of Red River Control Structure – LPP Flow Scenario 2: 2-year Event

Figure F21 Aerial of Red River Control Structure – LPP Flow Scenario 3: 10-year Event

Figure F22 Aerial of Red River Control Structure – LPP Flow Scenario 4: 100-year Event

Figure F23 Perspective of Red River Control Structure – LPP Flow Scenario 1: Average Flow

Figure F24 Perspective of Red River Control Structure – LPP Flow Scenario 2: 2-year Event

Figure F25 Perspective of Red River Control Structure – LPP Flow Scenario 3: 10-year Event

Figure F26	Perspective of Red River Control Structure – LPP Flow Scenario 4: 100-year Event
Figure F27	Rendering of Red River Control Structure – LPP Flow Scenario 1: Average Flow
Figure F28	Rendering of Red River Control Structure – LPP Flow Scenario 2: 2-year Event
Figure F29	Rendering of Red River Control Structure – LPP Flow Scenario 3: 10-year Event
Figure F30	Rendering of Red River Control Structure – LPP Flow Scenario 4: 100-year Event
Figure F31	Rendering of Red River Control Structure and Fish Passage System– LPP Flow Scenario 3: 10-year Event
Figure F32	Rendering of Fully Open Gates on the Red River Control Structure looking from upstream– LPP Flow Scenario 1: Average Flow
Figure F33	Rendering of Fully Open Gates on the Red River Control Structure looking from upstream– LPP Flow Scenario 2: 2-year Event
Figure F34	Rendering of Partially Closed Gates on the Red River Control Structure looking from upstream– LPP Flow Scenario 3: 10-year Event
Figure F35	Rendering of Fully Open Gates on the Red River Control Structure looking from above – LPP Flow Scenario 1: Average Flow
Figure F36	Rendering of Fully Open Gates on the Red River Control Structure looking from above – LPP Flow Scenario 2: 2-year Event
Figure F37	Rendering of Partially Closed Gates on the Red River Control Structure looking from above – LPP Flow Scenario 3: 10-year Event
Figure F38	Rendering of Partially Closed Gates on the Red River Control Structure looking from above – LPP Flow Scenario 4: 100-year Event
Figure F39	Mosaic of Renderings of the Red River Control Structure - LPP
Figure F40	LPP at Sheyenne River – Spillway & Crossing Design Flows for Coincidental Events to Peaks in the Red River of the North
Figure F41	LPP at Maple River – Spillway & Crossing Design Flows for Coincidental Events to Peaks in the Red River of the North
Figure F42	Aerial of Maple River Hydraulic Structures – LPP Flow Scenario 1: low flow in diversion channel, no Maple flow diverted over spillway
Figure F43	Aerial of Maple River Hydraulic Structures – LPP Flow Scenario 2: open channel flow in diversion channel, Maple flow over spillway
Figure F44	Aerial of Maple River Hydraulic Structures – LPP Flow Scenario 3: pressurized flow in diversion channel, Maple flow over spillway
Figure F45	Perspective of Maple River Hydraulic Structures – LPP Flow Scenario 1: low flow in diversion channel, no Maple flow diverted over spillway
Figure F46	Perspective of Maple River Hydraulic Structures – LPP Flow Scenario 2: open channel flow diversion channel, Maple flow over spillway

Figure F47 Perspective of Maple River Hydraulic Structures – LPP Flow Scenario 3: pressurized flow in diversion channel, Maple flow over spillway
Figure F48 Rendering of Maple River Hydraulic Structures looking from above – LPP Flow Scenario 1: low flow in diversion channel, no Maple flow diverted over spillway
Figure F49 Rendering of Maple River Hydraulic Structures looking from above – LPP Flow Scenario 2: open channel flow in diversion channel, Maple flow over spillway
Figure F50 Rendering of Maple River Hydraulic Structures looking from above – LPP Flow Scenario 3: pressurized flow in diversion channel, Maple flow over spillway
Figure F51 Rendering of Maple River Hydraulic Structures looking from upstream in the diversion channel – LPP Flow Scenario 1: low flow in diversion channel, no Maple flow diverted over spillway
Figure F52 Rendering of Maple River Hydraulic Structures looking from upstream in the diversion channel – LPP Flow Scenario 2: open channel flow in diversion channel, Maple flow over spillway
Figure F53 Rendering of Maple River Hydraulic Structures looking from upstream in the diversion channel –LPP Flow Scenario 3: pressurized flow in diversion channel, Maple flow over spillway
Figure F54 Rendering of Maple River Hydraulic Structures looking from above (detail) – LPP Flow Scenario 1: low flow in diversion channel, no Maple flow diverted over spillway
Figure F55 Rendering of Maple River Hydraulic Structures looking from above (detail) – LPP Flow Scenario 2: open channel flow in diversion channel, Maple flow over spillway
Figure F56 Rendering of Maple River Hydraulic Structures looking from above (detail) – LPP Flow Scenario 3: pressurized flow in diversion channel, Maple flow over spillway
Figure F57 Mosaic of Renderings of Maple River Hydraulic Structures-LPP

TABLES

Table F1	Major Hydraulic Structures per Alignment
Table F2	Preliminary design of Control Structures on the Red River of the North and Wild Rice River (for 500-year event flow on Red River of the North)
Table F3	Preliminary design of Inlet Structures of Diversion Channel at the Red River of the North (RRN) and the Wild Rice River (WRR)
Table F4	Fish Passage Operational Range Water Surface Elevations, Corresponding Interpolated Discharges, and Flood Frequency Data for 1901-2009 at the USGS Gage in Fargo, North Dakota
Table F5	Preliminary design of Aqueduct on Sheyenne River and related Transition to Concrete Lined Section in LPP Diversion Channel
Table F6	Preliminary design of Aqueduct on Maple River and related Transition to Concrete Lined Section in LPP Diversion Channel
Table F7	Preliminary design of 300 ft Weir Spillway to divert waters from Sheyenne River to LPP Diversion Channel (crest elevation 912.56 ft)
Table F8	Preliminary design of 300 ft Weir Spillway to divert waters from Maple River to LPP Diversion Channel (crest elevation 893.63 ft)
Table F9	Preliminary design of Stepped Drop to fully divert waters from Lower Rush River to LPP Diversion Channel
Table F10	Preliminary design of Stepped Drop to fully divert waters from Rush River to ND LPP Diversion Channel
Table F11	Phase 3: Gated Structures Load Cases
Table F12	Phase 4: Gated Structures Load Cases
Table F13	Comparison of Pile Reactions: Rigid Cap vs. GROUP
Table F14	Phase 4: Aqueduct Structures Load Cases

DRAWINGS

G-001	Cover Sheet Vicinity Map and Location Map
G-002	Drawing Index and General Information
G-003	Overall Project Layout
S-401	FCP Control Structure on Red River of the North Plan View
S-402	FCP Control Structure on Red River of the North Gates Plan, Section and Elevation
S-403	FCP Control Structure on Red River of the North Wing Wall Plan, Elevation and Section
S-404	FCP Fish Passage Structure on Red River of the North Plan View and Sections
S-405	FCP Weir Structure on Red River of the North Plan View and Sections
S-406	FCP Outlet Structure on Red River of the North Plan View
S-407	LPP Control Structure on Red River of the North Plan View
S-408	LPP Control Structure on Red River of the North Gates Plan, Section and Elevation
S-409	LPP Control Structure on Red River of the North Wing Wall Plan, Elevation and Section
S-410	LPP Fish Passage Structure on Red River of the North Fish Plan View and Sections
S-411	Wolverton Creek Closure/Drainage Structure Plan View
S-412	Wolverton Creek Closure/Drainage Structure Plan and Sections
S-413	Diversion Weirs and Control Structure on Wild Rice River Plan View
S-414	Storage Area 1 Inlet and Control Structure on the Wild Rice River Plan and Elevation
S-415	Control Structure on the Wild Rice River Sections
S-416	Fish Passage System on Wild Rice River Plan and Sections
S-417	Weir Structure from Connecting Channel to Wild Rice River Plan View and Elevation
S-418	Storage Area 1 Plan View
S-419	Storage Area 1 Plan View and Elevations
S-420	Storage Area 1 Typical Closure/Drainage Structure Plan and Sections
S-421	Inlet Structure to Diversion Plan View
S-422	Inlet Structure to Diversion Plan View and Elevations
S-423	Inlet Structure to Diversion Sections
S-424	Sheyenne River Aqueduct Plan View
S-425	Sheyenne River Aqueduct Foundation Plan, Plan View
S-426	Sheyenne River Aqueduct General Arrangement Plan, Plan View

S-427 Sheyenne River Aqueduct Sections
S-428 Sheyenne River Aqueduct Sections (Continued)
S-429 Sheyenne River Aqueduct Spillway Weirs
S-430 Drain 14 Drop Structure Plan View and Sections
S-431 Maple River Aqueduct Plan View
S-432 Maple River Aqueduct Foundation Plan, Plan View
S-433 Maple River Aqueduct General Arrangement Plan, Plan View
S-434 Maple River Aqueduct Sections
S-435 Maple River Aqueduct Sections (Continued)
S-436 Maple River Aqueduct Spillway Weirs
S-437 Lower Rush River Drop Structure Plan View
S-438 Lower Rush River Drop Structure Plan and Elevations
S-439 Lower Rush River Drop Structure Sections
S-440 Lower Rush River Fish Passage System Plan View and Profile
S-441 Rush River Drop Structure Plan View
S-442 Rush River Drop Structure Plan and Elevations
S-443 Rush River Drop Structure Sections
S-444 Rush River Fish Passage System Plan View and Profile
S-445 LPP Outlet Structure on the Red River of the North Plan View
S-446 LPP Outlet Structure on the Red River of the North Plan, Elevation and Sections
S-447 LPP Outlet Structure on the Red River of the North Sections
S-448 Low Flow Channel Downstream of Lower Rush River Plan View and Section
S-449 Side Channel Control Structure (Typical) Plan View and Sections
S-450 Side Inlet 72” RCP (Typical) Plan View and Sections
S-451 Side Inlet 2-72” RCP (Typical) Plan View and Sections

EXHIBITS

Exhibit A Background Hydrologic Information
Exhibit B Alternatives Matrix for Control Structure on Red River of the North and Inlet Weir of Diversion Channel
Exhibit C Alternatives Matrix for Diversion Structures at North Dakota Tributaries
Exhibit D Hydraulic Design Computations – Diversion Structures
Exhibit E Hydraulic Design Computations – Storage Area 1
Exhibit F Hydraulic Design Computations – Drains (Floodplain impacts to the West)
Exhibit G Hydraulic Design Computations – Fish Passage

Exhibit H	2D Hydraulic Modeling of Diversion Structures and Fishways
Exhibit I	Evaluation of Impacts on Sediment Transport and Geomorphology
Exhibit J	Ice Aspects of Preliminary Designs
Exhibit K	Erosion Control Methods for Low Flow Channel
Exhibit L	Review of Geotechnical Data
Exhibit M	Seepage Analysis
Exhibit N	Slope Stability
Exhibit O	Pile Capacity
Exhibit P	Structural Design Criteria
Exhibit Q	Structural Design Computations – Control Structures
Exhibit R	Structural Design Computations – Aqueducts
Exhibit S	Structural Design Computations – Sheetpile/Rockfill Spillways
Exhibit T	Structural Design Computations – Stepped Spillways

APPENDIX F
HYDRAULIC STRUCTURES

F1.0 INTRODUCTION

This Appendix F of the February 28, 2011 submittal (Phase 4 report) represents an updated and revised version of the Appendix F included in the August 6, 2010 submittal (Phase 3 report). The updates and revisions have been driven by design modifications primarily associated with changes to the hydrology and hydraulic modeling. The updates and revisions presented include staging of water upstream of the Diversion Channel, construction of an engineered storage area, more detailed hydraulic modeling including use of a HEC-RAS unsteady flow model for project feasibility design, revised hydrology (in particular, for the ND tributaries: Wild Rice River, Sheyenne River, Maple River, Lower Rush River and Rush River), additional fish passage and ice control considerations, new geotechnical information and analysis, different structural design criteria, and enhanced grading development of the hydraulic structures. Furthermore, the updates and revisions incorporate the majority of the comments received from the U.S. Army Corps of Engineers (USACE) Project Delivery Team (PDT), USACE Agency Technical Review (ATR), USACE Independent External Peer Review (IEPR), the City of Fargo (North Dakota), the City of Moorhead (Minnesota), and the Natural Resources Agencies.

This Appendix F of the Phase 4 report presents the feasibility design of the major hydraulic structures required for two alternatives:

- Minnesota Short Alignment 35,000 cfs Alternative (MN Short 35K), Federally Comparable Plan (FCP); and
- North Dakota East Alignment, Locally Preferred Plan (LPP).

The FCP alternative corresponds to a target diversion flow (35,000 cfs when the 500-yr flood event occurs in the Red River of the North) and no staging or storage upstream of the diversion works, which results in some impacts on flood levels downstream of the diversion (henceforth referred to as downstream impacts). Hydraulic structures along the FCP were not redesigned during Phase 4. Feasibility designs of the FCP presented in this Appendix F refer to Phase 3 hydrology using a HEC-RAS steady flow model and are included for completeness. The LPP alternative corresponds to target stages (water surface elevations) at the U.S. Geological Survey (USGS) gage in Fargo for specific design flood events in the Red River of the North, but including staging and storage immediately upstream of the diversion works to eliminate downstream impacts. The major hydraulic structures included in the two alternatives are presented in Table F1, and their approximate locations are presented in Figure F01.

The design presented here has been carried out to a feasibility level using general hydrologic, hydraulic, environmental, geotechnical, structural and civil design considerations. Given the constraints imposed by the amount and quality of the information available and the timeframe to complete the feasibility study of the project, these designs are deemed sufficient to develop Class 3 cost estimates (see Appendix G) for congressional budgetary appropriation per USACE Engineer Regulation ER 1110-2-1302. However, it is acknowledged that additional investigations on fish passage, ice engineering and sediment transport; future updates to the hydrology; refinements of the HEC-RAS unsteady flow model; physical modeling of some of the hydraulic structures; detailed structural design; and additional site specific information (e.g., topography, soil borings, soil mechanics laboratory tests, field-scale pile driving tests) that become available for further evaluation of the alternative selected in the next stage of study and design may result in changes to the proposed configuration and functioning of the hydraulic structures.

F2.0 HYDRAULICS AND ENVIRONMENTAL CONSIDERATIONS

It should be noted that three sets of hydrology were analyzed in Appendix F of the Phase 3 report. Following input from the USACE-PDT, only the Year 0 hydrologic scenario was analyzed in this Phase 4 report for purposes of hydraulic modeling of the LPP Diversion Channel (see Appendix C) using the HEC-RAS unsteady flow model developed for the study area (see Appendix B). The background and details about the hydrology are provided in Appendix A.

The referred hydrologic and hydraulic modeling included the evaluation of peak flows in the Red River of the North and coincidental events in the ND tributaries (see Figures F05-F08), as well as, the evaluation of peak flows in the ND tributaries and coincidental events in the Red River of the North (see Figures F09-F12). The feasibility design of the LPP hydraulic structures was completed using the HEC-RAS unsteady flow models developed for local peak flows and water surface elevations rather than coincidental events. The feasibility design of the LPP hydraulic structures was also guided by the evaluation of downstream impacts for the four design floods associated with peak flows in the Red River of the North (10-percent chance or 10-yr, 2-percent chance or 50-yr, 1-percent chance or 100-yr, and 0.2-percent chance or 500-yr synthetic events). As a reference for the general public (not for feasibility design), the evaluation of downstream impacts also included the analysis of the four more recent, larger historic floods during the spring of 1997, 2006, 2009 and 2010 (see Figures F13-F16). However, the evaluation of downstream impacts was not extended to include the analysis of peak flows in the ND tributaries because hydrologic information was not available for coincidental events downstream of Perley, MN.

The differences in flows and water surface elevations in the LPP Diversion Channel as well as in the Red River of the North and the ND tributaries for the Year 0, 25 and 50 hydrology discussed in Appendix F of the Phase 3 report are not significant enough for the larger flood events. Therefore, additional refinements or modifications of the hydraulic designs based on the Year 25 and 50 hydrology are not deemed justified at this feasibility level. In this regard, discussions with the USACE-PDT determined that only Year 0 hydrology was to be used for feasibility design of the hydraulic structures and evaluation of downstream impacts during Phase 4. However, additional assessment of uncertainties in the hydrology (especially for some major tributaries downstream of the diversion) would certainly provide valuable guidance to develop more comprehensive operational rules of the entire diversion system (including the hydraulic structures) during the design phase, and potentially help to further define downstream impacts. Some specific recommendations for this and other tasks related to hydrologic and hydraulic modeling are provided in Appendix C.

F2.1 Federally Comparable Plan (FCP)

The following information regarding the FCP was previously presented as part of Appendix F in the Phase 3 report, and is included here for completeness. The FCP results and hydraulic structure designs discussed below refer to the HEC-RAS steady flow models developed as part of the Phase 3 feasibility analysis.

This Section F2.1 discusses the feasibility design of the Control Structure and Fishway on the Red River of the North and Inlet Structure of the Diversion Channel for the FCP. The flows to divert from the Red River of the North into the FCP Diversion Channel for the Year 0, 25 and 50 hydrology are presented in Figure F17, whereas the general layout and cross sections of these diversion structures are presented in Drawings S-401 through S-405.

This Section F2.1 also discusses the feasibility design of the Outlet to the Red River of the North from the FCP Diversion Channel. The general layout of this structure is presented in Drawing S-406.

This Section F2.1 does not discuss the feasibility design of the Inlet Structure of the Extension Channel. The discussion about this structure is presented in Appendix B of the Phase 3 report.

F2.1.1 Control Structure on Red River of the North

The general design considerations discussed below also apply to the Red River Control Structure for the LPP, which is presented in Section F2.2.1. On the other hand, the 3D renderings of the Red River Control Structure for the LPP that are presented in Figures F19 through F39 may be helpful to better visualize the feasibility design proposed for the Red River Control Structure for the FCP as well.

For the FCP, a Control Structure located on the Red River of the North immediately downstream of the Inlet Structure of the Diversion Channel is necessary to limit the amount of water flowing into the Protected Area (i.e., the Cities of Fargo, ND and Moorhead, MN). Another design goal for this structure is to avoid increasing water surface elevations upstream in the Red River of the North for the 100-yr and 500-yr flood events, while minimizing differences in water surface elevations between existing conditions and with-project for smaller flood events. Because of the former consideration, using gates that are raised from the bottom of the Red River of the North, as in the Manitoba Floodway, is not a feasible option. Maintaining the rating curve upstream of the diversion would help to keep the observed (natural) runoff storage in the floodplain upstream of the study area, and consequently, to keep the associated peak flow attenuation effect. In addition, maintaining the rating curve upstream of the diversion should help to reduce the potential for adverse morphologic impacts in the Red River of the North (e.g., development of head cutting along the main channel as a result of increased flow velocities due to stage reduction without discharge reduction).

It is worthwhile mentioning here that the objectives of the Breckenridge diversion project and the Fargo Moorhead diversion project are different. The objective of the Breckenridge diversion project was to lower stages on the Red River of the North and the Bois de Sioux rivers by about one ft, such that the combined diversion and levee project would provide an adequate degree of protection with sufficient benefits to justify the project. Because of the flat topography, there was an optimum ground elevation for the top of the Breckenridge levee to tie into. If the levee were to be raised above that elevation, the Breckenridge levee length would have become excessive either approaching or becoming essentially a ring levee around the City of Breckenridge. As implied in Appendix C , one main objective of the Fargo Moorhead diversion project is to lower stages sufficiently on the Red River of the North to significantly reduce flood damages in the Protected Area and thus to provide benefits that would justify the relatively elevated project cost. Therefore, a great level of active control and management (through gates operation) of the flows that pass into the cities of Fargo and Moorhead is warranted.

The configuration of the Red River Control Structure depends on the configuration of the Inlet Structure of the Diversion Channel. Different combinations of proposed configurations for these two hydraulic structures are presented in Exhibit B, including a qualitative assessment of the advantages and disadvantages of each concept in terms of hydraulic performance, handling of flood flows and low flows, potential environmental impacts, permitting, and operation and maintenance. For this feasibility design, the concept selected for the Red River Control Structure is the one that would better accomplish the project goals outlined above. This concept consists of a concrete gravity dam with three 50 ft-wide bays each including a lower ungated area and an upper gated area per bay (using primary tainter gates, and secondary bulkheads in case the tainter gates malfunction), and wingwalls. In addition, a gated secondary by-pass channel for fish passage (i.e., the fishway) will be located on one of the sides of the primary Control Structure. Following discussions with the USACE-PDT, the Control Structure is recommended to be built off the existing Red River of the North channel.

Exhibit A presents a summary of background hydrologic information for the Red River of the North. This information combined with the main project goal of reducing the flows to pass into the Protected Area, not only for the 500-yr flood event but also for flows in the Red River of the North greater than 9,600 cfs (at the USGS gage in Fargo; 9,600 cfs approximately corresponds to the 3.6-yr flow), guided the hydraulic design that is summarized in Table F2 and is presented in greater detail in Exhibit D. The width and height of the main bays were determined based on the primary goals of having redundancy and flexibility in controlling the flows to pass into the Protected Area during large flood events when the Control Structure gates will be partially or fully closed, and also of providing sufficient freeboard (5 ft, following input from the USACE-PDT during the July 1, 2010 meeting) during the 500-yr flood event in the Red River of the North. Furthermore, the width of the main bays has been set such to minimize the impact on the flow velocity across the Control Structure with respect to existing conditions as well as to safely pass river ice when the Control Structure gates will be fully open. In addition, the width of the bays has been somewhat determined by two additional design considerations. First, the overall width and associated flow area of the gated section of the Control Structure would not be significantly different than that of the natural Red River of the North channel up to bankfull flow conditions. Second, it would provide room for a minimum height of a lower ungated area.

For flows in the Red River of the North greater than 9,600 cfs (actually beginning with the 5-yr flow), the range of operation of the Control Structure gates has been defined using the orifice equation for submerged flow conditions, with a discharge coefficient of 0.80 (HEC-RAS Hydraulic Reference Manual, v4.0.0, 2008) and given headwater and tailwater conditions for the different return periods analyzed (see Exhibit D). Based on studies done for Lock and Dam 22 in the Mississippi River and the guidelines provided in the Hydraulic Design Criteria 320-8 (USACE, 1987), the discharge coefficient could be closer to the range 0.90-1.00, but it was agreed with the USACE-PDT (email communication dated December 9, 2009) to be conservative on the required gated area (i.e., use a lower discharge coefficient and consequently a larger area). It is worthwhile indicating that for the submerged flow conditions that prevail for all the cases analyzed when the gates are partially or fully closed, the discharge but not the flow velocity through the gates is determined by the opening height of the gates. The flow velocity is instead determined by the difference between the headwater and tailwater values, with the latter depending on the target flow to pass into the Protected Area for the corresponding return period analyzed. This design has been used in the hydraulic modeling presented in Appendix B of the Phase 3 report.

It is important to highlight here that although similar results (in terms of flows passing into the Protected Area) can be achieved by operating the tainter gates of only two of the 50-ft bays and having the third one completely closed (using the secondary bulkhead) during a large flood event, fully operational tainter gates in the three 50-ft bays have been considered in this feasibility design to account for additional redundancy and flexibility in the operation of the Control Structure. However, it is recommended to further evaluate in the design phase an alternative configuration that has only two gated bays, likely wider than 50 ft each, and for which operation of the tainter gate of only one bay while the other is completely closed (using the secondary bulkhead) could provide comparable diversion flows for the different return periods analyzed, which could translate into a lower construction cost of the Control Structure. Another reason for giving this recommendation is that the overall width of the gated area (with the three 50-ft bays) is relatively greater than the natural cross section of the Red River of the North up to bankfull flow conditions (see Exhibit D), and also a greater width of each bay could be favorable for better handling of river ice (see Exhibit J). In addition, the approach and exit channels to the Control Structure were assumed to have sideslopes of 7H:1V based on geotechnical stability considerations (see Exhibit N). However, the natural cross section of the Red River of the North has steeper slopes. Provided appropriate geotechnically engineered stabilization measures, working with steeper slopes would certainly offer a more efficient hydraulic connection of the approach and exit channels to the Control Structure, even more if the total width of the gated area is reduced, and all of this combined could translate into a lower construction cost of the Control Structure.

As indicated above, the proposed design of the Control Structure includes an ungated area below the main gates that is at approximately the same elevation as the river channel bottom (see Exhibit D). The height of this area, which is always open even when the gates are fully closed, is approximately 2 ft. This ungated area is intended to allow the Red River of the North to pass bedload transport through the Control Structure for flows greater than 9,600 cfs when the gates will be either partially or fully closed. Although the Red River of the North is primarily a silt- and clay-bed river, hence it does not typically transport large amounts of sediment as bedload (proportionally, the amount of sediment transported in suspension will be dominant), in absolute terms bedload transport is likely important in magnitude during the largest flood events. The proposed design with the lower ungated area will prevent sediment accumulation that otherwise would occur if the bays are closed down to the invert of the Control Structure. With the proposed design, the significantly high flow velocities through the gated area suggest that most sediment (both bedload and suspended) could be passed downstream of the Control Structure. However, it is strongly recommended that this issue is further evaluated in the design phase, when site specific sediment transport measurements conducted by the USGS during the last 2010 spring flood event are processed and become available. Moreover, this further evaluation could provide additional justification for studying the alternative configuration proposed above to have only two wider bays in the gated section of the Control Structure, which might lead to a greater height of the lower ungated area, therefore it could provide greater assurance that sediment will not accumulate upstream.

As a general design consideration, it will be preferable to maintain ice and debris flows in the rivers rather than in the Diversion Channel. Exhibit J presents a summary of the ice aspects to consider in these feasibility designs, as recommended by Andrew Tuthill from the Ice Engineering Group at the USACE Cold Regions Research and Engineering Laboratory (CRREL). Exhibit J also includes a memorandum summarizing a more recent conversation with Andrew Tuthill on July 8, 2010 regarding general ice control measures for this project. It was emphasized in this conversation that the design intent should be to preserve existing ice passage conditions on the Red River of the North main channel while preventing ice from entering and jamming in the FCP Diversion Channel. For this reason, the sheet ice cover on the Red River of the North may need to be stabilized and retained upstream of the Control Structure (and also upstream of the Inlet Structure of the Diversion Channel). Ice and debris booms have been considered in this feasibility design. The necessity and design of ice retention schemes will depend, however, on further analysis of expected ice conditions and ice processes in the vicinity of the diversion entrances and will need to be examined in more detail during the design phase. In terms of passing ice or debris through the Control Structure, pending additional site specific studies, a minimum width of 40 ft for each bay could ensure ice passage (note that the width of each bay is 50 ft) when the gates are fully open. The determination of the operational measures to handle river ice when the gates are partially or fully closed is strongly recommended to be worked out during the design phase.

The configuration proposed for the Control Structure resembles a bridge with large openings when no water from the Red River of the North is diverted into the Diversion Channel. As indicated above, the gates would be fully open for flows smaller than 9,600 cfs (at the USGS gage in Fargo; 9,600 cfs approximately corresponds to the 3.6-yr flow). The discussion in Appendix B of the Phase 3 report clearly indicates, however, that this design criterion does not necessarily translate into an immediate operation of the Control Structure gates when the flows are greater than 9,600 cfs. In any case, what is important to highlight here is that because modeled flow velocities through the Control Structure are within the range of the measured values when the gates are fully open, fish passage is secured. Furthermore, the rough bottom of the Control Structure (achieved by adding boulders or baffles) should help create the desired complexity in the flow pattern and areas of relatively low flow velocity. The results of the 2D hydraulic modeling (*for demonstration, not for design*) presented in Exhibit H suggest that areas of low flow velocity would exist at all bays when the gates are fully open. More specifically, for a flow of 9,600 cfs, velocities for post-construction conditions were similar to velocities for existing conditions. Peak velocities in the Red River of the North for both existing and post-construction conditions were roughly 2.0 fps. However, when the gates are lowered, the modeled flow velocities through the openings could be as high as approximately 25 fps. The Control Structure then becomes a barrier for fish passage. To solve this problem, a secondary by-pass channel consisting of riffles and pools (with gated openings at one side wingwall of the Control Structure) has been designed to provide fish passage for flows up to the 50-yr flood events. The configuration of the fish bypass channel is discussed in Section F2.1.2 below, and the design details are presented in Exhibit G.

F2.1.2 Fish Passage on Red River of the North

The general design considerations discussed below also apply to the fish bypass channel on the Red River of the North for the LPP, which is presented in Section F2.2.2.

Fish passage consisting of multiple parallel channels of alternating pools and riffles would allow fish to move upstream of the Control Structure for flows up to the 50-yr flood event on the Red River of the North. Additional details, tables, and figures regarding the design of fish passages at the FCP Control Structure are presented in Exhibit G.

Fish passages at the FCP Control Structure are designed with consideration to several criteria based on discussions with the USACE-Environmental, the Natural Resources Agencies, and the feasibility design in the USACE‟s Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H. The design criteria include:

- Inclusion of pool-riffle sequences, allowing fish to rest in pools between areas of high velocities in riffles
- Maximum flow velocities of approximately 6 fps in riffles and gated area of the fishway
- Average velocities of approximately 1.5 fps in pools
- Average slope of between 1 and 3 percent
- Minimum depth of 1 ft at the Control Structure entrance to the fish passage
- Design flow of 1% to 2% of the flow through the Control Structure entrance
- Minimum downstream invert 1 to 2 ft below the 5-yr tailwater elevation

The fish passage designed for the Red River Control Structure for the FCP includes three parallel channels, each including alternating pools and riffles. Multiple channels instead of a single switchback channel (as proposed in the Phase 2 design) are necessary because of the nearly 10 ft variability in headwater elevations between the 5-yr and 50-yr events in the Red River of the North (see Exhibits A and D). A single fish passage channel capable of operating during the smaller flood events cannot maintain velocities acceptable for fish passage during the larger flood events. Three fish passage channels with entrance inverts staggered vertically by 3.5 ft are necessary to cover the wide range in headwater elevation for which the fishway would be active while maintaining velocities less than 6 fps through the riffles and the Control Structure entrance. The critical condition (i.e., highest velocities and highest flows) occur when the headwater at the Control Structure entrance to a fish passage channel is the greatest (approximately 4.5 ft). Three adjacent gates (10 ft wide by 5 ft tall, with identical invert elevations) are proposed at the Control Structure entrance to each fish passage channel. The reason for having a gated entrance is to prevent water from entering a fish passage channel when the headwater elevation is too high (thus limiting the maximum flow through the fish passages). When the gates of a given fish passage channel are closed, the gates leading to the next fish passage channel with a higher entrance invert will be opened.

A more detailed discussion of pool and riffle design is included in Exhibit G. Each riffle is designed to achieve a 1 ft drop in elevation over a 20 ft reach. Pools at least 25 ft in length are located between each riffle and have a depth of at least 5 ft. The minimum pool dimensions are based on achieving a volumetric energy of between 3.1 to 4.2 ft pounds per second per cubic foot (see USACE‟s Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H). Each pool has a bottom width on the order of 40 ft. The first riffle in the pool-riffle sequence of each fish passage channel (i.e., downstream of the gated section on the wingwall of the Control Structure) has a narrower width intended to limit the flow through the fish passage structure at higher headwater elevations. Subsequent riffles have widths similar to the pools in order to simplify the design. The pool-riffle sequence of each fish passage channel result in an overall slope of about 2 percent. The three fish passage channels include 10, 6, and 3 pool-riffle sequences, respectively (the number of sequences is based on the drop between the upstream gate invert and the downstream 5-yr tailwater elevation in the Red River of the North). All these fish passage channels converge into a single channel at the confluence with the exit channel of the Control Structure, downstream of the domain of the submerged hydraulic jump that will develop downstream of the primary Control Structure when the gates of the 50-ft bays are partially or fully closed. The results of the 2D hydraulic modeling presented in Exhibit H show that flow velocities at this downstream entrance to the fishway are relatively small (less than 1 fps).

The flow through the fishway ranges from 40 cfs to about 600 cfs (or 0.4 to 6 percent of the total flow through the Control Structure). Reducing the range of flow through the fish passage channels would require more channels with smaller vertical spacing, but this could significantly increase the construction cost of the Control Structure. In addition, for cost estimating purposes (see Appendix G) the feasibility design assumes that an approach channel will be constructed upstream of the Fishway entrance to the Control Structure. This is not required for the proper hydraulic functioning of the Fishway, but it is anticipated that the Natural Resources Agencies would prefer to see a fish route that continues several hundred feet upstream of the Control Structure to reduce the risk of fish being swept downstream through the main bays of the Control Structure. The results of the 2D hydraulic modeling presented in Exhibit H indicates that this concern is not necessarily justified, as the depth-averaged flow velocities upstream of the Control Structure are not that high; the velocities near the bottom of the water column are the ones that are high, not the ones near the water surface where the fish would return to the approach channel of the Control Structure. It is recommended that these two issues are further evaluated during the design phase, when in addition, more precise information about the fish communities in the project area becomes available to the design team.

F2.1.3 Inlet Weir on Red River of the North

The configuration of the Inlet Structure on the Diversion Channel depends on the configuration of the Red River Control Structure. Different combinations of proposed configurations for these two hydraulic structures are presented in Exhibit B, including a qualitative assessment of the advantages and disadvantages of each concept in terms of hydraulic performance, handling of flood flows and low flows, potential environmental impacts, permitting, and operation and maintenance. For this feasibility design, the concept selected for the Inlet Structure on the Diversion Channel is the one that (when combined with the Red River Control Structure) would better accomplish the project goals outlined above. This concept consists of a passive (i.e., no gates or movable parts) compound weir with a crest elevation approximately 0.5 ft above the water surface elevation for the 3.6-yr event (9,600 cfs). The compound weir has been selected to maximize diversion efficiency for the different return periods analyzed while not modifying flood elevations upstream of the Control Structure.

This Inlet Structure will be constructed downstream of the Extension Channel (see discussion in Appendix B of the Phase 3 report). The hydraulic design of the Inlet Structure is summarized in Table F3 and is presented in greater detail in Exhibit D. Given the dimensions of this structure (in particular the height of the walls), it is recommended that a comparison of the proposed sheetpile-rockfill weir versus a more conventional Ogee type concrete spillway (likely more expensive) or a stepped vertical drop (similar to the ones proposed for the Lower Rush River and Rush River, also likely more expensive) is conducted. However, it is suggested to first obtain site specific geotechnical information before proceeding with this evaluation of alternative designs. It is worthwhile highlighting here that the proposed sheetpile-rockfill weir is adequate from a hydraulics viewpoint, and more importantly, it provides an environmentally sound design with regard to the potential need for fish passage from the FCP Diversion Channel upstream into the Red River of the North.

F2.1.4 Outlet to Red River of the North

Similar to the design for the Breckenridge Diversion Project, the Outlet of the FCP Diversion Channel into the Red River of the North consists of riprap over approximately the downstream 300 ft of the Diversion Channel.

Two-dimensional (2D) hydraulic models for the Outlet of the FCP Diversion Channel were created to assess the (vertically-averaged) velocity distribution as it relates to fish passage. Existing conditions were compared to post-construction conditions for two flow conditions, the 2-yr and the 20-yr events. For the 2-yr condition, velocities for post-construction conditions were similar to velocities for existing conditions. Peak velocities in the Red River of the North for both existing and post-construction conditions were roughly 2.0-2.5 fps. For the 20-yr flow condition, velocities for post-construction conditions were similar to velocities for existing conditions as well. Peak velocities in the Red River of the North for both existing and post-construction conditions downstream of the Outlet were roughly 2.5-3.0 fps. Peak velocities in the FCP Diversion Channel were roughly 2.0-2.5 fps. An explanation of the modeling methods and a more in-depth discussion of the results are presented in Exhibit H.

F2.2 Locally Preferred Plan (LPP)

The feasibility design of the LPP hydraulic structures was developed together with the feasibility design of the LPP Diversion Channel (see Figures F02 and F04; or for more details see Appendix C), such that the incorporation of staging and storage immediately upstream of the diversion works (see Figure F03) would not only allow to meet the stages at the USGS gage in Fargo that were met in Phase 3 (i.e., project benefits in Phase 4 would be the same as in Phase 3), but would also help to eliminate downstream impacts. Thus, the overall concept of the LPP evolved from diversion only in Phase 3 to diversion and staging/storage in Phase 4.

This Section F2.2 presents the hydraulic feasibility design of the primary LPP hydraulic structures, and it also includes some information about ice aspects, fish passage and sediment transport considerations accounted for in the feasibility design. More specifically, Section F2.2 includes discussions about:

- The Control Structure and Fishway on the Red River of the North. The flows to divert from the Red River of the North (and the Wild Rice River) into the LPP Diversion Channel (through the Connecting Channel) are presented in Figure F18, whereas the general layout and cross sections of these diversion structures are presented in Drawings S-407 through S-410. In addition, 3D renderings of the Red River Control Structure for the LPP (including the Fishway and tie-back levees) are presented in Figures F19 through F39 to better visualize the feasibility design proposed for conditions ranging from average flow to the 100-yr flood event.

- The Control Structure and Fishway on the Wild Rice River, and the primary Diversion Inlet Structure. As indicated above, the flows to divert from the Wild Rice River into the LPP Diversion Channel are presented in Figure F18, whereas the general layout and cross sections of these diversion structures are presented in Drawings S-413 through S-417 and S-421 through S-423.
- The Diversion Channel transition, aqueduct crossing and spillway diversion at the Sheyenne River and Maple River crossings. The flows to divert from the Sheyenne River and Maple River into the LPP Diversion Channel are presented in Figures F40 and F41, respectively, whereas the general layout and cross sections of these diversion structures are presented in Drawings S-424 through S-429 and S-431 through S-436, respectively. In addition, 3D renderings of the hydraulic structures at the Maple River crossing of the LPP Diversion Channel are presented in Figures F42 through F57 to better visualize the feasibility design proposed for conditions ranging from no flow diverted from the tributary to high flows in the LPP Diversion Channel running beneath the aqueduct crossing combined with flows diverted from the tributary and passing through into the Protected Area.
- The Drop Structure and Fishway at the Lower Rush River and Rush River crossings. The general layout and cross sections of these diversion structures are presented in Drawings S-437 through S-440 and S-441 through S-444, respectively.
- The Outlet to the Red River of the North from the LPP Diversion Channel. The general layout of this structure is presented in Drawings S-445 through S-447.
- Storage Area 1, the Control Structure at Wolverton Creek, and structures proposed for local drains. The general layouts of these structures are presented in Drawings S-418 through S-420, S-411 through S-412, and S-430, respectively.

F2.2.1 Control Structure on Red River of the North

For the LPP, a Control Structure located on the Red River of the North immediately downstream of the Connecting Channel (but upstream of the confluence with the Wild Rice River) is necessary to limit the amount of water flowing into the Protected Area (i.e., the Cities of Fargo, ND and Moorhead, MN). Another design goal for this structure is to increase water surface elevations upstream in the Red River of the North during flood events, in order to eliminate downstream impacts. As indicated in Appendix C, one main objective of the Fargo Moorhead diversion project is to lower stages sufficiently on the Red River of the North to significantly reduce flood damages in the Protected Area and thus to provide benefits that would justify the relatively elevated project cost. Therefore, a great level of active control and management (through gates operation) of the flows that pass into the cities is warranted.

The configuration of the Red River Control Structure (and also that of the Wild Rice River Control Structure) depends on the configuration of the primary Diversion Inlet

Structure (see Section F2.2.3). Different combinations of proposed configurations for these two hydraulic structures are presented in Exhibit B, including a qualitative assessment of the advantages and disadvantages of each concept in terms of hydraulic performance, handling of flood flows and low flows, potential environmental impacts, permitting, and operation and maintenance. For this feasibility design, the concept selected for the Red River Control Structure is the one that would better accomplish the project goals outlined above. This concept consists of a concrete gravity dam with three 50 ft-wide bays (other possible gates configurations are presented in Exhibit D), each including a lower ungated area and an upper gated area per bay (using primary tainter gates, and secondary bulkheads in case the tainter gates malfunction), and wingwalls. In addition, a gated secondary by-pass channel for fish passage (i.e., the fishway) will be located on one of the sides of the primary Control Structure. Following discussions with the USACE-PDT, the Control Structure is recommended to be built off the existing Red River of the North channel.

Exhibit A presents a summary of background hydrologic information for the Red River of the North. This information combined with the main project goal of reducing the flows to pass into the Protected Area, not only for the 500-yr flood event but also for flows in the Red River of the North greater than 6,100 cfs (equivalent to 9,600 cfs at the USGS gage station in Fargo, which is located downstream of the confluence with the Wild Rice River; 9,600 cfs approximately corresponds to the 3.6-yr flow), guided the hydraulic design that is summarized in Table F2 and is presented in greater detail in Exhibit D. The width of the main bays were determined based on the primary goals of having redundancy and flexibility in controlling the flows to pass into the Protected Area during large flood events when the Control Structure gates will be partially or fully closed. The height of the main bays was determined to provide sufficient freeboard during the 500-yr design flood, and to allow safe diversion of a more extreme event (see Section F2.2.18) without overtopping of the Red River Control Structure and the levees forming the main line of flood protection in the Red River of the North (and Wild Rice River). Furthermore, the width of the main bays has been set such to minimize the impact on the flow velocity across the Red River Control Structure with respect to existing conditions as well as to safely pass river ice when the Red River Control Structure gates will be fully open. In addition, the width of the bays has been somewhat determined by two additional design considerations. First, the overall width and associated flow area of the gated section of the Red River Control Structure would not be significantly different than that of the natural Red River of the North channel up to bankfull flow conditions. Second, it would provide room for a minimum height of a lower ungated area.

The gates at the Red River Control Structure are expected to be operated during flood events for which the forecasted peak flow of the hydrograph in the Red River of the North at the USGS gage in Fargo will exceed 9,600 cfs. The operational scheme of the gates proposed in this feasibility analysis is as follows. At the beginning of each flood event the gates would begin to close so that they are in the lowest position during most of the rising limb of the hydrograph, i.e., before the incoming peak flows in the Red River of the North and Wild Rice River. The gates would remain in the lowest position until the majority of the incoming flows during the rising limb of the hydrograph are conveyed through the Diversion Channel. Only after the peak flow has passed through the downstream end of the Diversion Channel, the gates on the Red River would begin to open. Allowing the peak flow in the Diversion Channel to reach the Red River of the North downstream of the diversion before the gates begin to open would result in a decoupling of the peak flow in the Diversion Channel from the one that would pass through the Red River of the North into the Protected Area, which helps to eliminate downstream impacts and offers the possibility of further reducing with-project stages at the USGS gage in Fargo (see discussion below and Exhibit D). A more complete evaluation of the merits of this operational scheme is presented in Appendix C.

So for the Phase 4 feasibility design the gates begin in a position that is less than fully open, but the gates are sufficiently open such that water is not staged upstream of the Red River Control Structure at the very beginning of the simulation with the HEC-RAS unsteady flow model. For all design events, the gates are then gradually closed over several days to reach a minimum opening during the rising limb of the hydrograph, before the peak flow. This was done to keep the HEC-RAS model computationally stable and to minimize the number of iterations required to converge on a solution for a given time step while lowering the gates. In reality, it may be possible to lower the gates much quicker and/or to begin with a bigger opening, without this necessarily having to translate in downstream impacts. This further optimization of the gates operational scheme is recommended for the next phase of study and design.

During the Phase 3 feasibility design, flow was not diverted into the Diversion Channel until 9,600 cfs occurred at the USGS gage in Fargo. Actually, Phase 3 work was based on HEC-RAS <u>steady</u> flow models that were used to evaluate diversion of the peak of the hydrograph for a given flood event; that is, the analysis was based on a single, constant discharge value for a given flood event. There was no need in Phase 3 to evaluate variable gate openings over the pass of the hydrograph, hence there was no assessment of gates operation for flows smaller than 9,600 cfs. The feasibility analysis in Phase 4 was based on the HEC-RAS <u>unsteady</u> flow model developed for feasibility design (see Appendix C), which deal with the entire flood hydrograph, not only the peak flow. The gates would not be operated when the forecasted peak flow of the hydrograph at the USGS gage in Fargo is less than 9,600 cfs. In other words, the frequency with which the gates could be operated is determined by the likelihood of peak flows larger than 9,600 cfs. This has happened 20 times during the 108 years of record, but 11 of those 20 have happened in the past 18 years. On the other hand, the Cities may determine not to operate the gates for events somewhat larger than 9,600 cfs, therefore reducing the frequency with which the gates would be operated. For instance, this could be the decision during summer floods (the historic peak flows are in the order of 12,000-13,000 cfs, which is equivalent to a stage of 30 or the beginning of major flooding), but not during spring floods (the historic 2009 flood of record had a peak flow near 30,000 cfs and a stage near 40, about 10 ft above the stage of major flooding).

As indicated above, trial runs using different operational schemes suggested that in order to eliminate downstream impacts for flood events with a peak flow larger than 9,600 cfs, the gates at the Red River Control Structure would need to be operated during the rising limb of the hydrograph, i.e., before 9,600 cfs occurs at the USGS gage in Fargo. The feasibility analysis in Phase 4 also found that in order to minimize downstream impacts, the gates have to restrict flow passing into the Protected Area earlier for more frequent events. This is primarily due to two reasons:

- Downstream impacts are caused by the timing along the entire rising limb of the hydrograph, not just the peak of the hydrograph. In order to control the timing and shape of the rising limb of the hydrograph, the gates must begin operating at lower flows/stages for smaller events.
- For larger events there is a larger floodplain or "pool" downstream of the project. This means that the downstream impacts are relatively less sensitive to shape and timing of the rising limb of the hydrograph. That is, for larger events a small increase in discharge along the rising limb of the hydrograph may not result in measurable increases to flood levels downstream of the diversion. However, that same increase in discharge for smaller events will likely result in measurable impacts downstream of the diversion.

When the gates would start to open after the peak flow of the hydrograph, they would be opened at a rate that increases water surface elevations at the USGS gage in Fargo up to the target elevation. During the receding limb of the hydrograph, the gates would only be operated for one hour during the day. This allows the model to account for the travel time between the Red River Control Structure and the USGS gage in Fargo. If the gates are continuously operated, then there is greater potential to allow too much water into the Protected Area and exceed the target elevation at the USGS gage in Fargo. For this Phase 4 feasibility analysis, it was agreed with the USACE-PDT (email communication dated February 12, 2011) that the models should match the Phase 3 with-project stages at the USGS gage in Fargo within 0.10-0.15 ft, such that the project benefits in Phase 4 would differ by less than 5% from those estimated in Phase 3. As a result of allowing the peak flow on the Diversion Channel to pass through the system first, during the recession limb of the hydrograph the gates may be opened either faster to reduce the duration that water is stored upstream of the Protected Area, or slower to further reduce the water surface elevation at the USGS gage in Fargo. This allows the Cities to potentially achieve additional protection, above the target elevation at the USGS gage in Fargo, by simply slowing the rate at which the gates are opened during the receding limb of the hydrograph. This is another recommendation for further evaluation in the next phase of study and design.

Following discussions with the USACE-PDT, it was determined to utilize user defined rules to calculate the flow through the gated control structures rather than the default HEC-RAS gate routines. This was primarily because the default routines in HEC-RAS for tainter gates assume the gates are elevated over a sill, which does not accurately characterize the proposed configuration of the control structures proposed in this feasibility design for the Red River of the North and Wild Rice River. In this regard, the review of discharge measurements through the tainter gates at the Mississippi River Lock and Dam 6-10 and of the very extensive database in *Discharge Algorithms for Canal Radial Gate: REC-ERC-83-9* (US Bureau of Reclamation 1983) allowed development of a simple relationship for the discharge coefficient in the orifice equation as a function of the degree of submergence (see Exhibit D). It is important to note here that for most of the duration of the design flood hydrographs, the degree of submergence at the control structures is relatively high because even with the larger flows being diverted and the upstream staging and storage, the flows being passed into the Protected Area are still significant, hence the tailwater is relatively high. In addition, user defined rules were utilized to control the gate opening throughout the duration of the design flood events. This provided the flexibility to control the gate opening based on the amount of water being let into the Protected Area throughout the duration of a flood event, with the goal of meeting the Phase 3 maximum stage at the USGS gage in Fargo.

It is important to highlight here that although similar results (in terms of flows passing into the Protected Area) can be achieved by operating the tainter gates of only one of the 50-ft bays and having the other two completely closed (using the secondary bulkhead) during a large flood event, fully operational tainter gates in the three 50-ft bays have been considered in this feasibility design to account for additional flexibility in the operation of the Red River Control Structure. However, it is recommended to further evaluate in the design phase an alternative configuration that has only two gated bays, likely wider than 50 ft each, and for which operation of the tainter gate of only one bay while the other one is completely closed (using the secondary bulkhead) could provide comparable results in terms of flood damage reduction for the different design events, and potentially reduce the cost of this structure. Another reason for giving this recommendation is that the overall width of the gated area (with the three 50-ft bays) is relatively greater than the natural cross section of the Red River of the North up to bankfull flow conditions (see Exhibit D), and also a greater width of each bay could be favorable for better handling of river ice (see Exhibit J).

As indicated above, the proposed design of the Red River Control Structure includes an ungated area below the main gates that is at approximately the same elevation as the river channel bottom (see Exhibit D). This ungated area is intended to allow the Red River of the North to pass bedload transport through the Control Structure for flows greater than 6,100 cfs when the gates will be either partially or fully closed. Although the Red River of the North is primarily a silt- and clay-riverine system, hence it does not typically transport large amounts of sediment as bedload (the amount of sediment transported in suspension will be a few orders of magnitude larger; see Exhibit I), it is still recommended that bedload transport is allowed to pass into the Protected Area during the larger flood events. The proposed design with the lower ungated area will prevent sediment accumulation that otherwise would occur if the bays are closed down to the invert of the Control Structure. With the proposed design, the significantly high flow velocities through the gated area suggest that most sediment (both bedload and suspended) could be passed downstream of the Control Structure. An initial assessment of potential impacts of the proposed diversion on the sediment transport and channel morphology of the Red River of the North and ND tributaries is provided in Exhibit I. However, the USACE-PDT has retained West Consultant to further complete a geomorphology study in preparation for the next stage of study and design.

As a general design consideration, it will be preferable to maintain ice and debris flows in the rivers rather than in the Diversion Channel. Exhibit J presents a summary (from the Phase 2 report) of the ice aspects to consider in these feasibility designs, as recommended by Andrew Tuthill from the Ice Engineering Group at the USACE Cold Regions Research and Engineering Laboratory (CRREL). Exhibit J also includes a memorandum summarizing a subsequent conversation with Andrew Tuthill on July 8, 2010 regarding general ice control measures for this project. It was emphasized in this conversation that the design intent should be to preserve existing ice passage conditions on the Red River of the North main channel while preventing ice from entering and jamming in the LPP Diversion Channel. For this reason, the sheet ice cover on the Red River of the North may need to be stabilized and retained upstream of the Control Structure (and also upstream of the Connecting Channel). Ice and debris booms were thus considered in Phase 3, but the upstream staging considered in Phase 4 eliminates this need (except upstream of the Diversion Inlet Structure) as the depth-averaged flow velocities upstream of the Red River Control Structure (and also upstream of the Wild Rice River Control Structure) will be extremely low. Pending additional site specific studies, a minimum width of 40 ft for each bay was recommended to ensure ice passage (note that the width of each bay is 50 ft) when the gates are fully open. The determination of the operational measures to handle river ice when the gates are partially or fully closed is strongly recommended to be worked out during the design phase. The USACE-PDT has retained Andrew Tuthill for additional ice analysis in preparation for the next stage of study and design.

The configuration proposed for the Control Structure resembles a bridge with large openings when no water from the Red River of the North is diverted into the Diversion Channel. As indicated above, the gates would be fully open when the forecasted peak flow of the hydrograph at the USGS gage in Fargo is less than 9,600 cfs. The discussion above indicates that the design criteria require operation of the Control Structure gates during the rising limb of the hydrograph prior to a flow rate of 9,600 cfs occurring at the USGS gage in Fargo in order to minimize downstream impacts. What is important to highlight here is that because modeled flow velocities through the Control Structure are within the range of the measured values when the gates are fully open, fish passage is secured for low to average flow conditions and smaller flood events. Furthermore, the rough bottom of the Control Structure (achieved by adding boulders or baffles) should help create the desired complexity in the flow pattern and areas of relatively low flow velocity.

The results of the Phase 3 2D hydraulic modeling (*for demonstration, not for design*) presented in Exhibit H suggest that areas of low flow velocity would exist at all bays when the gates are fully open. More specifically, for a flow of 6,100 cfs, velocities for with-project were similar to velocities for existing conditions (see Exhibit A). Velocities in the Red River of the North for both existing and with-project conditions were roughly 1.5 fps. Because the configuration of the Control Structure bays was not changed in Phase 4, the results of the 2D hydraulic modeling when the gates are fully open are also valid for the Phase 4 feasibility design. However, when the gates are lowered, the modeled flow velocities through the openings could be as high as approximately 30 fps in

Phase 4 (they were also very high in Phase 3). The Control Structure then becomes a barrier for fish passage. To solve this problem, a secondary by-pass channel consisting of riffles and pools was included in the feasibility design (see Section F2.2.2).

F2.2.2 Fish Passage on Red River of the North

A fish passage channel was designed to allow fish to travel from downstream to upstream of the Red River Control Structure when the gates are partially closed and flow velocities are very high at the primary bays. The fish passage would allow fish migration for flows up to the 50-yr flood event on the Red River of the North. Fish passages at the Red River Control Structure (and also at the Wild Rice River, Lower Rush River and Rush River) are designed with consideration to several criteria based on discussions with the USACE-Environmental, the Natural Resources Agencies, and the feasibility design in the USACE"s Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H. The design criteria include:

- Inclusion of pool-riffle sequences, allowing fish to rest in pools between areas of high velocities in riffles.
- Maximum flow velocities of approximately 6 fps in riffles and gated area of the fishway.
- Average velocities of approximately 1.5 fps in pools.
- Average slope of between 1% and 3%.
- Minimum depth of 1 ft at the Control Structure entrance to the fish passage.
- Design flow of 1% to 2% of the flow through the Control Structure entrance.
- Minimum downstream invert 1 to 2 ft below the 5-yr tailwater elevation.

The fish passage designed for the Red River Control Structure includes two parallel channels, each including alternating pools and riffles. Multiple channels instead of a single switchback channel (as proposed in the Phase 2 design) are necessary because of the nearly 5-ft variability in headwater elevations between the 10-yr and 50-yr events in the Red River of the North (see Exhibits A and D). A single fish passage channel capable of operating during the smaller flood events cannot maintain velocities acceptable for fish passage during the larger flood events. Two fish passage channels with entrance inverts staggered vertically by 3.5 ft are necessary to cover the wide range in headwater elevation for which the fishway would be active while maintaining velocities less than 6 fps through the riffles and the Control Structure entrance. The critical condition (i.e., highest velocities and highest flows) occur when the headwater at the Control Structure entrance to a fish passage channel is the greatest (approximately 4.5 ft). Three adjacent gates (10 ft wide by 5 ft tall, with identical invert elevations) are proposed at the Control Structure entrance to each fish passage channel. The reason for having a gated entrance is to prevent water from entering a fish passage channel when the headwater elevation is too high (thus limiting the maximum flow through the fish passages). When the gates of a given fish passage channel are closed, the gates leading to the next fish passage channel with a higher entrance invert would be opened.

Additional details, tables, and figures pertaining to the design of fish passages at the Red River Control Structure are presented in Exhibit G, but pllease see Figure F31 for a rendering of the fish passage system for the 10-yr flood event in the Red River of the North. Each riffle is designed to achieve a 1 ft drop in elevation over a 20 ft reach. Pools at least 40 ft in length are located between each riffle and have a depth of at least 5 ft. The minimum pool dimensions are based on achieving a volumetric energy of between 3.1 to 4.2 ft pounds per second per cubic foot. Each pool has a bottom width on the order of 40 ft. The first riffle in the pool-riffle sequence of each fish passage channel (i.e., downstream of the gated section on the wingwall of the Control Structure) has a narrower width intended to limit the flow through the fish passage structure at higher headwater elevations. Subsequent riffles have widths similar to the pools in order to simplify the design. The pool-riffle sequence of each fish passage channel result in an overall slope of about 2%. The two fish passage channels on the Red River Control Structure include 15 and 11 pool-riffle sequences, respectively. The number of sequences is based on the drop between the upstream gate invert and the downstream 5-yr tailwater elevation in the Red River of the North. The Phase 4 work with the HEC-RAS unsteady flow models did not include the 5-yr event, so for this analysis the 5-yr water surface elevation was chosen to be 1-ft below the 10-yr water surface elevation. The two fish passage channels converge into a single channel at the confluence with the exit channel of the Control Structure, downstream of the domain of the submerged hydraulic jump that will develop downstream of the primary Control Structure when the gates are partially closed. The results of the 2D hydraulic modeling conducted in Phase 3 that are presented in Exhibit H show that flow velocities at this downstream entrance to the fishway are relatively small (less than 1 fps). The general configuration of this downstream entrance to the fishways has been maintained in Phase 4.

The flow through the fishway ranges from 40 cfs to about 600 cfs (or 0.4% to 6% of the total flow through the Control Structure). Reducing the range of flow through the fish passage channels would require more channels with smaller vertical spacing, but this could significantly increase the construction cost of the Control Structure and demand a very active operation during large flood events. In addition, for cost estimating purposes (see Appendix G) the feasibility design assumes that an approach channel will be constructed upstream of the fishway entrance to the Control Structure. This is not required for the proper hydraulic functioning of the fishway, but it is anticipated that the Natural Resources Agencies would prefer to see a fish route that continues several hundred feet upstream of the Control Structure to reduce the risk of fish being swept downstream through the main bays of the Control Structure. The results of the 2D hydraulic modeling conducted in Phase 3 that are presented in Exhibit H indicate that this concern is not necessarily justified, as the depth-averaged flow velocities upstream of the Control Structure are not that high; the velocities near the bottom of the water column are the ones that are high, not the ones near the water surface where the fish would return to the approach channel of the Control Structure. With the upstream staging considered in Phase 4, this concern is less valid.

The Phase 4 feasibility analysis also included the evaluation of the operation frequency of the Red River of the North fish passage. Detailed records exist of the Red River of the North discharges at the USGS gage in Fargo for the past 108 years. Correlating (based on exceedance probability) the maximum operable water surface elevations of each fish passage to a discharge in the Red River of the North at the USGS gage in Fargo allows quantification of the fish passage"s closure frequency. Table F4 present the results of this correlation, which was accomplished by noting the peak water surface elevations for each Phase 4 design flood (10-yr, 50-yr, 100-yr and 500-yr flood events) and determining the corresponding peak discharge in the Red River of the North at the USGS gage in Fargo from the HEC-RAS unsteady flow models. An exponential function was fit to this data and used to interpolate the discharges at the USGS gage in Fargo for each fish passage operational water surface elevation range. Table F4 includes the number of events and number of days which exceeded each fish passage"s design open (individual passage allows fish movement) and closed (individual passage prohibits fish movement) water surface elevations for the 1901 to 2009 historic record. Two caveats of this analysis need to be highlighted. First, there were only four data points available for the curve fitting. Second, the lowest and highest water surface elevations in Table F4 extend outside of the available data range.

F2.2.3 Diversion Inlet Structure

Because of the upstream staging required in the Phase 4 feasibility design, the Wild Rice River east and west weirs considered in Phase 3 have been dropped in Phase 4. It is also important to indicate that the Connecting Channel between the Red River of the North and Wild Rice River and the one between the Wild Rice River and the Diversion Inlet Structure are mostly intended to facilitate drainage during average flow to frequent flood events rather than to enhance hydraulic conveyance in these areas during the larger design floods.

The configuration of the Diversion Inlet Structure depends on the configuration of the Control Structures on the Red River of the North and Wild Rice River, as well as on the amount of upstream staging required to minimize impacts on flood levels downstream of the Outlet Structure. Different combinations of proposed configurations for these hydraulic structures are presented in Exhibit B, including a qualitative assessment of the advantages and disadvantages of each concept in terms of hydraulic performance, handling of flood flows and low flows, potential environmental impacts, permitting, and operation and maintenance.

For this feasibility design, the concept selected for the Diversion Inlet Structure is the one that (when combined with the Control Structures on the Red River of the North and Wild Rice River) would better accomplish the goals outlined above. This concept consists of a passive (i.e., no gates or movable parts) weir located west of the Wild Rice River. To be more precise, the Diversion Inlet Structure is located downstream of the Storage Area 1 Inlet-Outlet Opening (see Section 2.2.15), closer to the Sheyenne River. This location was selected to maximize the area available for staging and storage immediately upstream of the Protected Area. Because both the (hydraulic) head available and the (physical) drop in elevation at the Diversion Inlet Structure significantly increased in Phase 4 with respect to Phase 3 (see Figure F02), and also because this is a key structure for the overall performance of the proposed diversion, the design concept has changed from the sheetpile-rockfill protected weir in Phase 3 to an ogee-type concrete spillway in Phase 4.

As indicated above, the Diversion Inlet Structure will control the flows diverted from the Red River of the North and Wild Rice River into the Diversion Channel. The crest elevation of this weir is 1 ft above the water surface elevation associated with the 3.6-yr flow and is approximately 1.5 ft lower than the 5-yr water surface elevation in the Red River of the North at the location of the Red River Control Structure. So for flows greater than or equal to the 5-yr event (6500 cfs in the Red River of the North at the location of the Red River Control Structure and 10,200 cfs at the USGS gage in Fargo, based on Phase 3 hydrology), water would flow from the Red River of the North and Wild Rice River into the Diversion Channel. The hydraulic design of the Diversion Inlet Structure is summarized in Table F3 and is presented in greater detail in Exhibit D. Refinements to the configuration of the Diversion Inlet Structure might be needed during final design to reduce staging for the larger flood events.

As indicated in Section F2.2.1, there is a need for placing an ice boom upstream of the Diversion Inlet Structure and this has been included in the Phase 4 feasibility design. On the other hand, 2D hydraulic modeling conducted in Phase 3 is not applicable to Phase 4 because the average flow velocities through this structure are different given the change from highly submerged flow conditions in Phase 3 to free flow in Phase 4 (induced by the significant hydraulic head due to upstream staging). More importantly, the vertical drop at this location (see Figure F02) represents a barrier for fish in the Diversion Channel to migrate upstream through the Diversion Inlet Structure. However, it is assumed that fish swimming into the Diversion Channel through the Outlet Structure will have the possibility to migrate upstream of the Rush River or Lower Rush River (see Sections F2.2.11 and F2.2.13).

F2.2.4 Control Structure on Wild Rice River

For the LPP, a Control Structure located on the Wild Rice River north (immediately downstream) of the Connecting Channel (but upstream of the confluence with the Red River of the North) is necessary to limit the amount of water flowing into the Protected Area (i.e., the cities of Fargo and Moorhead). The design goal for this structure is similar to that for the one on the Red River of the North, and the qualitative assessment of alternative concepts presented in Exhibit B also applies to this structure.

For this feasibility design, the concept selected for the Wild Rice River Control Structure is the one that would better accomplish the project goals, with the understanding that a single large pool will form upstream of the Control Structures at the Red River of the North and Wild Rice River, from which water will be diverted through the Diversion Inlet Structure into the LPP Diversion Channel. The concept for the Wild Rice River Control Structure consists of a concrete gravity dam with two 30 ft-wide bays (using primary tainter gates, and secondary bulkheads in case the tainter gates malfunction), and wingwalls. In addition, a gated secondary by-pass channel for fish passage (i.e., the fishway) will be located on one of the sides of the primary Control Structure. Following discussions with the USACE-PDT, the Control Structure is recommended to be built off the existing Wild Rice River channel.

The feasibility design considerations used to size the Wild Rice River Control Structure are very similar to those of the Red River Control Structure. Therefore, they will not be repeated here, except for some specifics applicable to this structure.

Exhibit A presents a summary of background hydrologic information for the Wild Rice River. The hydraulic design is summarized in Table F2 and is presented in greater detail in Exhibit D. It is important to mention here that in the design phase, further evaluation of the operational scheme of the Control Structure needs to be refined for local flood events in the Wild Rice River. Related to this recommendation, it must be realized that the operational rules set for the Red River Control Structure will constrain those developed for the Wild Rice River Control Structure (and vice versa).

Regarding ice design considerations, the width of 30 ft set for each bay is less than the suggested minimum width of 40 ft (see Exhibit J). It is recommended that in the design phase, the dimensions of the bays are re-evaluated. One option is to work with one bay 40-ft wide and the other 20-ft wide. The underlying assumption is that ice would be directed to the wider bay by appropriate alignment of ice booms. However, the upstream staging considered in Phase 4 eliminates this need (except upstream of the Diversion Inlet Structure) as the depth-averaged flow velocities upstream of the Wild Rice River Control Structure will be extremely low. Still it is recommended to further evaluate in the design phase an alternative configuration that has only one gated bay, likely wider than 30 ft, and for which the redundancy would be biased toward the mechanical/electrical components of the gate operation.

As indicated above, the proposed design of this Control Structure includes an ungated area below the main gates that is intended to allow the Wild Rice River to pass bedload transport through the Control Structure when the gates will be partially closed. Different from the Red River of the North, the Wild Rice River appears to be a sand-bed river but the preliminary assessment of project impacts on sediment transport and geomorphology presented in Exhibit I shows that sediments are basically transported is suspension, and that most of these sediments are silts and clays. In addition, the related analysis of impacts of the Horace to West Fargo diversion (which has been in place for nearly 20 years) on the sediment transport and geomorphology characteristics of the Sheyenne River suggest that the impacts of a diversion would not be significant. However, the proposed design with the lower ungated area is still recommended to prevent sediment accumulation that otherwise would occur if the bays are closed down to the invert of the Control Structure.

What is important to highlight here is that because modeled flow velocities through the Control Structure are within the range of the measured values when the gates are fully open, fish passage is secured for average flow conditions to smaller flood events. Furthermore, the rough bottom of the Control Structure (achieved by adding boulders or baffles) should help create the desired complexity in the flow pattern and areas of relatively low flow velocity.

The results of the Phase 3 2D hydraulic modeling (*for demonstration, not for design*) presented in Exhibit H suggest that areas of low flow velocity would exist at all bays when the gates are fully open. More specifically, for a flow of 2350 cfs, which is larger than the peak of the 2-yr flood event, velocities for with-project conditions were somewhat similar to velocities for existing conditions. Peak velocities in the Wild Rice River for existing conditions were roughly 2.0 fps, whereas for with-project conditions were roughly 2.5 fps at the Control Structure and roughly 1.5 fps in the natural (undisturbed) Wild Rice River channel upstream and downstream of the diversion. However, when the gates are lowered, the modeled flow velocities through the openings could be as high as approximately 15 fps. The Control Structure then could become a barrier for fish passage. To solve this problem, a secondary by-pass channel consisting of riffles and pools was included in the feasibility design (see Section F2.2.5)

F2.2.5 Fish Passage on Wild Rice River

A fish passage channel was designed to allow fish to travel from downstream to upstream of the Control Structure for flows up to the 50-yr flood event on the Wild Rice River. The feasibility design considerations used to size the fish passage facility on the Wild Rice River are very similar to those on the Red River of the North. Therefore, they will not be repeated here, except for some specifics applicable to this facility. Details, tables, and figures regarding the design of fish passages at the LPP Control Structure are presented in Exhibit G.

The fish passage designed for the Wild Rice River Control Structure includes two parallel channels, each including alternating pools and riffles. The two fish passage channels include 18 and 14 pool-riffle sequences, respectively. The number of sequences is based on the drop between the upstream gate invert and the downstream 5-yr tailwater elevation in the Wild Rice River. The Phase 4 HEC-RAS unsteady flow model did not include the 5-yr event, so for this analysis the 5-yr water surface elevation was chosen to be 1-ft below the 10-yr water surface elevation. All these fish passage channels converge into a single channel at the confluence with the exit channel of the Control Structure, downstream of the domain of the submerged hydraulic jump that develops downstream of the primary Control Structure when the gates of the 30-ft bays are partially closed. The results of the 2D hydraulic modeling presented in Exhibit H (taken from Exhibit G of Appendix F in the Phase 3 report) show that flow velocities at this downstream entrance to the fishway are relatively small (less than 1 fps). The configuration of the bays have not changed in Phase 4, therefore the results of the 2D hydraulic modeling in Phase 3 are valid for the Phase 4 feasibility design.

The flow through the fishway ranges from 40 cfs to about 600 cfs. In the Phase 4 feasibility design, the flows above represent 3.5% to 100% of the modeled flow through the Wild Rice River Control Structure. The current modeling approach involves nearly full closure of the Wild Rice River Control Structure gates and controlling the flow into the Protected Area with the Red River Control Structure. In future models, controlling the flow into the Protected Area will most likely include a different combination of the Wild Rice River and the Red River Control Structures operation. The percentage of flow passing through the fish passage with respect to the Wild Rice River Control Structure will then be reduced, which could imply having more fish passage channels with smaller vertical spacing and in turn result in an increase of the construction cost of this facility. It is strongly recommended to revisit the design criterion for fish passage at this location in the next stage of study and design, especially when the fish passage infrastructure on the Red River of the North operates up to the 50-yr flood event.

The gates at the upstream ends of the fish passages will be placed in the west wingwall of the Control Structure. The selection of this location is to favor fish migration upstream of the Wild Rice River rather than through the Connecting Channel to the Red River of the North. Actually, to put things in the right context, for larger flood events when the diversion works need to be operated, fish migrating upstream will encounter a pool of water where flow velocities will be relatively low.

F2.2.6 Diversion Channel Transition and Aqueduct at the Sheyenne River

The general design considerations discussed below also apply to the Diversion Channel transition and aqueduct at the Maple River, which is presented in Section F2.2.8. On the other hand, the 3D renderings of the Diversion Channel Transition and aqueduct at the Maple River that are presented in Figures F42 through F57 may be helpful to better visualize the feasibility design proposed for the Diversion Channel Transition and aqueduct at the Sheyenne River.

A combination of three hydraulic structures is proposed at the LPP Diversion Channel crossing of the Sheyenne River; a transition on the Diversion Channel, and an aqueduct and a spillway on the tributary. An aqueduct structure was chosen for three main reasons: there is a significant difference in the elevations (approximately 15 ft) of the thalweg in the Sheyenne River and the invert of the Diversion Channel, local sponsors prefer to minimize the number of "active operation" structures, and it provides a good solution for fish and ice passage. One intention of these structures is to allow conveyance of the entire tributary flow into the Protected Area for flows up to the local 2-yr flood event in the Sheyenne River. Allowing flows up to the 2-yr event will minimize impacts to aquatic ecosystems, fish passage, sediment transport and channel morphology upstream and downstream of the proposed diversion. On the other hand, the other intention of these structures is to maximize diversion of tributary flows into the LPP Diversion Channel for flows larger than the local 2-yr flood event in the Sheyenne River. During these times when a portion of the flow is diverted, the structures will allow a fraction of the Sheyenne River flow (somewhat greater than the local 2-yr flow) to pass through the aqueduct to the Protected Area while maximizing flows diverted (through the spillway) to the diversion channel. This Section F2.2.6 discusses the design of the first two of the three structures referred to above, with the hydraulic functioning being governed not only by the local flood events but also by the coincidental (to the peaks in the Red River of the North) flood events in the Sheyenne River.

Different combinations of proposed configurations for the Diversion Channel transition and the aqueduct at the Sheyenne River are presented in Exhibit C, including a qualitative assessment of the advantages and disadvantages of each concept in terms of effectiveness for fish passage through the aqueduct, reduction of risk for winter freezing of low flows in the aqueduct, suitability to handle sediment transport and ice issues in the tributary and LPP Diversion Channel, overall hydraulic performance, and cost considerations. For this feasibility design, the concept selected is the one that would better accomplish the project flood damage reduction goals, while minimizing potential environmental and geomorphologic impacts on the Sheyenne River. This concept consists of a transition in the Diversion Channel from the earth-excavated sections upstream and downstream of the tributary crossing to a 250-ft wide concrete lined section at the tributary crossing with 7:1 slopes on either side. In addition, a concrete open channel (approximately 50 ft wide by 17 ft deep) is proposed for the aqueduct to convey tributary flows over the Diversion Channel. The aqueduct will include a smaller rectangular channel (approximately 10 ft wide by 10 ft deep) to ensure that low flows during winter will not freeze; no inverted siphons are recommended to handle low flows. In comparison to the concept design consisting of closed box culvert aqueducts presented in the August 31, 2009 submittal (Phase 1 report), working with an open channel allows for natural light into the crossing, which may help encourage fish passage through the aqueduct and may reduce the possibility of freeze/thaw damage to this structure.

Following discussions with the USACE, the aqueduct and Diversion Channel transition are recommended to be built off the existing Sheyenne River channel. It is important to highlight here that the feasibility design proposed is largely determined by the existing profile of the Sheyenne River, the magnitude of the tributary flow to pass into the Protected Area (a maximum of 2000 cfs, to maintain the current level of flood protection provided by the Horace-West Fargo diversion), and the very high flows in the Diversion Channel at the Sheyenne River crossing (approximately 18,100 cfs for the 500-yr flood event).

Exhibit A presents a summary of background hydrologic information for the Sheyenne River. This information combined with the main project goal of reducing the flows to pass into the Protected Area for tributary flows larger than the local 2-yr flood event, without negative impacts on fish passage or river geomorphology, guided the hydraulic design that is summarized in Table F5 and is presented in greater detail in Exhibit D. More specifically, the design of the aqueduct followed two basic design considerations. First, following input from the USACE-Environmental and the Natural Resources Agencies, the cross sectional-averaged flow velocity in the aqueduct should be the same as that in the natural Sheyenne River channel for the local 2-yr flood event (this determined the width of the aqueduct). The 50 ft width was determined during Phase 3 and was adopted in Phase 4. Second, the height of the aqueduct would avoid overtopping of the aqueduct into the Diversion Channel and vice-versa. Both objectives were achieved in the current feasibility design. In addition, the invert of the aqueduct was lowered approximately 0.5 ft to contain flows up to 2,000 cfs within the aqueduct. At this point along the diversion channel, the flows are low enough that raising the aqueduct to improve flow in the Diversion Channel through the structure is not necessary. In comparison to the natural variation in the Sheyenne River channel bottom, this lowering of the thalweg is insignificant since gradual transitions will be used on both the entrance and exit to the aqueduct. From the Phase 4 unsteady flow modeling, the velocities in the aqueduct will never be greater than 3 fps and will always remain in the range of measured existing velocities as seen in Exhibit A. Although the aqueduct will be constructed of concrete, the concrete will be formed in such a way to include roughness elements that will create complexity in the flow patterns, and consequently encourage fish passage upstream of the Sheyenne River.

The cross sections of the approach and exit channel to the aqueduct are assumed to have sideslopes of 5H:1V (see Exhibit D). Steeper slopes are a better match to the existing cross sections of this tributary. However, the stability analysis, using a factor of safety of 1.4, recommends shallower slopes (see Exhibit N). Therefore, slopes of 5:1 were used for both cost estimating purposes and hydraulic design.

The dimensions of the Diversion Channel transition have been determined based on a parametric study presented in Exhibit C, which compares the hydraulics for variations of different structural dimensions of the aqueduct and provided guidance (following an iterative process) to the structural design of the aqueduct presented in Exhibit R. As indicated above, a portion of the Diversion Channel transition (extending upstream and downstream of the aqueduct crossing) would be concrete lined, such that the reduced channel roughness (compared to the earth excavated sections of the Diversion Channel) would translate into increased velocities that would compensate the reduction in flow area without resulting in greater water depths. The upstream transition in the Diversion Channel would be a radial vertical wall with a radius of approximately 200 ft. The downstream transition is an approximately 350-ft long tapered vertical wing wall which is lined at the bottom with concrete for the first 100 ft and then protected at the bottom with rip-rap. With this configuration, the goal has been to limit the flow velocities at the aqueduct crossing to approximately 10-15 fps, or alternatively to a head loss that does not compromise the overall hydraulic conveyance of the diversion channel.

The opening under the aqueduct has been set at the same 250-ft width of the earth excavated Diversion Channel bottom. Structural analysis determined supporting the aqueduct requires pier widths of 3 ft, a pier spacing of 30 ft (centerline to centerline), and a 2 ft minimum aqueduct bottom deck thickness. The invert of the Diversion Channel cross section has been lowered by 3 ft from the typical down to the invert of the low flow channel. Pressurized flow in the Diversion Channel underneath the aqueduct crossing will occur for events in the Red River of the North greater than or equal to the 50-yr event. A summary of the hydraulic design is summarized in Table F5, and it is presented in greater detail in Exhibit D.

2D hydraulic models of the Sheyenne River crossing structures were created during Phase 3 to assess the (vertically-averaged) velocity distribution as it relates to fish passage. This modeling was not revised during the Phase 4 study because flow conditions in and the feasibility design of the aqueduct, which is the critical route for fish passage upstream of the Sheyenne River, have changed only slightly from Phase 3 to Phase 4. The following results concerning velocities are from Phase 3. Existing conditions and with-project conditions were analyzed for several flow conditions including the 2-yr local, and 5-yr, 10-yr, and 50-yr coincidental events. For the 2-yr flow, velocities for with-project conditions were similar to velocities for existing conditions. Peak velocities through the aqueduct crossing and in the existing channel were roughly 1.5-2.0 fps. For the 5-yr flow, peak velocities in the diversion channel were roughly 1.5-2.0 fps. For the 10-yr condition, velocities for with-project conditions through the aqueduct crossing were roughly 2.0 fps and roughly 2.0-2.5 fps upstream and downstream in the natural channel. For the 50-yr condition, peak velocities in the LPP Diversion Channel upstream and downstream of the aqueduct crossing were roughly 2.5-3.0 fps and increased to roughly 6.5 fps in the Diversion Channel directly under the aqueduct. These flow velocities in the diversion channel near the aqueduct have increased from Phase 3 to Phase 4, in particular for the larger flood events (50-yr event and above) when flow under the aqueduct becomes pressurized, but fish passage under the aqueduct is not a feasibility design target for these very extreme events. Peak velocities in the existing channel of the Sheyenne River were roughly 2.5-3.0 fps. An explanation of the modeling methods and a more in-depth discussion of the results are presented in Exhibit H.

F2.2.7 Spillway at Sheyenne River

A weir spillway has been selected to divert waters from the tributary into the LPP Diversion Channel. The main design criteria used for designing the weir spillway are:

- The crest of the weir spillway will be set, as a minimum, at the water surface elevation on the tributary associated with the 2-yr local flood event, allowing the entire 2-yr flow to pass into the Protected Area of the tributary;
- The length of the weir spillway will be such to maximize diversion flows into the LPP Diversion Channel, hence to minimize the maximum flow into the Protected Area of the tributary during the occurrence of the 500-yr local flood event; and
- The weir spillway can maximize diversion flows from the tributary into the LPP Diversion Channel for coincidental events, when the anticipated head available could be less than for local events.

In Phase 3, the spillway was designed with a crest elevation of 912.71 (WSEL at 1,200 cfs) and a width of 55 ft. In Phase 4, the crest elevation was revised to 912.56 due to an updated rating curve. Additionally in Phase 4, the width has dramatically increased to 300 ft due to lower WSELs in the tributary for the larger flood events. Increasing the weir width is necessary to make up for the lower water surface elevations. However, as the weir width increases, the upstream water level continues to drop, making any width changes negligible at large widths. Therefore, 300 ft was settled on for the weir width because it was the minimum size that met the constraint of passing no more than 2,000 cfs through the aqueduct. The resulting design is presented in Table F7 and details about the hydraulic calculations are presented in Exhibit D.

Results from the two-dimensional (2D) hydraulic models in Phase 3 indicate that peak velocities at the entrance to the spillway for the 10-yr condition were 1.5 fps and increase to 5.0 fps near the crest of the spillway. The Phase 4 study did not re-examine this modeling, but it is not a feasibility design criterion to allow for fish passage from the Diversion Channel to upstream of the Sheyenne River. For more information, see Exhibit H.

F2.2.8 Diversion Channel Transition and Aqueduct at Maple River

A combination of three hydraulic structures is proposed at the LPP Diversion Channel crossing of the Maple River; a transition on the Diversion Channel, and an aqueduct and a spillway on the tributary. An aqueduct structure was chosen for three main reasons: there is a significant difference in the elevations (approximately 7 ft) of the thalweg in the Maple River and the invert of the Diversion Channel, local sponsors prefer to minimize the number of "active operation" structures, and it provides a good solution for fish and ice passage. One intention of these structures is to allow conveyance of the entire tributary flow into the Protected Area for flows up to the local equivalent 2-yr flood event in the Maple River. The term equivalent is used here because the 2-yr local flows along the Rush River and Lower Rush River are included in this quantity. The Lower Rush and Rush Rivers are entirely diverted into the Diversion Channel. Therefore, the 2-yr equivalent flow from those rivers (717 cfs combined) is added to the Maple River 2-yr local flow (970 cfs) to obtain the equivalent 2-yr local flow (1,687 cfs). Allowing flows up to the 2-yr event will minimize impacts to aquatic ecosystems, fish passage, sediment transport and channel morphology upstream and downstream of the proposed diversion. On the other hand, the other intention of these structures is to maximize diversion of tributary flows into the LPP Diversion Channel for flows larger than the equivalent 2-yr flood event in the Maple River. During these times when a portion of the flow is diverted, the structures will allow a fraction of the Maple River flow (somewhat greater than the equivalent 2-yr flow) to pass through the aqueduct to the Protected Area while maximizing flows diverted (through the spillway) to the Diversion Channel. This Section F2.2.8 discusses the design of the first two of the three structures referred to above, with the hydraulic functioning being governed not only by the local flood events but also by the coincidental (to the peaks in the Red River of the North) flood events in the Maple River.

Different combinations of proposed configurations for the Diversion Channel transition and the aqueduct at the Maple River are presented in Exhibit C, including a qualitative assessment of the advantages and disadvantages of each concept in terms of effectiveness for fish passage through the aqueduct, reduction of risk for winter freezing of low flows in the aqueduct, suitability to handle sediment transport and ice issues in the tributary and LPP Diversion Channel, overall hydraulic performance, and cost considerations. For this feasibility design, the concept selected is the one that would better accomplish the project flood damage reduction goals, while minimizing potential environmental and geomorphologic impacts on the Maple River. This concept consists of a transition in the Diversion Channel from the earth-excavated sections upstream and downstream of the tributary crossing to a 250-ft wide concrete lined section at the tributary crossing with 7:1

slopes on either side. In addition, a concrete open channel (approximately 50 ft wide by 21 ft deep) is proposed for the aqueduct to convey tributary flows over the Diversion Channel. The aqueduct will include a smaller rectangular channel (approximately 4 ft wide by 4 ft deep) to ensure that low flows during winter will not freeze; no inverted siphons are recommended to handle low flows. In comparison to the concept design consisting of closed box culvert aqueducts presented in the Phase 1 report, working with an open channel allows for natural light into the crossing, which may help encourage fish passage through the aqueduct and may reduce the possibility of freeze/thaw damage to this structure.

Following discussions with the USACE, the aqueduct and Diversion Channel transition are recommended to be built off the existing Maple River channel. It is important to highlight here that the feasibility design proposed is largely determined by the existing profile of the Maple River, the magnitude of the tributary flow to pass into the Protected Area (a maximum of approximately 3,500 cfs), and the very high flows in the Diversion Channel at the Maple River crossing (approximately 25,300 cfs for the 500-yr flood event).

Exhibit A presents a summary of background hydrologic information for the Maple River. This information combined with the main project goal of reducing the flows to pass into the Protected Area for tributary flows larger than the equivalent 2-yr flood event, without negative impacts on fish passage or river geomorphology, guided the hydraulic design that is summarized in Table F6 and is presented in greater detail in Exhibit D. More specifically, the design of the aqueduct followed two basic design considerations. First, following input from the USACE-Environmental and the Natural Resources Agencies, the cross sectional-averaged flow velocity in the aqueduct should be the same as that in the natural Sheyenne River channel for the equivalent 2-yr flood event (this determined the width of the aqueduct). The 50 ft width was determined during Phase 3 and was adopted in Phase 4. Second, the height of the aqueduct would avoid overtopping of the aqueduct into the Diversion Channel and vice-versa. Both objectives were achieved in the current feasibility design. In addition, the invert of the aqueduct was raised approximately 1.5 ft to allow for more flow area and lower velocities in the Diversion Channel under the aqueduct crossing. In comparison to the natural variation in the Maple River channel bottom, this raising of the thalweg is not significant since gradual transitions will be used on both the entrance and exit to the aqueduct. From the Phase 4 unsteady flow modeling, the velocities in the aqueduct were between 4 and 5 fps. However, Exhibit A shows that these velocities are of the same order of magnitude as measured existing velocities. Although the aqueduct will be constructed of concrete, the concrete will be formed in such a way to include roughness elements that will create complexity in the flow patterns, and consequently encourage fish passage.

The cross sections of the approach and exit channel to the aqueduct are assumed to have sideslopes of 5H:1V (see Exhibit D). These slopes are a good match to the existing cross sections and were stable using a safety factor of 1.4 (see Exhibit N). Therefore, slopes of 5:1 were used for both cost estimating purposes and hydraulic design.

The dimensions of the Diversion Channel transition have been determined based on a parametric study presented in Exhibit C, which compares the hydraulics for variations of different structural dimensions of the aqueduct and provided guidance (following an iterative process) to the structural design of the aqueduct presented in Exhibit R. As indicated above, a portion of the Diversion Channel transition (extending upstream and downstream of the aqueduct crossing) would be concrete lined, such that the reduced channel roughness (compared to the earth excavated sections of the Diversion Channel) would translate into increased velocities that would compensate the reduction in flow area without resulting in greater water depths. The upstream transition in the Diversion Channel would be a radial vertical wall with a radius of approximately 200 ft. The downstream transition is an approximately 350 ft long tapered vertical wing wall which is lined at the bottom with concrete for the first 100 ft and then protected at the bottom with rip-rap. The opening of the Diversion Channel bottom under the aqueduct has been set at the same 250-ft width of the earth excavated Diversion Channel bottom. With this configuration, the goal has been to limit the flow velocities at the aqueduct crossing to approximately 10-15 fps, or alternatively to a head loss that does not compromise the overall hydraulic conveyance of the diversion channel.

Structural analysis determined supporting the aqueduct requires pier widths of 3 ft, a pier spacing of 30 ft (centerline to centerline), and a 2 ft minimum aqueduct bottom deck thickness. The invert of the Diversion Channel cross section has been lowered by 3 ft from the typical down to the invert of the low flow channel. Pressurized flow in the Diversion Channel underneath the aqueduct crossing will occur for events in the Red River of the North greater than or equal to the 10-yr event. A summary of the hydraulic design is summarized in Table F6, and it is presented in greater detail in Exhibit D.

2D hydraulic models of the Maple River crossing structures were created during Phase 3 to assess the (vertically-averaged) velocity distribution as it relates to fish passage. This modeling was not revised during the Phase 4 of the study because flow conditions in, and the feasibility design of the aqueduct, which is the critical route for fish passage upstream of the Maple River, have changed only slightly from Phase 3 to Phase 4.

F2.2.9 Spillway at Maple River

A weir spillway has been selected to divert waters from the tributary into the LPP Diversion Channel. The main design criteria used for designing the weir spillway are:

- The crest of the weir spillway will be set, as a minimum, at the water surface elevation on the tributary associated with the equivalent 2-yr local flood event, allowing the entire 2-yr flow to pass into the Protected Area of the tributary;
- The length of the weir spillway will be such to maximize diversion flows into the LPP Diversion Channel, hence to minimize the maximum flow into the Protected Area of the tributary during the occurrence of the 500-yr local flood event; and
- The weir spillway can maximize diversion flows from the tributary into the LPP Diversion Channel for coincidental events, when the anticipated head available could be less than for local events.

The design concept for the spillway from the Maple River to the LPP Diversion Channel is similar to that used for the spillway from the Sheyenne River to the LPP Diversion Channel.

In Phase 3, the spillway was designed with a crest elevation of 893.32 (WSEL at 1,687 cfs) and a width of 150 ft. In Phase 4, the crest elevation was revised to 893.63 due to an updated rating curve. Additionally in Phase 4, the width has dramatically increased to 300 ft due to lower WSELs in the tributary for the larger flood events. Increasing the weir width is necessary to make up for the lower water surface elevations. However, as the weir width increases, the upstream water level continues to drop, making any width changes negligible at large widths. Therefore, 300 ft was settled on for the weir width because it was the minimum size that met the constraint of passing a maximum of approximately 3,500 cfs. The resulting design is presented in Table F8 and details about the hydraulic calculations are presented in Exhibit D.

F2.2.10 Drop Structure at Lower Rush River

The type of structure recommended for the Lower Rush River is a stepped concrete spillway that will divert all flows directly into the LPP Diversion Channel. Fish passage between the upstream portion of these rivers and the LPP Diversion Channel would be accommodated by a separate low flow channel. The existing portions of the Lower Rush River and Rush River downstream of the Diversion Channel are primarily straight drainage channels and do not display many characteristics typically associated with natural streams. Downstream of the confluence of the Lower Rush fish passage and the LPP Diversion Channel, habitat enhancements and low flow channel meandering would be implemented, thereby increasing the quality and quantity of habitat in these rivers, when compared to existing conditions. This is further discussed in Exhibit K.

To maintain the natural 2-yr flow in the lower stretch of the Sheyenne River (downstream of the confluence with the Maple River), an additional amount of water equivalent to the 2-yr flows from the Lower Rush and Rush Rivers will be allowed to pass into the Protected Area in the Maple River tributary crossing (see Section F2.2.8). Exhibit A presents a summary of background hydrologic information for the Lower Rush River.

Exhibit D presents the hydraulic and structural design of the diversion structures on the Lower Rush River, and the summary results are presented in Table F9. The crest of the stepped spillway has been set about one ft above the invert of the river to divert low and average flows into the fish passage channel. The rise and run of the steps are 0.9 and 1.5 ft, respectively. The stepped spillway is placed several feet upstream of the confluence of the river beds and the LPP Diversion Channel side slope. The stilling basin downstream of the stepped spillway has been sized by calculating the head loss over the steps under skimming flow, for 10-yr floods and larger events. Tailwater effect was not incorporated in sizing the stilling basins. The channel bed from downstream of the stilling basin to the bed of the LPP Diversion Channel is assumed to be lined with riprap.

Two-dimensional (2D) hydraulic models of the Lower Rush River connection to the Diversion Channel were created in Phase 3 (see Exhibit H) to assess the (vertically-averaged) velocity distribution as it relates to fish passage. With-project conditions were analyzed for two flow conditions including the mean annual flow in the river and the 20-yr coincidental event. For the mean annual flow condition, peak velocities in the Lower Rush River were roughly 0.1 fps and approximately 0.5 fps in the Diversion Channel. For the 20-yr condition, peak velocities in the Lower Rush River were roughly 0.5 fps and 2.5 fps in the Diversion Channel. An explanation of the modeling methods and a more in-depth discussion of the results are presented in Exhibit H. It is worthwhile mentioning here that the Phase 4 flows for the 20-yr condition are smaller than those in Phase 3; therefore the 2D modeling in Phase 3 provides a conservative figure of what could be expected with the Phase 4 feasibility designs.

F2.2.11 Fish Passage on Lower Rush River

Fish passage consisting of a single channel of alternating pools and riffles allows fish to move between the Diversion Channel and the Lower Rush River for events ranging up to approximately the 10-yr event on the Red River. Additional details, tables, and figures pertaining to the design of fish passage at the Lower Rush River Drop Structure is included in Exhibit G.

The design criteria used of this fish passage at the Lower Rush River are similar to those used for the Red River of the North (see Section F2.2.2) and will not be repeated, except for some specifics applicable to this site only. For example, an additional design consideration is the use of the fish passage channel to pass low flows in the Lower Rush River into the Diversion Channel (for water surface elevations in the Lower Rush River less than the invert of the Drop Structure). The fish passage at the Lower Rush River was also designed to have passive operation (i.e., it does not have to be actively managed during flood events).

The fish passage designed for the Lower Rush River Drop Structure includes one channel of alternating pools and riffles. The upstream invert of the fish passage channel is set at the approximate channel bottom, thus allowing low flows to pass into the Diversion Channel without significant ponding. The downstream end of the fish passage ends at the low flow channel within the Diversion Channel. No gates are needed at the entrance to the fish passage channel. However, headwater elevations above 4.5 ft over the fish passage invert will result in velocities through the channel that may prevent fish passage during the larger flood events.

The fish passage channel includes 17 alternating pools and riffles, with an overall slope of 2%; the number of sequences is based on the drop between the upstream invert in the Lower Rush River and the invert of the low flow channel downstream in the Diversion Channel. Each riffle is designed to achieve a 1 ft drop in elevation over a 20 ft reach. Pools at least 40 ft in length are located between each riffle and have a depth of at least 5 ft. Each pool has a bottom width on the order of 40 ft. The first riffle in the pool-riffle sequence of each fish passage channel has a narrower width intended to limit the flow through the fish passage structure at higher headwater elevations. Subsequent riffles have widths similar to the pools in order to simplify the design.

The hydraulics of the fish passage channel varies as the headwater elevation at the entrance to the fish passage channel increases. The hydraulics were evaluated for the full range of relative headwater elevations for which a fish passage channel is intended to function, and are presented in Exhibit G. The critical condition (i.e. highest velocities and highest flows) which still allow for fish passage occur when the headwater at the entrance to the fish passage is approximately 4.5 ft. When the headwater on the fish passage entrance is less than 4.5 ft, the maximum velocities through the riffles and gates remain below 6 fps and the average flow through the pools is less than 1.5 fps.

It is very important to indicate here that the combined operation of the Lower Rush River fish passage and drop structure has not yet been evaluated. Because the invert of the fish passage is set below that of the Lower Rush River Drop Structure, the fish passage channel will convey all of the flow below some threshold, above which both the fish passage and drop structure will transfer flow to the Diversion Channel. What this means is that the feasibility cost estimates included in this Phase 4 feasibility design are on the conservative side, and further evaluation or refinement of the fish passage feasibility design could demonstrate that the drop structure is not needed.

F2.2.12 Drop Structure at Rush River

The design of the drop structure at the Rush River is similar to the design for the Lower Rush River outlined in Section 2.2.10. Exhibit D presents the hydraulic design for the diversion structures on the Rush River. The rise and run of the drop structure steps are 1.1 and 1.7 ft, respectively. The stepped spillway is placed several feet upstream of the confluence of the river bed and the LPP Diversion Channel side slope. The stilling basin downstream of the stepped spillways has been sized by calculating the head loss over the steps under skimming flow during 10-yr floods and larger events. Tailwater effect was not incorporated in sizing the stilling basins. The channel bed from downstream of the stilling basin to the bed of the LPP Diversion Channel is assumed to be lined with riprap. The resulting designs are presented in Table F10.

Two-dimensional (2D) hydraulic models of the Rush River connection to the Diversion Channel were created to assess the (vertically-averaged) velocity distribution as it relates to fish passage in Phase 3 (see Exhibit H). With-project conditions were analyzed for two flow conditions including the mean annual flow and the 20-yr coincidental event. For the mean annual flow condition, peak velocities in the Rush River were roughly 0.3 fps and approximately 1.0 fps in Diversion Channel. For the 20-yr condition, peak velocities in the Rush River were roughly 0.5 fps and 3.0 fps in the Diversion Channel. It is worthwhile mentioning here that the Phase 4 flows for the 20-yr condition are smaller than those in Phase 3; therefore the 2D modeling in Phase 3 provides a conservative figure of what could be expected with the Phase 4 feasibility designs.

F2.2.13 Fish Passage on Rush River

The design considerations used for the fish passage at the Rush River are similar to those at the Lower Rush River (see Section F2.2.11), and they will not be repeated here except for some specifics applicable to this facility. Additional details, tables, and figures regarding the design of fish passages at the Rush River Drop Structure are presented in Exhibit G.

The fish passage channel includes alternating pools and riffles. A more detailed discussion of pool and riffle design is included in Exhibit G. Each riffle is designed to achieve a 1 ft drop in elevation over a 20 ft reach. Pools at least 40 ft in length are located between each riffle and have a depth of at least 5 ft. Each pool has a bottom width on the order of 40 ft. The first riffle in the pool-riffle sequence of each fish passage channel has a narrower width intended to limit the flow through the fish passage structure at higher headwater elevations. Subsequent riffles have widths similar to the pools in order to simplify the design. The pool-riffle sequence of each fish passage channel result in an overall slope of about 2 percent. The fish passage channel includes 14 pool-riffle sequences. The number of sequences is based on the drop between the upstream gate invert and the invert of the low flow channel downstream in the Diversion Channel.

It is very important to indicate here that the combined operation of the Rush River fish passage and drop structure has not yet been evaluated. Thus, the total flow through the fish passage channel is not known for specific local and coincidental flood events. Because the invert of the fish passage is set below that of the Rush River Drop Structure, the fish passage channel will convey all of the flow below some threshold, above which both the fish passage and drop structure will transfer flow to the Diversion Channel. What this means is that the feasibility cost estimates included in this Phase 4 feasibility design are on the conservative side, as further evaluation of the fish passage could demonstrate that the drop structure is not needed.

F2.2.14 Outlet Structure to Red River of the North

The Phase 3 Outlet of the Diversion Channel into the Red River of the North consisted of riprap over the downstream 300 ft of the Diversion Channel. This outlet configuration was possible because the Outlet Structure elevation was near the bottom of the Red River. However, with the introduction of staging in Phase 4, the peak flows diverted through the Diversion Inlet Structure was reduced from 35,000 cfs down to 19,000 cfs. As a result the cross sectional area of the Diversion Channel was reduced and the bottom invert was raised. As a consequence of these changes, the drop into the Red River at the Outlet Structure has increased from approximately 11 ft to 20 ft

In order to securely convey flow over the drop from the Diversion Channel to the Red River, the Outlet Structure has been modified from a riprap channel to a concrete ogee type spillway. To prevent ponding at the Outlet Structure, the crest of the ogee spillway was set slightly above the main invert of the Diversion Channel. Additional information regarding the hydraulic design of the Outlet Structure from the Diversion Channel can be found in Exhibit D.

Two-dimensional (2D) hydraulic models of the Outlet Structure were created in Phase 3 (reprinted in Exhibit H) to assess the (vertically-averaged) velocity distribution as it relates to fish passage. Existing conditions were compared to with-project conditions for two flow conditions, the 2-yr and the 20-yr events. For the 2-yr flow, velocities for with-project conditions were similar to velocities for existing conditions. Peak velocities in the Red River of the North for both existing and with-project conditions were roughly 2.0-2.5 fps. For the 20-yr flow, velocities downstream of the Outlet Structure for with-project were slightly higher than for existing conditions. Downstream peak velocities in the Red River of the North for existing conditions were roughly 0.5 fps and 1.0 fps for with-project conditions. Peak velocities in the LPP Diversion Channel were roughly 1.5-2.0 fps. Notwithstanding the feasibility design proposed in Phase 4 for the Outlet Structure is different from that in Phase 3, the flows at the Outlet Structure are smaller in Phase 4.

F2.2.15 Storage Area 1

The hydraulic design of Storage Area 1 focused on assessing alternatives inlet and outlet controls. Exhibit E presents the feasibility design analysis for Storage Area 1. This feasibility design is also presented in Drawings S-414 and S-418 through S-420. The footprint of Storage Area 1 is 4360-acres. The peak storage during the 100-yr and 500-yr design flood events is over 55,000 acre-ft. During flood events water enters and leaves Storage Area 1 through the 1400-ft wide Inlet-Outlet Opening near the Wild Rice River Control Structure at the southeast corner of the storage area. The hydraulic analysis of Storage Area 1 evaluated the benefits of different opening widths, inlet elevations and locations along the south side of the storage area.

The inlet elevation generally has the largest effect on smaller flood events. A higher inlet elevation delays the point at which Storage Area 1 begins receiving water from the diversion system. It also increases the amount of water that is retained in the storage area after the flood has passed. Existing ground elevations set the practical lower limit for an inlet elevation. The existing ground along the southern portion of Storage Area1 ranges from elevation 911 to 915. The preliminary design uses an elevation of 910, with the assumption that some grading will be required to facilitate internal drainage within the storage area and also provide a way for water to enter the area that is slightly below existing grades. By setting the inlet elevation as low as feasibly possible, the storage area can be utilized during smaller flood events as well as large ones.

The hydraulic analysis result indicated that the location of the opening affected the peak stage in Storage Area 1. By placing the opening further upstream, the Diversion Channel profile is higher, so the peak stage in the storage area will also be higher. Although, it needs further assessment to determine whether there will actually be much of a drop in the water surface elevation between the Wild Rice River Control Structure and the Diversion Inlet Structure. The staging area and Storage Area 1 will behave like a large reservoir, which means the hydraulic grade line will be flat. The location of the Inlet-Outlet Opening is governed by site constraints rather than hydraulic benefit.

The hydraulic analysis looked at widths for the Inlet-Outlet Opening that ranged from 1000-ft to 10,000-ft. Longer opening widths tended to reduce the upstream staging depth, but increased downstream flood elevations. The differences between the different openings tested were very minor; on the order of a few hundredths of a foot. The effects were not consistent across the different design flood events. However, the main conclusion is that as long as the opening is large enough for the water in Storage Area1 to equalize with the rest of the staged water to the south, the optimal opening width becomes governed by construction cost issues rather than a hydraulic constraints.

Although not modeled, the analysis also considered leaving the south side of Storage Area 1 completely open. This option was not carried forward for several reasons, including: one, the diversion channel spoils need to go somewhere and it will be cheaper to place them adjacent to the channel; and two, having the South Levee will reduce the fetch length for wave action across the stage area and Storage Area 1.

F2.2.16 Wolverton Creek

For the LPP, a Control Structure located on Wolverton Creek is necessary to limit the amount of water flowing into the Protected Area (i.e., the cities of Fargo and Moorhead). The proposed Wolverton Creek Control Structure functions as an open-close structure and is shown in Drawings S-411 and S-412. In other words, the Control Structure remains completely open during low flow events when it is desirable to have little impact on flows and water surface elevations during the smaller, more frequent flood events. During larger flood events the gates are completely closed. The flows on Wolverton Creek are very small compared to flows on the Red River and Wild Rice River which determine how high water is staged upstream of the project. For this reason, the gates on Wolverton Creek are fully closed, and flows conveyed into the Protected Area are controlled by the gates located on the Red River and Wild Rice River. The gates at the Wolverton Creek Control Structure would be opened following the flood event.

The number of the gates in the Wolverton Creek Control Structure is driven by the ability of the design to provide similar conveyance capacity to the culvert crossing that currently exists. Currently two 10x10ft box culverts are located below 130th Avenue South. The proposed Control Structure provides similar capacity culverts with functionality to close, or restrict, flows conveyed into the Protected Area. Exhibit A presents a summary of background hydrologic information for Wolverton Creek, and the hydraulic design of the Wolverton Creek Control Structure is presented in greater detail in Exhibit D.

F2.2.17 Local Drains

The design of the drop structure at Drain 14 is similar to the design for the Rush River and Lower Rush River (see Sections F2.2.10 and F2.2.12). Exhibit F presents the hydraulic design for the stepped drop structure. The drop structure is also shown in

Drawing S-430. The rise and run of the drop structure steps are 0.7 and 1.5 ft, respectively. The stilling basin downstream of the stepped spillway has been sized by calculating the head loss over the steps under skimming flow during 10-yr floods and larger events. Tailwater effect was not incorporated in sizing the stilling basins. The channel bed from downstream of the stilling basin to the bed of the LPP Diversion Channel is assumed to be lined with riprap.

The remaining local drainage inlets into the Diversion Channel were not resized as part of the Phase 4 analysis. The flows associated with these inlets are very small compared to those of the primary hydraulic structures discussed above. However, during development of the HEC-RAS unsteady flow model, the outlet elevations for local drainage inlets assumed during previous phases were verified to ensure that they were above the existing topography, and conveyed drainage from the adjacent storage areas in the HEC-RAS unsteady flow model.

As part of the Phase 4 analysis, impacts to the floodplain west of the Diversion Channel, between the Sheyenne River and Maple River were characterized to quantify the affect the Diversion Channel has on the existing floodplain. In general the majority of the floodplain located to the west of the Diversion Channel is not impacted by the project. The portion of floodplain most impacted by the project is located along Drain 14, immediately upstream of the Diversion Channel. In this location the water surface elevation in the Diversion Channel is higher than the spillcrest of Drain 14, and water from the Diversion Channel flows into Drain 14 resulting in an increase to the 100-yr flood elevation in this area. Potential methods to mitigate this increase in flood elevations were not analyzed during this feasibly phase, but should be considered during final design. Exhibit F includes additional details regarding the impacts to the floodplain west of the Diversion Channel.

F2.2.18 Standard Project Flood (SPF) Analysis

The tie-back levees south of the project were determined by analysis of the Standard Project Flood (SPF). Levee heights were selected so that during an SPF event, which is larger than the 500-yr event, flows will overtop County Road 17 and be conveyed west prior to overtopping the main east-west levee and flowing into the Protected Area. SPF hydrographs were provided by the USACE in order to set the levee heights south of the project.

The top of the north-south levee along County Road 17 was set at elevation 923, or the elevation to which water is staged during floods larger than the 100-yr event. The top of the east-west levee (i.e., the main levee that runs through the Control Structures on the Wild Rice River, Red River, and Wolverton Creek) was set at elevation 927.

The HEC-RAS unsteady flow model was used to calculate the water surface elevation upstream of the project during an SPF event. This analysis made a number of assumptions, which are listed below. During final design these assumptions should be checked, and additional analysis of the SPF event should be completed along the complete length of the LPP Diversion Channel to identify locations that may fail during an SPF event.

- During the SPF event, there is no tailwater restricting flows over County Road 17
- During an SPF event, the Control Structure gates will operated similarly to the 500-yr event
- Rating curves from the 500-yr model were used in locations were the model was clipped

The SPF analysis indicated that the water surface elevation in the staging area immediately upstream of the Red River control structure will reach 925.2, or 1.8 ft below the top elevation of 927 at which the east-west levee is overtopped and flow is allowed into the Protected Area.

F3.0 GEOTECHNICAL ENGINEERING

The geotechnical engineering of the hydraulic structures, including description of available geotechnical data, seepage analysis, slope stability and pile capacity, are described below.

F3.1 Review of Geotechnical Data

The existing geotechnical information from the FMMFS was reviewed (see Exhibit L). This included the field and laboratory data including index properties, shear strength, compressibility, and permeability. The information available was complete for the fesibility design Phase 3 of work and only two additions or revisions of the existing data were made for use in this Phase 4 report. It should be noted that data from the Spring/Summer 2010 field investigation and associated laboratory testing campaign were incorporated into the previous Phase 3 dataset and this report reflects the new data.

The first addition was the interpretation of the cone penetration tests (CPT) to estimate the undrained shear strength. In this regard, the existing information was further expanded with additional values of undrained shear strength obtained from the CPT. The undrained shear strength is a very important parameter for deep foundation design and thus this step was necessary to expand the understanding of this soil parameter. All the structures will be supported by deep foundations for this project and thus the undrained shear strength becomes important in the design of this foundation type.

The other aspect reviewed was the drained shear strength of the Brenna, OX Brenna, and Argusville formations to be used in the analysis and design of deep foundations under drained conditions. The data presented in the FMMFS study utilized the large strain (15% strain) failure criterion, which accounts for the softening phenomena mainly applicable in design of slope problems. The data were revisited and new failure envelopes were developed using the peak stress failure criterion for the Brenna, OX

Brenna, and Argusville formations of the Lake Agassiz clays. This design failure criterion is applicable in the design of deep foundations.

The soil parameters used in this feasibility phase are adequate and relatively conservative at this feasibility design stage. The preliminary analyses presented herein clearly indicate the areas where additional data will be necessary as the project progresses into final design. The existing data is extensive on the Lake Agassiz clay deposits. However, the existing information on soil characterization of the underlying glacial till is rather limited, though some laboratory testing has been done for Phase 4. Further sampling and testing of the glacial till is still warranted for final design. This is because the proposed pile foundation system will be supported on this unit. Furthermore, in addition to sampling and testing, pile load tests are also recommended. These sampling, testing, and load tests could provide valuable information that could result in significant cost savings.

F3.2 Seepage Analysis

A transient seepage analysis was initially performed as part of Phase 3 to compute the anticipated uplift pressures along the bottom of the Red River Control Structure and the Wild Rice River Control Structure. In the transient seepage model (SEEP/W in the GeoStudio 2007 suite), sheet pile cutoffs were assumed to extend a variable distance into the foundation soils and the width of the structure was also varied. Using full hydrographs for upstream and downstream of the Red River Control Structure (LPP) for the 100-yr event, the development of increased porewater pressures was computed versus time. Then, the highest uplift pressure distribution was provided to the structural engineers performing the feasibility study structural design.

A meeting was held on July 1, 2010, between Barr Engineering, the USACE, Moore Engineering, Houston Engineering, and the City of Fargo at the offices of Barr Engineering in Edina, MN. The uplift pressures on the foundations were discussed in this meeting. It was decided that vertical holes would be installed through the foundation such that the increased pressures from the floodwaters would be transmitted to the foundation bottom to increase the uplift pressures. This would act to reduce the structural loads. Consequently, because the uplift pressures are virtually identical to the increased downward pressures from the floodwaters, the results from the transient seepage analysis were made obsolete. No further description of the seepage analysis is included herein. It should be noted that 10-ft sheet piles are still anticipated to be installed on the upstream and downstream edges of the structures for piping and scour protection.

F3.3 Slope Stability

Slope stability is a major concern for many natural and engineered slopes in the Red River Valley of North Dakota and Minnesota. The soils deposited by glacial Lake Agassiz are medium to stiff and exhibit high plasticity, which leads to low shear strengths. The shear strengths can be especially low under drained (long-term) conditions because of the mineralogical composition of the material, and the drained strength of the material typically controls the design of stable slopes. As such, the slopes along the diversion channel and tributary approach channels at and near the hydraulic structures required a separate stability analysis in addition to the overall channel stability analysis performed by the USACE. The structures for which these additional analyses were performed include:

- Tributary Hydraulic Structure at the Maple River
- Tributary Hydraulic Structure at the Sheyenne River
- Tributary Hydraulic Structure at the Wild Rice River
- Red River Control Structure
- Road and Rail Bridges

The stability of the tie-back levees associated with proposed Storage Area 1 were also analyzed.

The slope stability analysis presented in Exhibit N provides methodology, input parameters, results, and recommendations.

F3.3.1 Methodology

The main objective of the slope stability analysis was to evaluate the stability of earth slopes and associated structural elements near the hydraulic structures to ensure that slope movement does not impact the integrity of the hydraulic structures and impede their intended functions. The stability of the tie-back levees associated with Storage Area 1 is also critical as they will contain a very large volume of water. The impact of groundwater flow on stability was also assessed, and steady-state seepage conditions were used in the stability analysis.

Two types of stability analyses are typically performed for slopes: the Undrained Strength Stability Analysis (USSA) and the Effective Stress Stability Analysis (ESSA). The USSA is performed to analyze the case in which loading or unloading is applied rapidly and excess pore-water pressures do not have time to dissipate during shearing. The ESSA is performed to account for much slower loading or unloading, or no external loading, in which the drained shear strength of the materials is mobilized and no excess pore-water pressures are allowed to develop.

Only the ESSA was performed as part of the slope stability analysis for the hydraulic structures because previous analysis of the main channel slopes identified that the drained (long term) strength of the soils resulted in lower factors of safety. Thus, the ESSA was the controlling case for slope stability. However, for the Storage Area 1 tie-back levees, both cases (USSA and ESSA) were examined because embankment construction could possibly mobilize the undrained shear strength of the material and the USSA could be the controlling case.

For typical long-term conditions, such as with the normal river conditions, the minimum recommended factor of safety for levees and embankments is 1.40 according to USACE standard EM 1110-2-1913, Table 6-1b (USACE, 2003). For typical flood conditions, assuming steady-state seepage, the minimum recommended minimum factor of safety is also 1.40 (USACE, 2003).

Stability analyses for transient (sudden drawdown) conditions are performed for drained and undrained strength parameters. A factor of safety of 1.0 to 1.2 is accepted according to USACE standard EM 1110-2-1913, Table 6-1b (USACE, 2003). A minimum factor of safety of 1.20 was used here for the transient analyses.

SEEP/W and SLOPE/W were the software packages used for seepage and slope stability modeling, respectively. They are both integrated within the GeoStudio 2007 suite. SEEP/W is a finite element groundwater flow model and SLOPE/W is a limit equilibrium slope stability model. Pore-water pressures computed from SEEP/W were imported into SLOPE/W to accurately compute effective stresses.

F3.3.2 Storage Area 1 Levees

A slope stability analysis was carried out to demonstrate that proposed levees associated with Storage Area 1 are stable. The levee geometry was assumed to have a 15-ft crest width, a crest elevation of 927, and 4H:1V slopes. The ground surface was taken to be elevation 908, which is the lowest ground surface elevation found along the levee alignment (along the northern portion). This leads to the highest embankment and represents the most critical embankment cross-section.

To account for the variability in stratigraphy encountered in the soil borings and its impact on levee stability, four stratigraphic cases were analyzed. Using these four stratigraphy types, stability was analyzed for steady-state seepage assuming flood conditions on the interior of Storage Area 1 with the goal of achieving a factor of safety of 1.40 for the ESSA case. A factor of safety of 1.30 was desired for the USSA case.

The ESSA factors of safety were adequate for two of the four stratigraphy types. For the other two cases, a transient seepage analysis was performed. The transient analysis utilizes the 100-yr hydrograph and adjusted permeability values. Details of the transient analysis can be found in Exhibit N. The minimum factor of safety was then reported for the upstream and downstream levee slopes (ESSA and USSA). Using this procedure, adequate stability was computed for all four stratigraphy types.

F3.3.3 Approach Channels

A slope stability analysis was performed on several proposed approach channels to the hydraulic structures to assess their stability. The tributary channels that were evaluated include the Maple River, Sheyenne River, and Wild Rice River as well as the Red River approach. Stratigraphy and soil properties used in the models were provided by the USACE. The initial models were designed with river channel slopes of 3H:1V.

Transient conditions were also evaluated for each river considering the 100-yr and 500-yr flood conditions provided by the Phase 4 HEC-RAS unsteady flow model hydrographs providing water levels for flood conditions over a period of 36 days. It was found that these analyses did not represent the governing case for slope stability, so the results are not presented in the detailed model outputs included in Exhibit N.

At the Maple River tributary approach channel, a 3H:1V slope was determined to be inadequate in terms of stability. A cost analysis was performed to determine at what slope adequate stability would be achieved and what kinds of stabilization alternatives would be required to make a 3H:1V slope stable and the relative costs for each. The stabilization alternatives examined were one and two rows of steel pipe piles on each slope. The flattened slope giving an acceptable factor of safety was 5H:1V. The cost comparison showed that flattening the slope to 5H:1V was much less expensive and this slope was analyzed at the Sheyenne and Wild Rice approach channels. The analysis found that these slopes were stable using the steady-state ESSA case and surpassing a 1.40 factor of safety.

At the Red River Control Structure approach channel, the required factor of safety was met when the channel slopes were 7H:1V. This results from the channel here being significantly deeper than the other approach channels. The deeper channel not only decreases the factor of safety by itself, but the steady-state phreatic surface is higher in the slope, which also leads to lower factors of safety. More details can be found in Exhibit N.

F3.3.4 Hydraulic Structures at Sheyenne and Maple Rivers

The slope stability analysis addressed the global stability of the radial walls at the immediate entrance and exit of the diversion channel crossing of the Maple River and Sheyenne River aqueducts. These structures are critical and the slopes along the approach channels near the structures must remain stable to ensure that no damage to the structures occur especially during diversion channel operation in a flood event.

Global stability was analyzed with the five rows of H-piles beneath the wall footing incorporated into the slope stability model. The factor of safety was computed without and with piles to ascertain the difference between the two cases. For the case with piles, the unfactored shear capacity of the piles was increased until an adequate factor of safety of 1.4 was computed for global stability using the steady-state ESSA case. These required shear capacities were then compared to the actual shear capacity of the HP14x73 piles and found to be well below the actual shear capacity. Thus, the radial walls are considered stable using this simplified limit equilibrium approach. In final design, a more rigorous method should be used which takes into account the interaction of the pile against the soil and better captures the moment capacity of the piles. If this analysis indicates that the factor of safety against global instability is inadequate, additional piles and/or tie-backs may be required. See Exhibit N for more details.

F3.3.5 Road and Rail Bridges

Discussions were had with the USACE regarding the costs associated with improvement techniques to ensure a stable 5H:1V slope at the bridge abutments relative to extending the bridge length to cross the full channel width. Because the improvement techniques could incur significant costs, the cost estimate presented herein uses the extended bridge lengths across the full channel width. However, possible improvement techniques to consider in final design are:

- Piles
- Deep soil mixing
- Lightweight fill

These techniques could be used in association with retaining walls to ensure stability of the abutments.

F3.4 Pile Capacity

Each of the proposed structures is situated with a base elevation that is at or near the Brenna formation, which is incapable of supporting the heavy concrete flood control and aqueduct structures. Commonly, deep foundations such as driven pipe-pile or H-pile are used throughout the region to support bridges or other heavily loaded structures. Typical allowable pile capacities, used by governmental transportation agencies in the area and other organizations, generally range from 60 to 100 tons per pile and all pile used in this region are supported by the glacial till layer which exists below the Brenna formation at depths ranging from 80 to 100 ft below the ground surface. Therefore because of the stratigraphy, difficult foundation conditions, and regional familiarity of deep foundations on governmental projects, deep foundations will be also used for support of each of the structures along the project alignment. For structural reasons, an HP14X73 was used in the analysis. This decision to use the HP14X73 is discussed in subsequent sections of this report but is consistent with many foundations constructed in the region.

F3.4.1 Methodology

The analysis and calculation of axial and lateral pile capacities required that the stratigraphy at each structure location was known and that material strength properties were available. The USACE provided a geotechnical profile of the foundation conditions along the project alignment. The profile was based on investigations performed previously by the USACE.

Soil strength parameters were provided by the USACE, but these parameters were developed using the large strain failure criterion, which is applicable for the evaluation of slope stability. Revised soil parameters for the analysis and design of deep foundations were developed instead, and they are based on laboratory testing that uses the peak deviator stress as failure criterion. The appropriate design parameters were developed using the available laboratory test data and are summarized in Exhibit O.

The EM 1110-2-2906 “Design of Pile Foundations” (15 Jan 1991) manual was utilized as guidance for estimation of axial pile capacities. Although the manual is outdated and other design methods and manuals area available, the methodology presented in the engineering manual provided enough relevant guidance to complete the analysis. The manual requires the calculation of pile capacity using both the drained (effective stress) and undrained (total stress) approach. The appropriate capacity is then used for each design load condition to determine the pile spacing and layout during structural design. As requested by the USACE, both the drained and undrained axial capacities were computed for each structure. In contrast, the current practice as discussed with geotechnical engineers in the departments of transportation in the region is to only calculate and use in design the undrained capacity of the piles founded on the glacial till layer.

On this project, the flood control and aqueducts have large dead loads and minimal live loads except under extreme conditions and therefore the USACE manual requires the use of the drained pile capacity to evaluate the long-term pile capacity. However, in the USACE manual, the drained analysis is only required for the condition when normally consolidated clays are relied upon for pile capacity. At this site, the Brenna, Argusville, and glacial till formations are all overconsolidated with overconsolidation ratios ranging from one to three.

The glacial till layer is very dense or hard and it has been found that during investigations the Standard Penetration (SPT) N-values are greater than 50 blows per ft and even higher values of 100 blows per ft have been reported. Sampling in the layer for laboratory testing has not been performed historically other than SPT samples so there is not significant documented strength data on the glacial till layer because only estimates of the glacial till strength can be made using the N-values. When the piles are driven into the glacial till layer refusal conditions generally prevail and piles can only be advanced a few feet into the glacial till. This indicates that the glacial till has a very high undocumented strength and the use of the undrained pile strength is likely conservative. Typically, pile designers use the structural capacity of the pile element itself when founded on the till. For this updated study, several undisturbed samples were obtained in the till and tested for strength. This information was used to evaluate the drained pile capacities.

Similar to the axial analysis, the drained and undrained lateral capacity of the piles was also computed for use in resisting the lateral design loads. The capacity was determined for the piles at three movement criteria. The criteria are 0.5 inches, 0.67, and 0.875 inches of movement. The criteria were developed based on Phase 3 structural requirements discussed in subsequent sections of the report.

The pile settlement under working loads was also estimated at each of the structures for a single pile.

Two different design software packages were used to facilitate the calculation of pile capacities and settlement. The software packages were Apile developed by Ensoft, Inc.

and AllPile developed by Civiltech Software. The software allowed easy modification of pile attributes and soil conditions. The Apile software uses the API, FHWA, USACE, and Lambda methods to calculate axial capacity. AllPile uses the Navfac axial pile methodology for driven pile. The lateral analyses were compiled using AllPile which uses COM 624, an FHWA-approved design software package. Lpile was also used for the updated analyses to calculate the lateral deflections under load, similar to COM 624. The settlement analyses were completed using AllPile and the Vesic methodology.

F3.4.2 Design Capacity Summary

A detailed summary of the pile capacities at each structure is presented in Exhibit O, and should be used for this feasibility design. The ultimate axial pile capacities ranged from 230 to greater than 500 kips for the undrained condition. These capacities are similar to those used in regional pile design when back-calculations are performed to compare and check skin friction and adhesion as well as end bearing of the piles. Since the piles for this project are much shorter, by 40 to 50 ft, than most bridge foundation piles, the skin friction component is much less than for bridge foundations. However, the end bearing is similar for the same size pile. The ultimate drained or long-term axial pile capacities ranged from 126 to 500 kips and are generally significantly less than the capacities determined for the undrained analysis. The structural capacity of the pile element itself may govern the design capacity used.

The lateral pile capacities range from 21 to 33 kips for the drained condition and 35 to 48 kips for the undrained condition. Settlement of the piles is estimated to be less than 0.5 inches under working loads.

F3.4.3 Recommendations for Final Design

The pile capacities presented for the drained or long-term condition are less than are typically used in the region. Additional sampling and testing is recommended to validate the assumed strength parameters of the glacial till. Such sampling could consist of Pitcher barrel sampling to cut undisturbed samples from the glacial till. The samples should be tested in both drained and undrained triaxial tests with pore pressure measurements. It is likely these tests will show an increase in available tip resistance (end bearing) within the glacial till.

Pile load tests are also recommended prior to final design. The soil formations located in this area are susceptible to strength gain over time. A long-term load test concept should be developed to install test piles at the structures and then load them dynamically and statically to evaluate both the short-term and long-term load carrying capabilities. These tests could show that pile strength gain could contribute additional capacity that could reduce the number of piles and increase their spacing.

F4.0 STRUCTURAL DESIGN

The structural design of the hydraulic structures, including loads, load combinations, reinforced concrete design, pile design, sheet piles, and assumptions are described below. The structural design performed for this Phase 4 study is at the feasibility level only, to support feasibility cost estimates for the proposed project.

F4.1 Structural Design Criteria

The USACE is governed by engineering regulations (ER"s), engineering manuals (EM"s), engineering technical letters (TL"s) and engineering circulars (EC"s). Industry standards were used when USACE criteria were not available. USACE publications used in Phase 4 include:

USACE Engineer Manuals:

- EM 1110-2-1612 "Ice Engineering" (October 30, 2002)
- EM 1110-2-2100 "Stability Analysis of Concrete Structures" (December 1, 2005)
- EM 1110-2-2104 "Structural Design for Reinforced-Concrete Hydraulic Structures" (August 20, 2003 updates)
- EM 1110-2-2200 "Gravity Dam Design" (June 30, 1995)
- EM 1110-2-2502 "Retaining and Flood Walls" (September 29, 1989)
- EM 1110-2-2504 "Design of Sheet Pile Walls" (March 31, 1994)
- EM 1110-2-2702 "Design of Spillway Tainter Gates" (January 1, 2000)
- EM 1110-2-2906 "Design of Pile Foundations (January 15, 1991)

For flood projects the USACE classifies each load combination in accordance with the return period of the event. Load combinations are classified as Usual, Unusual, or Extreme. The Factor of Safety associated with each load case varies with the frequency (probability) of each flood event (classification).

All structures in Phase 4 work were designed in accordance with USACE standards as outlined in the publications and/or as modified from recent discussions (February 1, 2011) with USACE personnel familiar with recent USACE criteria not found in the publications (see Exhibit P).

The effort of the structural design in Phase 4 was to perform preliminary design to a sufficient level to support the project feasibility cost estimate per ER 1110-2-1150 13.5. Stability checks of the individual structures were made to size the overall structure size, members, and foundation requirements so that quantities could be determined to develop the feasibility cost estimate.

F4.1.1 Loads

Loads applied to the structures include:

- Dead Load

- Live load
- Hydrostatic water loads (lateral)
- Uplift
- Ice
- Dynamic Ice (Crushing and flexing Force on Piers from ice floes)
- Wind (during Construction cases)

Since the project is located in a low seismic zone (Zone 0), seismic loads were not evaluated during this phase.

F4.1.2 Load Combinations

The load combinations for each structure will be discussed in Sections F4.2, F4.3 and F4.4.

F4.1.3 Reinforced Concrete Design

Concrete reinforcing was assumed for most structures. When individual concrete members were checked, they were made in accordance with EM 1110-2-2104 using the following criteria:

- A hydraulic load factor, Hf = 1.3 (for members in direct tension, use Hf = 1.65) will be applied in addition to the standard load factors found in ACI 318.
- Fluid pressure shall have a 1.7 load factor.
- For preliminary design of members use a maximum tension reinforcement ratio = $0.25\rho_b$

F4.1.4 Pile Design

Geotechnical capacity, displacements, and structural capacity must be considered in the design of piles. The geotechnical capacity of the piles varies along the project and is covered in Section F3.4. The design of laterally loaded piles is governed by the maximum allowable lateral movement as discussed in the geotechnical section and this section. The interaction between the soil and the pile must be considered in determining the lateral movement.

a. Soil Parameters
Undrained strength parameters were used to determine pile capacities for all short term temporary events, which include all flood cases and construction loading. Drained strength parameters were used to determine pile capacities for long term normal water levels.

b. Axial Loads
A stability analysis was made to determine all the vertical and horizontal loads acting on the structures along with the corresponding resultant location at the base of the pile cap. A rigid cap analysis was then used to determine the axial loads on each pile. The rigid cap method makes a simplified assumption that the foundation cap is perfectly rigid. It is free to translate

and rotate about all axes, but it will not bend. For many cases, this simplified assumption is valid. However, for large groups, thin caps, and/or widely spaced piles, the rigid cap assumption may lead to significant error.

The number of piles and spacing were adjusted so no pile loads exceeded the allowable axial compression, axial tension, and lateral capacity. The Ultimate axial loads (see Section F3.4) were divided by factors of safety based on the load combination classification assuming piles will be load tested (see Load Test below). The factors of safety used include the following:

F.S. = 2.0 (Usual)
F.S. = 1.5 (Unusual)
F.S. = 1.15 (Extreme)

c. Load Test

A pile driven into soft clays (Brenna) tends to disturb the clay around the pile, which results in an immediate loss of shear strength of the clay. After driving a pile, the clay begins to consolidate around the pile and recovers most of its original shear strength (pile set-up) within a month. For this reason, pile driven into soft clays should not be tested for several weeks to a month after driving, depending on the soil characteristics.

d. Horizontal Loads (lateral displacement)

In a group of piles containing only vertical piles, horizontal loads are resisted by bending of the piles. The bending of piles is analyzed using the P-y method. The P-y method for a single pile was used to determine the horizontal capacity in the geotechnical evaluation using COM624 and LPILE. The lateral pile capacity of the pile group is based on the location of the pile in the group. The lateral response of a group of piles is a function of the center-to-center spacing and the pile diameter. The effect of leading and trailing piles on each other may reduce the capacity of the pile. When groups of piles contain battered piles, the analysis becomes more complex. This is because the horizontal component of the axial load in a battered pile contributes to the horizontal resistance. Three dimensional P-y programs, like GROUP (Ensoft) which account for multiple piles connected together can be used to determine individual pile loads for these types of foundations. To check the results of the rigid cap analysis and allowable horizontal capacities of the individual piles in the foundation groups (see Section F3.0), a P-y analysis using GROUP (Ensoft) was made at 2 sections of the wing wall at the Red River Control Structure. Axial loads obtained from the GROUP analysis shows a good correlation with results from the rigid cap method. Results of the GROUP analysis and a comparison with the results from the rigid cap analysis can be found in Exhibit Q and are summarized in Table F13. Since the COM624 and LPILE results for horizontal capacity in the Section F3.0 did not include a check on the structural capacity, the allowable horizontal loads were reduced to 31 kips for the Usual Case (Undrained Case). From a structural stand point, the larger the horizontal load on a pile the larger the bending moment. Therefore, using a larger horizontal load (increased bending moment) decreases the axial capacity of a given pile cross section when checking the steel interaction equation. Structural pile capacity checks can be found in Exhibit Q and are summarized in Table F13.

e. Structural Capacity
The structural strength of the steel pile must be checked as a compression or tension member, and in the case of laterally loaded piles, the effects of both bending moment and axial load must be investigated. HP14 x73 piles were assumed during the geotechnical investigation and feasibility cost estimates. However, a larger pile may be required once corrosion and actual loads are determined during final design. Both HP14 x73 and HP14 x89 piles were evaluated in checking the structural capacity of piles (see Exhibit Q). Procedures outlined in EM 1110-2-2906 were used to determine actual stresses in piles at several locations to verify the adequacy of the HP sections. Recently the steel industry introduced new HP16 and HP18 sections which should be investigated during final design. These larger sections may reduce the number of piles and offer a more economical foundation.

F4.1.5 Sheet Pile

Sheet pile structures will be designed in accordance with the requirements of EM 1110-2-2504. Lateral earth pressures for unbalanced soil loads were checked for both the undrained and drained parameters.

Sheet pile used for cut-off walls were neglected in uplift computations. Cut-off walls were assumed to be 10 ft long to prevent piping and scouring at the foundations.

F4.2 Control Structures

This section is applicable for the following structures:

- Red River Control Structure - LPP
- Wild Rice River Control Structure - LPP
- Red River Control Structure - FCP

The Control Structures consist of a gated section in the middle and wing walls on each size. The wing walls have stepped footings which follow up the 7:1 side slopes of the channel section and tie the concrete structure into the adjacent levee. Structural computations for these structures can be found in Exhibit Q.

F4.2.1 Load Combinations

The soil conditions along the entire project (see Section F3.0) are underlain by "soft" clay, susceptible to large settlements. Therefore, piles are proposed at all concrete structures. EM 1110-2-2100 "Stability Analysis of Concrete Structures" provides guidance for stability analysis of concrete gravity structures, but is not applicable to pile supported structures. Methods for designing piles are covered in EM 1110-2-2906, however the loads and load combinations are not defined in this manual.

Since load combinations for pile supported structures are not clearly defined by the USACE publications, Barr Engineering sent a memorandum on June 7, 2010 (see Exhibit P) to the USACE summarizing the intended Structural Design Criteria to use for Phase 3 based on Barr‟s interpretations of the USACE‟s publications. The following load combinations were proposed based on EM 1110-2-2100 Table B-1 for Gravity Dams:

- Load Conditions 2 - Normal Operating with 10-yr event w/ice (usual)
- Load Condition 3 - Infrequent Flood with 300-yr event w/ice (unusual)
- Load Condition 7 - Flood at top of structure (that is, even larger than the SPF) w/ice (extreme)

Condition 1 (construction), 4, 5, and 6 would not be checked. Conditions 4, 5, and 6 all include earthquake loading, which typically does not control in a low seismic area.

The USACE did not agree with the proposed Structural Design Criteria and in a following conference call with the USACE on June 15, 2010, three different load combinations were suggested by the USACE, based on recent projects in the Fargo/Morehead area (see Exhibit P) as follows:

- Case 1 - 100-yr event w/ice (usual)
- Case 2 - Flood at top of structure w/ice (unusual)
- Case 3 - Construction, no water loads (unusual)

On July 1, 2010 a team meeting which included representatives from the USACE, local sponsors, and the consulting team met at Barr Engineering‟s Minneapolis office to go over all aspects of the project. At the meeting, the top of the concrete was raised 2 ft above the adjacent levees which are 3 ft above the 500-yr water surface elevation. Since levees would be over-topped when a flood was 3 ft higher than the 500-yr event it was agreed that the water at the structure could not be higher than the levee height. At the meeting the structural criteria was discussed and three new load cases were added to the proposed combinations originally suggested by the USACE. These combinations were used during the Phase 3 work and are presented in Table F11.

Recent review of the ice conditions during the 2009 flood in the Fargo/Moorhead area work by ice engineering expert Andrew Tuthill of the USACE-CRREL, revealed that the static ice loads assumed in load cases 1.1 and 2.1 from Table F11 were too conservative. Mr. Tuthill provided recommended ice thicknesses and effective ice crushing strengths to be used for ice floes acting on piers (Exhibit J). Calculations for the dynamic ice force follow the methods outlined in AASHTO LRFD Bridge Design Specifications (see Exhibit P).

Before finalizing the Phase 4 structural work, Barr decided that a conference call with the USACE structural team would allow all parties to reevaluate the load cases used in the Phase 3 work and discuss the latest USACE methodology on Flood Control Projects. The conference call on January 25, 2011 was brief since the USACE structural team had not seen the preliminary drawings for the proposed levee system around Storage Area 1 used for engineering storage that would be combined with upstream staging in the Phase 4 feasibility desing. A meeting with the USACE-PDT staff (Tony Fares, Kent Hokens and Aaron Buesing) was held at Barr"s office on February 1, 2011. The meeting was used to finish up the discussion of load cases after review of the upstream levee system by the USACE team. A summary of the load cases to be used for the gated Control Structures are summarized (see February 1, 2011 e-mail in Exhibit P) and in Table F12.

During the Phase 4 meeting it was concluded that the total height of the main concrete structure would remain at elevation 927, which is nearly 2 ft above the maximum water surface elevation during the SPF (see Section F2.2.18), and that the concrete rail (typically 2"-8" on bridges) on the access deck would provide the additional 2 ft freeboard required on recent USACE flood control projects.

F4.2.2 Assumptions

1) Gated Section Geometry
    a) Concrete structure on pile foundation
    b) Top of structure assumed 15 ft wide to allow maintenance vehicle access
    c) Top of structure is at elevation 927
    d) Downstream back slope of piers section: 0.71H to 1V (see EM 1110-2-2200).
    e) Length of the gated section was based on using three 50" wide gates at the Red River Control Structure (two- 30 ft gates at the Wild Rice River Control Structure) with 8 ft piers between each gate and adjacent to the gates.
2) Wing Wall Geometry
    a) Concrete structure on pile foundation
    b) Top of structure includes 15 ft wide to allow maintenance vehicle access
    c) Top of structure is at elevation 927
    d) Stem of the wing walls varies from 8 to 10 ft. During final design the wing walls which support the access bridge maybe reduced by using a much smaller wall thickness (2 to 3 ft) combined with regularly spaced pilasters used to stiffen the wall panel and support the access bridge.

3) Gated Section Reinforcing Steel

   a) Concrete reinforcing was not designed. The quantity of reinforcement was based on the following:

      i) Footing (pile cap)

         (1) #9 @ 9" Top & Bottom Transversely

         (2) #9@ 12" Top & Bottom Longitudinally

      ii) Piers

         (1) #8 @ 12" Ea. Way around entire perimeter

         (2) #11@6" for Trunion anchors

4) Wing Wall Reinforcing Steel

   a) Concrete reinforcing was not designed. The quantity of reinforcement was based on the following:

      i) Footing (pile cap)

         (1) #9 @ 6" Top & Bottom Transversely

         (2) #9@ 6" Top & Bottom Longitudinally

      ii) Wall Stem Reinforcing

         (1) #9 @ 6" Vertical bars or #9 @12 at lower heights

         (2) #9@12" Longitudinally

5) Deck Reinforcing Steel

   a) Concrete reinforcing was not designed. The quantity of reinforcement was based on 200 lbs/cy.

6) Piles – Undrained strength pile capacities were used for Cases 1, 1.1, 2, 2.1, and 3. Drained strength pile capacities were used for Case 4 only.

7) Sheet Pile Cut-offs – The sheet pile cut-off walls were assumed 10 ft long installed along both the upstream and downstream faces of the gated sections. For the wing walls it was assumed a single upstream cut-off wall would be used.

F4.3 Aqueduct Structures

This section is applicable for the following structures:

- Sheyenne River Aqueduct Crossing
- Maple River Aqueduct Crossing

Aqueduct structures consist of two level reinforced concrete frame structures. The top level consists of a "U" shape reinforced concrete channel with a 50-ft clear width. Channel profile dimensions are determined to match the existing tributary channel cross section. The reinforced concrete channel is supported by reinforced concrete walls below spaced at 30-ft on-center. The total width of the Diversion Channel is 250-ft. The approach walls are curved on the upstream side of the Diversion Channel and extend to meet the final grade elevation at 7:1 side slopes of the channel. Wing walls on the downstream side of the Diversion Channel extend straight parallel to the channel. The soil conditions along the entire project are underlain by "soft" clay, susceptible to large settlements. Therefore, aqueducts structures, approach walls and wing walls are supported by H-piles. See section F3.0 for more information about geotechnical considerations.

During the meeting held at Barr‟s office on February 1, 2011 with USACE-PDT staff (Tony Fares, Kent Hokens and Aaron Buesing), it was concluded that the elevation at the top of the walls of the aqueduct structures should be the same as the top elevation of the surrounding levee.

See Sections F2.2.6 and F2.2.8 for hydraulic details of the Sheyenne and Maple aqueduct crossings respectively. More detailed information about the design of these structures, including loads and loading conditions, can be found in Exhibit R.

F4.3.1 Load Combinations

A meeting with the USACE-PDT staff (Tony Fares, Kent Hokens and Aaron Buesing) was held at Barr‟s office on February 1, 2011. Structural design criteria, load cases and ice loading were discussed during the meeting for various structures including the aqueduct structures. The load cases used for the aqueduct structures are summarized (see February 1, 2011 e-mail in Exhibit P) in Table F14.

Based on EM1110-2-2104 "Structural Design for Reinforced-Concrete Hydraulic Structures", the following load combinations are assumed for the concrete design. Load combinations are increase by the hydraulic factor Hf = 1.3 except for members in direct tension.

Factored Load Combinations are listed below where:

D: Dead Loads

L: Live Loads

F1: Hydrostatic Load (Water level to the top of the structure on the Diversion Channel side only)

F2: Hydrostatic Load (Water level to the top of the structure inside the Tributary Channel only)

F3: Hydrostatic Load (Water level at the 100-yr event + ice on both the Tributary Channel and Diversion Channel)

F4: Hydrostatic Load (Water level to the top of the Low Flow Channel walls + ice loading applied at the top of the Low Flow Channel walls)

H: Earth Load

LOAD COMBINATION #24 1.3(1.4D+1.7L)
LOAD COMBINATION #27 1.3(1.4D+1.7L+1.7F1)
LOAD COMBINATION #32 1.3(1.4D+1.7L+1.7F2)
LOAD COMBINATION #33 1.3(1.4D+1.7L+1.7F3)
LOAD COMBINATION #34 1.3(1.4D+1.7L+1.7F4)
LOAD COMBINATION #37 1.3(1.4D+1.7L+1.7F1&F2)
LOAD COMBINATION #42 1.3(1.4D+1.7L+1.7H)
LOAD COMBINATION #45 1.3(1.4D+1.7L+1.7F1+1.7H)
LOAD COMBINATION #47 1.3(1.4D+1.7L+1.7F2+1.7H)
LOAD COMBINATION #50 1.3(1.4D+1.7L+1.7F3+1.7H)
LOAD COMBINATION #53 1.3(1.4D+1.7L+1.7F4+1.7H)
LOAD COMBINATION #55 1.3(1.4D+1.7L+1.7 F1&F2+1.7H)

F4.3.2 Assumptions

Finite Element Analysis (FEA) computer software is used to analyze the portion of the aqueduct structures. One FEA model is generated using STAAD Pro V8i structural analysis program (STAAD) since both Sheyenne and Maple structures are very similar as far as the geometry. The model included the maintenance access bridge, tributary channel slab and wall, diversion channel mat foundation and walls supporting the tributary channel. 90-ft section of the structure is modeled. Diversion channel mat foundation is assumed to be supported on H-piles. Walls supporting the Tributary channel are extended all the way to the top of the Tributary channel wall elevation on both upstream and downstream sides to act as pilasters and provide strength to the Tributary channel walls. These walls/pilasters will also act as impact resisting elements to protect the Tributary channel walls for any ice or debris during high flows. These walls/pilasters are also acting as beams and reducing the stresses on the Tributary channel walls. Concrete walls and slabs are modeled by using quadrilateral plate elements. Different thicknesses are assigned to the plates.

STAAD uses the stiffness matrix method for the beam elements and Mindlin-Reissner"s thick plate theory and finite elements for plates and shell elements. STAAD uses the stiffness matrix to distribute the loads between beams, columns and plates.

Models created in STAAD were used to generate the maximum envelope shear (V_u), moment (M_u), and axial (P_u) forces on the structural elements. Shear (ϕV_n) and moment (ϕM_n) capacities of the beams and slabs are calculated based on ACI 318 using Microsoft Excel and MathCAD spreadsheets.

Lateral loads such as such as wind and seismic forces were not included in the analysis as indicated above.

1) Geometry

   a) Aqueduct structure, approach and wing walls are supported by concrete foundations on H-piles.

   b) Bridge is assumed 15 ft wide to allow maintenance vehicle access

   c) Top of the concrete walls is assumed to be same at the same elevations as the surrounding levees.

   d) Tributary channel width is assumed to be 50 ft.

   e) Diversion Channel width is assumed to be 250 ft with 6 bays at 30-ft on center and two bays at both ends are 35 ft.

   f) Approach walls are assumed to be curved with radius of 205 ft.

   g) Wing walls are assumed to be straight and extending 354 ft beyond the downstream face of the Diversion channel.

   h) Stem of the approach and wing walls varies from 14 to 33 ft with 4 ft thick.

2) Reinforcing Steel

   a) Concrete reinforcing was designed preliminarily. The quantity of reinforcement was based on the following:

      i. Footings (pile cap)

         1. #9 @ 6" Top & Bottom Transversely

         2. #9 @ 6" Top & Bottom Longitudinally

      ii. Piers

         1. #9 @ 6" Ea. Way & Ea. Face

      iii. Walls

         1. #9 @ 6" Ea. Way & Ea. Face or #9 @ 12" at lower heights.

      iv. Elevated Slabs / Deck

         1. #9 @ 6" Top & Bottom Transversely

         2. #9 @ 6" Top & Bottom Longitudinally

3) Piles – Undrained strength pile capacities were used for Cases 1, 2, 3, 4, and 5. Drained strength pile capacities were used for Case 6 only.

4) Sheet Pile Cut-offs – The sheet pile cut-off walls were assumed 10 ft long installed along both the upstream and downstream faces of the aqueduct mat foundation. For the approach and wing walls it was assumed a single upstream cut-off wall would be used.

F4.4 Sheetpile Weirs/ Stepped Spillways/Ogee Spillways

Major assumptions for the sheetpile weirs and stepped spillways are discussed in this section. More detailed information about the design of these structures can be found in Exhibit S and T.

F4.4.1 Sheetpile Weirs

This section is applicable for the following structures:

- Sheyenne River spillway weir
- Maple River spillway weir

The weirs were sized based on the following assumptions:

1) Geometry was provided by hydraulics.
2) Sheet piling with a maximum exposed height of 10 ft was checked using the USACE program CWALSHT. Geotechnical soils information used as input was based on test data for the Brenna Clay:
   a) Saturated Unit Wt = 104 pcf
      i) Undrained Strength soil parameters
         (1) C = 800 psf
         (2) ϕ = 0 deg
      ii) Drained Strength soil parameters
         (1) C „= 267 psf
         (2) ϕ'= 14.2 deg (for effective normal stress between 1,000 to 2,000 psf))

   The undrained case does not control due to the soil properties. For the drained condition, the results showed an embedment of about 19.5‶ was required to achieve stability (FS = 1.0) for a 10 ft wall. The embedment length was increased 30% to 26 ft to simulate a FS on the passive side.
3) The remaining wall heights were assumed to have a similar embedment to exposed wall height ratio (26 / 10 = 2.6), and a ratio of 2.5 was used for quantity calculations.
4) No hydrostatic head differentials were assumed on either side of the weir. Weep holes and crushed aggregate backfill are methods to help balance hydrostatic loads.

F4.4.2 Stepped Spillways

This section is applicable for the following structures:

- Lower Rush River Drop Spillway Structure
- Rush River Drop Spillway Structure

These two structures are very similar, so were analyzed in tandem. Both structures consist of a transition from the natural channel into retaining walls on each side of the river. These retaining walls will confine the flow of the river and will have top of wall elevation 4.1 ft and 7.4 ft above the 500-yr flood elevation for the Lower Rush River and Rush River, respectively. Once these walls intersect the LPP Diversion Channel spoil piles, the top of wall is dropped to follow the slope of the diversion side slopes.

The soil conditions along the entire project (see Section F3.0) are underlain by “soft” clay, susceptible to large settlements. Therefore, piles will be used at the concrete wall and concrete step structures.

The Load Cases used for the Rush & Lower Rush River Structures are:

- The primary load case for retaining walls is generally the case where the retained soil pushes on the wall with nothing on the opposite side. In this case, soil on one side with no water on the other side. Under load case 1, the water table in the soil was assumed at 5 ft below top of wall to produce an increased horizontal force in the retained soil. This is considered a usual load case, and uses undrained pile values.
- Since the fish passage has no retained soil behind it, this load case looked at 22 ft of water on the river side of the wall with no retained soil on the back side of the wall. Even though unrealistic to impossible, the calculations for this load case assume this to be a usual load case, using undrained pile values.
- The third load case is the same as load case 1 with the exception of no water table in the soil. This load case uses the drained soil pile values, so it was assumed this load case would occur in a dry river and thus be considered unusual.

For the retaining walls, the largest retained soil height for either river is 22 ft above final grade. Wall thicknesses, footing size, and piling design were computed to satisfy 22 ft retained height. Conservatively, this same concrete thicknesses and pile requirements were continued throughout the design. Footings for walls were set at 5 ft thick to act as a pile cap. Foundation bearing pressures were computed and piles were placed to withstand the foundation pressures. Piles were computed to require battering to withstand the horizontal sliding force of the wall. Bottom of wall footings were placed 8 ft below grade for frost protection.

The step weight was also supported by piles. Slabs were provided for 20 ft leading into the steps and 50 ft leaving the steps. No piles were assumed necessary for these steps.

Reinforcing bar quantities were roughly estimated considering 12” bar spacing in both faces each way. Vertical bars in wall sections were assumed to weigh 5.3 pounds per ft of bar length in tall wall sections, 4 pounds per ft in medium height wall sections, and 3 pounds per ft in short wall sections. Horizontal bars were estimated at 3 pounds per ft. Footing bars were estimated at 4 pounds per ft each direction. The slabs on grade were estimated at 3 pounds per ft in each direction. The steps were computed at 4 pounds per ft based on the concrete perimeter distance.

This calculation assumes that concrete would be 4000 psi concrete compressive strength with 60,000 psi reinforcing bar yield strength.

F4.4.3 Ogee Spillways

This section is applicable for the following structures:

- Diversion Inlet Structure
- Outlet Structure

Geometry for the spillways was based on the hydraulic requirements and was provided by the H&H team. Both structures consist of an ogee weir and a downstream apron. The inlet structure has a 17 ft drop, while the Outlet Structure has an approximate drop of 20 ft. Retaining walls are used as wing walls to accommodate the levee heights and channel slopes on each side of the spillways.

The soil conditions along the entire project (see Section F3.0), are underlain by “soft” clay, susceptible to large settlements. Therefore, piles will be used at the concrete wall and concrete step structures.

The Load Cases used for the Ogee Spillway include:

- Normal Water (Ogee crest level)
- Normal Water + Ice (Static pressure: 2 ft tick @ 5000 psf = 10,000 plf)
- 100-yr event
- 500-yr event

The Load Cases used for the Ogee Apron include:

- Normal Water (Ogee crest level)
- 100-yr event
- 500-yr event

Reinforcing bar quantities for the Ogee Spillway and Apron were roughly estimated considering #9 @12” bar spacing transversely and #9 @ 12” longitudinally in both faces of foundation and along the exposed face of the ogee spillway.

For the retaining walls, it was assumed that water pressure would be equal on both sides of the walls. Therefore, the greatest lateral loads will occur when using moist soil earth pressures with no water. Standard load charts for retaining walls published by the Minnesota Department of Transportation were used for sizing the foundation requirements, concrete reinforcement and stem thickness. A constant stem thickness was used for the total height of each section. Bottom of wall footings were placed 5 ft minimum below grade for frost protection.

This calculation assumes that concrete would be 4000 psi concrete compressive strength with 60,000 psi reinforcing bar yield strength.

F5.0 CIVIL DESIGN AT HYDRAULIC STRUCTURES

The purpose of this section of Appendix F is to provide general information related to the siting and civil site design of the hydraulic structures associated with this project. After providing some general background data related to the sources of the data used in the civil design the discussion will address each major structure. Structures addressed will include:

- The Red River and Wild Rice River Control Structures
- The Sheyenne and Maple River Aqueducts
- The Lower Rush and Rush River drop structures
- The connecting channel weir, the primary Inlet Structure to the Diversion and Outlet Structure to the Red River of the North
- Additional structures related to Wolverton Creek, Drain 14, and Storage Area 1
- Storage Area 1

Discussion about each structure will be generally broken down into two main sections; the first covering structure Micro-Siting which will address issues pertaining to why the feature was sited at the specific location chosen within the overall project setting, and second Civil Site Design which will cover the following main points:

- General site dimensions and slopes used in design
- Setbacks if applicable
- Site access
- Site power
- Need for maintenance facilities
- Supervisory Control and Data Acquisition (SCADA) monitoring points
- Ice and debris control
- Need for erosion control measures such as rip rap or rock grade control structures

F5.1 Existing Conditions at Hydraulic Structures

Existing topographic information at each of the major hydraulic structure is based on Light Detection and Ranging (LIDAR) data obtained from the Red River Basin Mapping Initiative of the International Water Institute Center for Flood Damage and Natural Resources. Existing topographic information in the form of 1-meter digital elevation model (DEM) data was used to create a 3-dimensional surface model with Bentley Microstation InRoads, version V8i. Digital drawings reference the horizontal coordinate datum State Plane, ND South NAD 83, US Survey Feet and vertical coordinate datum North American Vertical Datum of 1988 (NAVD 1988), US Survey Feet. The LIDAR data used in this project has a vertical accuracy spec of +/- 15 cm.

Bathymetric data is based on data obtained for the hydraulics model. Please refer to Table B1 of Section B4.0 of Appendix B – Hydraulics for additional information.

Existing utility information is based on digital Microstation data provided by Kadrmas, Lee and Jackson in 2009. Existing parcel information for Cass County is based on May

2010 GIS data available on the Cass County Website and as provided in 2009 by Kadrmas Lee and Jackson. Existing parcel information for Clay County is based on information provided in 2009 by Kadrmas Lee and Jackson.

Parcels in the vicinity of hydraulic structures were checked for deed restrictions, which could limit or restrict the construction of project features on the property. Deed-restricted parcel information for Cass County, ND is based on GIS data provided by Cass County in June of 2010 as well as parcel map information available on the Cass County website at that time. Deed-restricted parcel information for Clay County, MN was not directly available. Information available for 20 Clay County properties located in the immediate vicinity of hydraulic structures was verified using the search tool Tapestry available on the Clay County website during July 2010. The 20 parcels at hydraulic structures were verified as privately-held properties. It is reasoned that deed-restricted properties would be listed as publicly-held. Since none of the 20 were listed as publicly held, it is assumed that none are deed-restricted properties.

Since this is a flood prone region and property buyouts related to flooding take time to process it will be important for future efforts on this project to include steps to re-verify if deed restrictions exist on any key parcels. Local authorities should be careful not to purchase flood damaged properties needed for this project using FEMA dollars in order to avoid deed restrictions being placed on key parcels.

F5.2 Civil Design at Gated Control Structures (Red River Control Structure, Wild Rice River Control Structure)

A feasibility-level civil site design is presented for each of the major hydraulic structures to exhibit the overall grading, footprint, functionality and context of each structure and for the estimation of construction quantities for cost estimates. This section summarizes the micro-siting and civil site design methodology used to develop feasibility grading and site plan for each of the similar-schemed gated control structures (two structures for the LPP, and one structure for the FCP). The feasibility designs for these structures all have gate bays, tie into adjacent levees with wing walls and are constructed in the dry. For additional information on these structures refer to Appendix F Drawings S-401 through S-404, S-407 through S-410, and S-413 through S-416.

A similar general strategy is used for all major hydraulic structures, focusing on coordinating locations with the proposed diversion corridor and tie-back levees or spoil banks, minimizing the construction footprint area required and siting them to interface with existing waterways.

F5.2.1 Hydraulic Structure Micro-Siting (Red River Control Structure, Wild Rice River Control Structure)

The feasibility micro-siting methodology of the Red River Control Structure (LPP & FCP) and Wild Rice River Control Structure (LPP) is described below. The siting of the individual structures sets the context for the civil site design at each structure.

It is assumed that the Wild Rice River Control Structure and Red River Control Structure will be constructed off of the existing river channel, in dry conditions. Micro-siting of each hydraulic structure is provided:

- The FCP Red River Control Structure is located east of the existing Red River in Minnesota;
- The LPP Red River Control Structure is located east of the existing Red River in Minnesota;
- The LPP Wild Rice River Control Structure is located east of the existing Wild Rice River in North Dakota;

For local project funding reasons it is important that the Red River Control Structure remain in Minnesota and not be moved over to North Dakota in later project phases. Rules related to how the State of Minnesota can participate in this project financially are tied to the location of constructed features. Therefore, structure siting of the Red River Closure structure is tied to things other than purely technical criteria.

Constructed channels are required for the Wild Rice River Control Structure and Red River Control Structure to redirect river flows from the existing river, to the hydraulic control structure and back into the existing river. As a design guideline, centerline radii were chosen that are at minimum three times the water surface top width in the constructed channel. The channel realignments balance large centerline radii while attempting to minimize the overall footprint of the site work.

A buffer of 300 ft between the proposed structure and the existing river banks was used in siting the gated portion of each structure. The structures were sited to avoid having existing river banks within this buffer. This area represents a conceptual construction work area, plus area required to excavate 10H:1V slopes down to the structure construction area and to allow room for the construction of temporary earthen flood protection levees around the work site, if necessary. The assumed offset is to facilitate constructability and must be verified during final design when detailed local information (geotechnical, groundwater, etc.) and temporary construction facilities are available.

A permanent easement of 30 ft, offset from the extents of grading work, is assumed at each of the hydraulic structures. A temporary construction easement of 15 ft, offset from the edge of permanent easement is assumed at each hydraulic structure. This right-of-way is consistent with what is assumed for the channel portions of the project.

Efforts were made to minimize the impacts at each hydraulic structure. The hydraulic structures were sited to avoid having grading work or constructed features on deed-restricted properties, minimize to the extent possible impacts to the estimated Ordinary-High-Water-Mark (OHWM) at each location and minimize the length of realigned river channels to the extent possible.

Future micro-siting efforts should address the following in greater detail:

- The sites of all structures that incorporate fish passage may have to be altered slightly to accommodate the final configuration of those features.
- Site locations may need to be modified to accommodate the incorporation of recreational features in final design
- Site locations may need to be altered pending the gathering of design data related to geotechnical parameters, and presence of local groundwater features that may impact structure stability.
- Physical modeling of each of these structures may dictate a better location or orientation with respect to channel flow is needed to obtain the desired performance.

F5.2.2 Hydraulic Structure Civil Site Design (Red River Control Structure, Wild Rice River Control Structure)

The feasibility civil site design methodology of the Red River Control Structure (LPP & FCP) and Wild Rice River Control Structure (LPP) is described below. These civil site designs are used to estimate quantities for feasibility cost estimates.

Access roadways are located to provide maintenance (not public) access to each hydraulic structure. Access roads are assumed to have gravel surfacing and be 12 ft wide for servicing minimal amounts of traffic during observation or maintenance activities, during both flooding and non-flooding times. Fences and gates are assumed for limiting or prohibiting public access. Access to each hydraulic structure is provided:

- Access for the FCP Red River Control Structure is from both the ND and MN sides on top of the protection levee. From Minnesota access will come from U.S. Highway 75 just south of Clay County 67. From North Dakota access will be from a local residential road called Forest River Road.
- Access for the LPP Red River Control Structure is from both the ND and MN sides on top of the protection levee. From Minnesota the Control Structure will be accessed from Clay County Road 59. From North Dakota the Control Structure will be accessed from South University Drive (Cass County Road 81).
- Access for the LPP Wild Rice River Control Structure is from the west side of the structure coming from the north off of Cass County Road 16 along the top of the east Storage Area 1 embankment.

A maintenance building and small parking area is located near each structure for storage of materials, equipment and usage during flooding events. Power is shown being brought to the site from the nearest power lines. Water and sanitary sewer service are not included.

Remote monitoring thru a Supervisory Control and Data Acquisition (SCADA) system is assumed. Data is gathered at strategic locations and delivered digitally back to a central monitoring station, where the information system is managed. Key monitoring points include:

- At FCP Red River Control Structure: monitoring of stage and head across the closure structure, monitoring of head across the ice/debris control measure, monitoring of head across the inlet weir to the diversion;
- At LPP Red River Control Structure: monitoring of stage and head across the closure structure;
- At LPP Wild Rice River Control Structure: monitoring of stage and head across the closure structure, monitoring of head across the weir at the connecting channel;

For both the FCP and the LPP there is a tie back levee assumed in this area that will connect the closure structure to high ground to the west and east of the facility. The tie back levee will be designed and constructed to strict compaction and stability requirements since it is on the leading edge of the project. Grading for the tie back levees are shown with 4H:1V slopes and rise to elevation 927. For the FCP slopes of excess spoil not needed to construct the levee are shown at 10H:1V. No-spoil areas are shown on the plan near each existing waterway due to geotechnical stability issues.

Since the Phase 4 concept now includes staging on this end of the project there is no excess spoil. The connecting channel between the Red River of the North and the Wild Rice River will be constructed as a borrow area for the staging levee. It will only be as wide and deep as needed to supply material for the construction of that levee. Grading for river channel excavations is shown with 7H:1V slopes. Connecting channel excavations at each structure are shown as:

- At FCP Red River Control Structure, 150 ft bottom width, 10H:1V side slopes rising up to a 50 ft wide bench on each side, then rising at a 7H:1V side slope up to the top of the diversion channel.
- At LPP Red River Control Structure, the connecting channel has a bottom width of 250 ft with side slopes of 7H:1V. The depth of the channel below existing grade shown near the structures is dependent on the amount of material needed to construct the protective levees and is shown at ~10 ft. Note that the protective levee rises up an additional 12 to 14 ft above existing grade to facilitate staging.
- At LPP Wild Rice River Control Structure, the connecting channel has a bottom width of 250 ft with side slopes of 7H:1V. Again the depth of the channel below existing grade shown near the structures in dependent on the amount of material needed to construct the protective levees and is shown at 10 ft. Note that the protective levee rises up an additional 12 to 14 ft above existing grade to facilitate staging.

Low-flow channels are included where water generated by normal, seasonal precipitation and runoff events (non-flooding events) must be drained to a waterway. Key locations for low-flow channels include:

- At FCP Red River Control Structure the low flow channel will be sloped to drain back to the Red River of the North from the connecting channel weir located near I-29.

- At LPP Red River Control Structure the low flow channel will be sloped to drain back to the Red River of the North from the connecting channel weir located near I-29.
- At LPP Wild Rice River Control Structure the low flow channel will be sloped to drain back to the Wild Rice River from the connecting channel weir located near I-29.

Areas requiring permanent riprap or other erosion control measures are shown for each hydraulic structure. The extents shown are feasibility level, based on Phase 3 2-dimensional velocity modeling work and must be refined in final design. Key areas that must be permanently protection from erosion by high flows and/or turbulence include:

- Bank slopes where realigned channels enter or exit existing river channels;
- Banks and channel bottom in the vicinity of each gated Control Structure;
- Banks and channel bottom in the vicinity of all weirs;
- Locations where low-flow channels empty into rivers;
- Channel bottom protection is required upstream of the realigned portion of the Wild Rice River;
- Other areas where high flow velocities or turbulence are anticipated;

Ice and Debris Control Measures are located in areas with the strategy to direct ice away from the diversion channel, keeping the diversion channel as free of ice as possible. These measures are located to make them somewhat accessible during flooding events (the level of accessibility depends on the final measures implemented). Ice and Debris Control Measures are shown for the FCP for the Red River Control Structure. Because the concept for the LPP now includes staging upstream of the protective levee, concentrated flow in a connecting channel in the area between the Red River Control Structure and the inlet to the diversion will no longer occur; instead a temporary pool is staged during flood events. Because of this, ice and debris control measures are no longer needed in these areas.

Fish passage systems are included with each of these Control Structures in both the FCP and LPP plans. They are located downstream of one of the Control Structure wing walls and provide connectivity up the proposed river channel, from downstream (protected side) of the Control Structure to upstream (unprotected side) of the structure via a system of operable gates. A system of riffles and pools, protected from erosion by rock and boulders is incorporated. Fish passage systems are provided at Control Structures:

- At FCP Red River Control Structure
- At LPP Red River Control Structure
- At LPP Wild Rice River Control Structure

Topsoil stripping, replacing and site restoration is assumed to be required for all areas permanently acquired by the project as well as permanent easement areas.

For hydraulic structure feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

Future hydraulic structure civil site design efforts should address the following in greater detail:

- Due to the critical nature of this project and the unstable soil conditions present in the region the design should include a full detailed review of the protective levee to see if shallower side slopes and a wider top section are warranted.
- Filling and full abandonment of all unused river channels is recommended to reduce risk of levee failure in these areas.
- Detailed design must address the issue of scour at hydraulic structures. Phase 4 erosion protection is based on Phase 3 2-dimensional velocity modeling and is conceptual.
- A detailed review of ground water needs to be included in final design to ensure that slope stability is not compromised due to local groundwater flow patterns.
- Due to the critical nature of these Control Structures access roads may warrant full pavement sections since floods in this region normally occur during the spring when gravel roads can be difficult to traverse if not maintained well.
- Site designs may need to be heavily modified depending upon the nature of recreational features incorporated in final design.
- Projects of this scale will often include public art or other modifications to enhance the visual appeal of what is built. Civil site works may need to be modified to accommodate these features.
- Access to the full length of the fish passages needs to be considered.
- Maintenance access to the areas upstream and downstream of all Control Structures and on both sides of the rivers should be provided in final design.
- Detailed design must address issues and requirements related to the Levee System Evaluation for the National Flood Insurance Program. Detailed design should address operation and maintenance concerns associated with regular evaluation and certification under this program. For example, developing a comprehensive strategy related to instrumentation, monitoring, inspections, etc. and incorporating project features to facilitate system evaluation.

F5.3 Civil Design at Aqueduct Structures (Sheyenne River Structure, Maple River Structure)

A feasibility-level civil site design is presented for each hydraulic structure to exhibit the overall grading, footprint, functionality and context of each structure and for the estimation of construction quantities for cost estimates. This section summarizes the micro-siting and civil site design methodology to develop feasibility grading and site plan for each of the similar-schemed aqueduct hydraulic structures (two structures for the LPP). The feasibility designs for these structures all have an aqueduct structure, tie into adjacent levees with wing walls and are constructed in the dry. For additional information on these structures refer to Appendix F Drawings S-424 through S-429 and S-431 through S-436.

F5.3.1 Hydraulic Structure Micro-Siting (Sheyenne River Structure, Maple River Structure)

The feasibility micro-siting methodology of the Sheyenne River Hydraulic Structure (LPP) and Maple River Hydraulic Structure (LPP) is described below. The siting of the individual structures sets the context for the civil site design at each structure.

It is assumed that the Sheyenne River Hydraulic Structure and Maple River Hydraulic Structure will be constructed off of the existing river channel, in dry conditions. Micro-siting of each hydraulic structure is provided:

- The LPP Sheyenne River Hydraulic Structure is located east of the existing Sheyenne River;
- The LPP Maple River Hydraulic Structure is located south of the existing Maple River;

Constructed channels are required for the Sheyenne Hydraulic Structure and Maple Hydraulic Structure to redirect river flows from the existing river, to the hydraulic control structure and back into the existing river. As a design guideline, centerline radii were chosen that are at minimum three times the water surface top width in the constructed channel. The channel realignments balance large centerline radii while attempting to minimize the overall footprint of the site work.

A minimum buffer of 300 ft between the main aqueduct and the existing river was used in siting the location of each aqueduct. The structures were sited to avoid having existing river banks within this buffer. This area represents a conceptual construction work area, plus area required to excavate 10H:1V slopes down to the structure construction area and construct temporary earthen levees around the work site, if necessary. The assumed offset is to facilitate constructability and must be verified during final design when detailed local information (geotechnical, groundwater, etc.) and temporary construction facilities are available.

A permanent easement of 30 ft, offset from the extents of grading work, is assumed at each of the hydraulic structures. A temporary construction easement of 15 ft, offset from the edge of permanent easement is assumed at each hydraulic structure. This right-of-way is consistent with what is assumed for the channel portions of the project.

When the Sheyenne and Maple Rivers have reach a water stage over the 2-year event elevation, a portion of the flow is directed to a spillway which directs this flow to the diversion channel. The spillway entrance is located upstream of the aqueduct entrance, and the control weir is at an elevation which only allows flows greater than the 2-yr event pass through the spillway. The Sheyenne River spillway has a series of four sheet-pile weirs and a riprap sill where the spillway enters the main diversion channel.

The Maple River spillway consists of a series of three sheet-pile weirs and a riprap sill. Water flowing over each weir will be directed into a stilling pool that will be 4 ft deep when in use. The total fall over each weir is approximately 6 ft. The uppermost weir will be constructed to maintain water levels in the main river channel. However each of the subsequent weirs will be fitted with a drain that allows the stilling pool to empty slowly after each use so there will be no permanent pool under each weir.

The Sheyenne River spillway consists of a series of four sheet-pile weirs and a riprap sill. Water flowing over each weir will be directed into a stilling pool that will be 4 ft deep when in use. The total fall over each weir is approximately 6 ft. The uppermost weir will be constructed to maintain water levels in the main river channel. However each of the subsequent weirs will be fitted with a drain that allows the stilling pool to empty slowly after each use so there will be no permanent pool under each weir.

For both the Maple and Sheyenne spillways concrete ogee spillways were considered. Sheet pile weirs are currently shown due to their lower cost.

Efforts were made to minimize the impacts at each hydraulic structure. The hydraulic structures were sited to avoid having grading work or constructed features on deed-restricted properties, minimize to the extent possible impacts to the estimated Ordinary-High-Water-Mark (OHWM) at each location and minimize the length of realigned river channels to the extent possible.

Future micro-siting efforts should address the following in greater detail:

- Detailed design must address the issue of scour at hydraulic structures. Phase 4 erosion protection is based on Phase 3 2D velocity modeling and is conceptual.
- The exact location and orientation of the channel leading to the aqueduct off of the existing channel should be carefully designed based on geomophological studies currently underway.
- Flow characteristics and bed load analysis may lead to a concern over sediment deposition in the long channel leading up to the Sheyenne spillway weirs.
- Local groundwater flow patterns should be understood before structure locations are finalized so that issues related to saturated ground conditions can be fully accounted for during final design.
- Additional hydraulic analysis, possibly 2D flow modeling, of the spillways should be done to better understand the impact of introducing spillway flow into the diversion on coincidental flood flows in the diversion. It is possible that reorientation of the spillway will be needed to better align flow from the spillway with flows from the diversion to reduce head loss in the diversion.

F5.3.2 Hydraulic Structure Civil Site Design (Sheyenne River Structure, Maple River Structure)

The feasibility civil site design methodology of the Sheyenne River Hydraulic Structure (LPP) and Maple River Hydraulic Structure (LPP) is described below. These civil site designs are used to estimate quantities for feasibility cost estimates.

Access roadways are located to provide maintenance (not public) access to each hydraulic structure. Access roads are assumed to have gravel surfacing and be 12 ft wide for servicing minimal amounts of traffic during observation or maintenance activities, during both flooding and non-flooding times. Fences and gates are assumed for limiting or prohibiting public access. Access to each hydraulic structure is provided:

- Access for the LPP Sheyenne River Hydraulic Structure is from the west side of the existing Sheyenne River on top of the protection levee, from 100th Ave South.
- Access for the LPP Maple River Hydraulic Structure is on the east side of the diversion channel on top of the protection levee, from 33rd Street SE.

A maintenance building and small parking area is located near each structure for storage of materials, equipment and usage during flooding events. Water and sanitary sewer service are not included. Power is shown routed to the structures from the nearest power lines.

Remote monitoring thru a SCADA system is assumed for both aqueducts. Data is gathered at strategic locations and delivered digitally back to a central monitoring station, where the information system is managed. Key monitoring points include:

- Monitoring of stage and head in the diversion across the aqueduct structures
- Monitoring of stage and head in the aqueduct channel over the diversion
- Monitoring of head across the ice/debris control measures at the connecting channel, and
- Monitoring of stage and head across the uppermost spillway control weir;

Grading for embankment areas is shown with 7H:1V slopes. No-spoil areas are shown on the plan near each existing waterway. Grading for river channel excavations is shown with 5H:1V slopes. Diversion channel excavations at each structure are shown as:

- Sheyenne River Structure
    - Upstream – 250' bottom, 7:1 slope to existing grade, 50' bench, 12' high levee with a 105' top width
    - Downstream – 250' bottom, 7:1 slope to 8' high, 40' bench, 7:1 slope to existing grade, 50' bench, 15' high levee, 409' wide
- Maple River Structure
    - Upstream – 250' bottom, 7:1 slope to 8' high, 25' bench, 7:1 slope to EG, 50' bench, 15' high levee, 241' wide

- Downstream – 250' bottom, 7:1 slope to 8' high, 25' bench, 7:1 slope to existing grade, 50' bench, 15' high levee, 260' wide

Low-flow channels are included in the new excavated channels leading to both aqueducts to feed low flows, especially during winter months, into the specially designed low flow channels on the aqueducts.

Areas requiring permanent riprap or other erosion control measures are shown for each hydraulic structure. The extents shown are conceptual, based on Phase 3 2-dimensional velocity modeling work and must be refined in final design. Key areas that must be permanently protected from erosion by high flows and/or turbulence include:

- Bank slopes where realigned channels enter or exit existing river channels;
- Banks and channel bottom in the vicinity of the entrance and exist of the aqueduct structure;
- Banks and channel bottom in the vicinity of all weirs;
- Diversion channel bottom where Sheyenne and Maple spillways enter the channel
- Areas immediately upstream of where the spillway flows are diverted off the normal new channel where head cutting could occur from rapidly lowering water levels.
- Other areas where high flow velocities or turbulence are anticipated;

Ice and Debris Control Measures are located in areas with the strategy to direct ice away from the spillways that direct flood flows into the diversion channel, keeping the diversion channel as free of ice as possible. These measures are located to make them somewhat accessible during flooding events (the level of accessibility depends on the final measures implemented).

Topsoil stripping, replacing and site restoration is assumed to be required for all areas permanently acquired by the project as well as permanent easement areas.

For hydraulic structure feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

Future hydraulic structure civil site design efforts should address the following in greater detail:

- Detailed design must address the issue of scour at hydraulic structures. Phase 4 erosion protection is based on Phase 3 2-dimensional velocity modeling and is conceptual.

- The exact location of rock head cutting protection upstream of each spillway.
- A detailed review of ground water needs to be included in final design to ensure that slope stability is not compromised due to local groundwater flow patterns.
- Site designs may need to be heavily modified depending upon the nature of recreational features incorporated in final design.
- Projects of this scale will often include public art or other modifications to enhance the visual appeal of what is built. Civil site works may need to be modified to accommodate these features.
- Maintenance access to the areas upstream and downstream of all control structures and on both sides of the river and spillway should be provided in final design.
- The aqueducts are unique enough that access for public viewing may be warranted meaning fully paved roads and a small public parking area may be needed.

F5.4 Civil Design at Tributary Drop Structures (Lower Rush River Structure, Rush River Structure)

A feasibility-level civil site design is presented for each hydraulic structure to exhibit the overall grading, footprint, functionality and context of each structure and for the estimation of construction quantities for cost estimates. This section summarizes the micro-siting and civil site design methodology to develop feasibility grading and site plan for each of the similar-schemed drop structure and fish passage structures (two structures for the LPP). The feasibility designs for these structures all have drop structures, tie into adjacent levees with wing walls and are constructed in the dry. For additional information on these structures refer to Appendix F Drawings S-437 through S-444.

F5.4.1 Hydraulic Structure Micro-Siting (Lower Rush River Structure, Rush River Structure)

The feasibility micro-siting methodology of the Lower Rush and Rush River Drop Structures (LPP) is described below. The siting of the individual structures sets the context for the civil site design at each structure.

It is assumed that the Lower Rush River Drop Structure and Rush River Drop Structure will be constructed off of the existing river channel, in dry conditions. Micro-siting of each hydraulic structure is provided:

- The LPP Lower Rush River Drop Structure is located south and east of the existing Lower Rush River;
- The LPP Rush River Drop Structure is located north of the existing Rush River;

Constructed structure entrance channels are required for the Lower Rush River Drop Structure and Rush River Drop Structure to redirect river flows from the existing river, to the drop structure. As a design guideline, centerline radii were chosen that are at minimum three times the water surface top width in the constructed channel. The channel realignments balance large centerline radii while attempting to minimize the overall footprint of the site work.

A buffer of 300 ft between the main drop structure and the existing river was used in siting the structure. This area represents a conceptual construction work area, plus area required to excavate 10H:1V slopes down to the structure construction area and construct temporary earthen levees around the main drop structure work site, if necessary. The assumed offset is to facilitate constructability and must be verified during final design when detailed local information (geotechnical, groundwater, etc.) and temporary construction facilities are available.

A permanent easement of 30 ft, offset from the extents of grading work, is assumed at each of the drop structures. A temporary construction easement of 15 ft, offset from the edge of permanent easement is assumed at each hydraulic structure. This right-of-way is consistent with what is assumed for the channel portions of the project.

A fish passage is included with each drop structure to provide connectivity between the constructed meandering channel in the bottom of the diversion channel and the existing river upstream of the drop structures.

Efforts were made to minimize the impacts at each drop structure. The drop structures were sited to avoid having grading work or constructed features on deed-restricted properties, minimize to the extent possible impacts to the estimated Ordinary-High-Water-Mark (OHWM) at each location and minimize the length of realigned river channels to the extent possible.

Future micro-siting efforts should address the following in greater detail:

- A significant potential cost saving concept was identified late in this phase that should be explored as soon as is feasible. Rather than drop the Lower Rush River into the diversion as shown on these drawings consideration should be given to diverting the Lower Rush to flow to the north up above the diversion along the west side until the channel reaches the Rush River. At this point a combined drop structure could be constructed at a considerable savings to the project.
- Coordinate the fish passage with the mitigation of the meandering low flow channel in the diversion itself.

F5.4.2 Hydraulic Structure Civil Site Design (Lower Rush River Structure, Rush River Structure)

The feasibility civil site design methodology of the Lower Rush River Drop Structure (LPP) and Rush River Drop Structure (LPP) is described below. These civil site designs are used to estimate quantities for feasibility cost estimates.

Access roadways are located to provide maintenance (not public) access to each hydraulic structure. Access roads are assumed to have gravel surfacing and be 12 ft wide for servicing minimal amounts of traffic during observation or maintenance activities, during both flooding and non-flooding times. Fences and gates are assumed for limiting or prohibiting public access. Access to each hydraulic structure is provided:

- Access for the LPP Lower Rush River Drop Structure is from the west side of the diversion channel on top of the protection levee, from Cass County Road 20(40th Ave North).
- Access for the LPP Rush River Drop Structure is on the west side of the diversion channel on top of the protection levee, from local roads.

A maintenance building and small parking area is located near each structure for storage of materials, equipment and usage during flooding events. Water and sanitary sewer service are not included. Power is shown routed to the site from the closest local power lines.

Remote monitoring thru a SCADA system is assumed. Data is gathered at strategic locations and delivered digitally back to a central monitoring station, where the information system is managed. Key monitoring points for both structures include:

- Monitoring of stage and head across the drop structure

Grading for embankment and channel excavation areas are shown with 7H:1V slopes. Grading for banks within the fish passage are shown with 3H:1V slopes. Diversion channel excavations at each structure are shown as:

- Lower Rush River Structure
    - Upstream – 250' bottom, 7:1 slope to 8' high, 15' bench, 7:1 slope to existing grade, 50' bench, 11' high levee/spoil pile, 320' Wide
    - Downstream – 250' bottom, 7:1 slope to 8' high, 15' bench, 7:1 slope to existing grade, 50' bench, 11' high levee, 299' levee/spoil pile wide

- Rush River Structure
    - Upstream – 250' Bottom, 7:1 slope to 8' high, 15' bench, 7:1 slope to existing grade, 50' bench, 11' high levee/spoil pile, 299' Wide
    - Downstream – 250' Bottom, 7:1 slope to 8' high, 15' bench, 7:1 slope to existing grade, 50' bench, 11' high levee/spoil pile, 359' Wide

Low-flow channels are included where water generated by normal, seasonal precipitation and runoff events (non-flooding events) must be drained to a waterway. Key locations for low-flow channels include:

- At LPP Lower Rush River Drop Structure in the main Diversion Channel upstream and downstream of the structure. Note that at the point where the fish passage for the Lower Rush enters the diversion the low flow channel in the

diversion will receive additional restoration so that it will be mitigation for the channel lost due to the project.
- At LPP Rush River Drop Structure in the main Diversion Channel upstream and downstream of the structure. This entire low flow channel is specially restored for mitigation reasons.

Areas requiring permanent riprap or other erosion control measures are shown for each hydraulic structure. The extents shown are conceptual, based on Phase 3 2-dimensional velocity modeling work and must be refined in final design. Key areas that must be permanently protection from erosion by high flows and/or turbulence include:

- Bank slopes of the fish passage;
- Channel bottoms at the bottom of the drop structure where the stilling well discharges into the main diversion
- Other areas where high flow velocities or turbulence are anticipated;

Fish passage systems are included with the drop structures. They are located upstream of the drop structure and provide connectivity of the proposed meandering low flow channel at the bottom of the diversion up to the existing river channel via a system of riffles and pools, protected from erosion by rock and boulders. Fish passage systems are provided at both drop structures.

Topsoil stripping, replacing and site restoration is assumed to be required for all areas permanently acquired by the project as well as permanent easement areas.

For hydraulic structure feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

Future hydraulic structure civil site design efforts should address the following in greater detail:

- Detailed design must address the issue of scour at hydraulic structures. Phase 4 erosion protection is based on Phase 3 2-dimensional velocity modeling and is conceptual.
- These structures are not as critical to the flood fight as those previously discussed. Future design efforts should consider whether permanent maintenance structures and power to the sites are needed.
- Consider altering the shape and size of the southern spoil pile immediately adjacent to the diversion in the vicinity of each drop structure. It should be possible to significantly shorten the southern retaining walls.

F5.5 Civil Design at Diversion Drop Structures (Connecting Channel Weir, Diversion Inlet Structure, Outlet Structure to the Red River of the North)

A feasibility-level civil site design is presented for each hydraulic structure to exhibit the overall grading, footprint, functionality and context of each structure and for the estimation of construction quantities for cost estimates. This section summarizes the micro-siting and civil site design methodology to develop feasibility grading and site plan for each of the similar-schemed drop structures (Diversion Inlet Structure, Outlet Structure to the Red River of the North) and the weir in the connecting channel between the Red River of the North and the Wild Rice River. The feasibility designs for the Inlet and Outlet drop structures all have mass concrete ogee drop structures, tied into adjacent levees with wing walls and are constructed in the dry. The connecting channel weir is a single sheet pile weir encased on both sides in rip rap. For additional information on these structures refer to Appendix F Drawings S-417, S-418, S-421 through S-423, and S-445 through S-447.

F5.5.1 Hydraulic Structure Micro-Siting (Connecting Channel Weir)

The connecting channel weir is located at a point reasonable close to midway between the Red River of the North and the Wild Rice River. It has been located where the connecting channel crosses Interstate 29. This location was selected since modifications to the freeway will be needed in this location to accommodate the project. It is possible that the weir can be replaced by a series of box culverts or a bridge with the invert under the road set at the top of the control weir.

F5.5.2 Hydraulic Structure Micro-Siting (Diversion Inlet Structure, Outlet Structure to the Red River of the North)

The feasibility micro-siting methodology of the Diversion Inlet Structure and Outlet Structures and the weir in the connecting channel (LPP) is described below. The siting of the individual structures sets the context for the civil site design at each structure.

It is assumed that the Diversion Inlet Structure and Outlet Structures and the connecting channel weir will be constructed in dry conditions. Micro-siting of each hydraulic structure is provided:

- The Diversion Inlet Structure is located at the west edge of the staging and storage areas where Cass County Road 17 meets Storage Area 1;
- The Outlet Structure into the Red River of the North is located approximately 800 ft south west of where the diversion intersects the Red River of the North;
- The connecting channel weir is constructed where the connecting channel crosses I-29.

The location of the Diversion Inlet Structure is a factor of where the storage and staging areas are with respect to the beginning of the diversion channel itself. The location of this structure is tied to the beginning of the diversion channel. The location selected assumes that a containment levee will be constructed immediately east of Cass County Road 17. The Diversion Inlet Structure is located near the south west corner of Storage Area 1 and is located immediately upstream from CR17 and downstream of existing utilities crossing into Storage Area 1.

Similarly, the Outlet Structure will ultimately be located just upstream of wherever the diversion outlets to the Red River of the North. The main issues affecting its location are related to managing backwater from the Red River during construction and how much additional excavation will be generated by moving the structure farther away from the Red River of the North. It is the intent of this structure to drop diversion flow down to an elevation that is close to the invert of the Red River of the North. The farther the structure is moved away from the river the more excavation will be needed to construct the last reach of the connecting channel.

A buffer of 300 ft between the proposed Outlet Structure and the existing river banks was used in the siting phase of this work. The inlet structure and connecting channel weir are not near enough to any rivers to require such a buffer. The Outlet Structure was sited to avoid having existing river banks within this buffer. This area represents a conceptual construction work area, plus area required to excavate 10H:1V slopes down to the structure construction area and to allow room for the construction of temporary earthen flood protection levees around the work site, if necessary. The assumed offset is to facilitate constructability and must be verified during final design when detailed local information (geotechnical, groundwater, etc.) and temporary construction facilities are available.

A permanent easement of 30 ft, offset from the extents of grading work, is assumed at each of the hydraulic structures. A temporary construction easement of 15 ft, offset from the edge of permanent easement is assumed at each hydraulic structure. This right-of-way is consistent with what is assumed for the channel portions of the project.

A fish passage facility for the Outlet Structure is only shown conceptually at this point. None is needed for the Diversion Inlet Structure but there will be a need to address fish passage on the Outlet Structure if fish passage is to occur during the some of the low flow conditions. River stage is regularly high enough at this point to facilitate fish passage without any special fish passage feature.

Efforts were made to minimize the impacts at each drop structure. The drop structures were sited to avoid having grading work or constructed features on deed-restricted properties, minimize to the extent possible impacts to the estimated Ordinary-High-Water-Mark (OHWM) at each location and minimize the length of realigned river channels to the extent possible.

Future micro-siting efforts should address the following in greater detail:

- Coordinate how the Diversion Inlet Structure will be sited with relation to the Cass County Road 17 bridge.

- If the staging area is reconfigured the location of the Diversion Inlet Structure will need to be moved accordingly.
- Final design of the fish passage for the Outlet Structure may impact the drop structure micro-siting slightly. This should be taken into consideration when selecting the exact outlet location during final design.
- Consider Outlet Structure location with respect to potential sediment, ice and debris issues on the main stem of the Red River of the North. The current location will create a backwater condition that may tend to accumulate sediment or ice and debris.

F5.5.3 Hydraulic Structure Civil Site Design (Diversion Inlet Structure, Outlet Structure to the Red River of the North)

The feasibility civil site design methodology of the Diversion Drop Structures (Diversion Inlet Structure, Outlet Structure to the Red River of the North) and the connecting channel weir is described below. These civil site designs are used to estimate quantities for feasibility cost estimates. It is assumed that all of these structures, both the inlet and the outlet structures, will be constructed in the dry.

Diversion Inlet Structure

The Diversion Inlet Structure consists of a mass concrete ogee spillway with a crest width of 90 ft and elevation of 903.25. The drop from the crest to the downstream apron is 19.25 ft. Downstream of the ogee spillway is a stilling basin sized to contain the hydraulic jump. A concrete sill is located at the downstream end of the stilling basin structure to dissipate energy.

Access roadways are located to provide maintenance (not public) access to the Diversion Inlet Structure. Access roads are assumed to have gravel surfacing and be 12 ft wide for servicing minimal amounts of traffic during observation or maintenance activities, during both flooding and non-flooding times. Fences and gates are assumed for limiting or prohibiting public access. Access to the Diversion Inlet Structure will be provided off of CR17. Access will be provided on both sides since no bridge is planned on the structure itself as currently laid out. Since CR17 is immediately adjacent to the site this access is cost effective.

A maintenance building and small parking area is located near each structure for storage of materials, equipment and usage during flooding events. Water and sanitary sewer service are not included. Power is shown routed to the site from the closest local power lines.

Remote monitoring thru a SCADA system is assumed for the Diversion Inlet Structure. Data is gathered at strategic locations and delivered digitally back to a central monitoring station, where the information system is managed. Key monitoring points include:

- Monitoring of stage and head across the Diversion Inlet Structure

- Monitoring of head across the ice/debris control measures immediately upstream of the structure

The west levee of Storage Area 1 will be extended to the south and will tie directly into the Diversion Inlet Structure north retaining wall at elevation 924. The minimum crest width of 15 ft will be carried to the retaining wall allowing access for maintenance vehicles. The extended levee will maintain the same 4H:1V side slopes as the levee system around Storage Area 1. A levee of similar dimensions will extend from the south retaining wall and tie back into the staging area containment levee which will be just east of CR17.

The upstream channel leading to the Diversion Inlet Structure is 100 ft wide and has a negative slope draining water in the channel back to the Wild Rice River until water levels exceed the drop structure crest elevation of 903.25. The upstream channel does not contain a low flow channel due to the negative slope and has a finished ground elevation of 901 at the Diversion Inlet Structure. Side slopes of the channel are at 7H:1V and extend up to the existing ground surface elevation of approximately 915. Wing walls will be placed on the upstream side of the Diversion Inlet Structure and will extend out until the upstream channel ties into the existing ground elevation of 915. Riprap will also be placed in the channel bottom and side slopes for a distance of approximately 25 ft to prevent upstream scour.

The downstream channel widens to 250 ft and has positive drainage away from the structure. The channel includes a low flow channel with a bottom elevation of 884 and main channel elevation of 887. The channel will slope back to existing grade with 7H:1V side slopes and will include benches. Downstream wing walls will extend from the structure out to the point where the channel slope ties back into the finished grade of the diversion channel at elevation 912. Riprap will also be placed in the downstream channel and on the side slopes for an approximate distance of 50 ft to reduce the potential for downstream scour and erosion.

Immediately downstream of the Diversion Inlet Structure, CR17 crosses the diversion channel. A bridge will be required to span the channel. The bridge will be located downstream of the Diversion Inlet Structure crest to help prevent the potential for debris and ice build-up.

During the final design, the bridge may be incorporated as part of the Diversion Inlet Structure and at a minimum, will be sited to minimize the distance the bridge spans the channel. Modifications to the downstream channel will also be considered to minimize the length of the downstream wing walls. Additional consideration will also be given to energy dissipation. A combination of baffle blocks, sill, plunge pool, and other energy dissipation will be reviewed to minimize the downstream energy and size of the stilling basin structure.

The connecting channel weir is the starting point for the low flow channels leading toward both the Red River to the East and the Wild Rice River to the west. The weir is a single row of sheet pile encased in riprap. Access to the structure will be from the west along the top of the protective levee leading form the Wild Rice Structure.

An Ice and Debris Control Measure is located immediately upstream of the Diversion Inlet Structure to direct ice away from the ogee ramp spillway, keeping the diversion channel as free of ice and debris as possible. This measure is located to make it accessible during flooding events (the level of accessibility depends on the final measures implemented).

Topsoil stripping, replacing and site restoration is assumed to be required for all areas permanently acquired by the project as well as permanent easement areas.

For hydraulic structure feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

Future hydraulic structure civil site design efforts should address the following in greater detail:

- Detailed design must address hydraulics on ogee spillways and determination of design events.
- Detailed design must address the issue of scour at hydraulic structures. Phase 4 erosion protection is based on Phase 3 2-dimensional velocity modeling and is conceptual.
- Site design may need to be modified depending upon the nature of recreational features incorporated in final design.
- Projects of this scale will often include public art or other modifications to enhance the visual appeal of what is built. Civil site works may need to be modified to accommodate these features.
- Maintenance access to the areas upstream and downstream of all control structures and on both sides of the diversion channel should be provided in final design.

<u>Outlet Structure</u>

The Outlet Structure from the diversion back to the Red River of the North will be located approximately 800 ft upstream in the diversion from the river. The LPP Diversion Channel outlets to the Red River at Red River Station 2208555. The Outlet Structure consists of a mass concrete ogee spillway with a crest width of 250 ft and elevation of 866. The Outlet Structure drops to a downstream apron elevation of 843.9 for a total drop of 22.1 ft. A concrete stilling basin and concrete sill are immediately downstream of the ogee drop to control the hydraulic jump and dissipate energy. Concrete retaining walls border each side of the drop structure.

Access roadways are located to provide maintenance (not public) access to the outlet structure. Access roads are assumed to have gravel surfacing and be 12 ft wide for servicing minimal amounts of traffic during observation or maintenance activities, during both flooding and non-flooding times. Fences and gates are assumed for limiting or prohibiting public access. Access to the Outlet Structure will be provided off of 173rd Ave SE (CR31). Access will be provided on both sides since no bridge is planned on the structure itself as currently laid out. Since CR31 is immediately adjacent to the site this access is cost effective.

A maintenance building and small parking area is located near each structure for storage of materials, equipment and usage during flooding events. Water and sanitary sewer service are not included. Power is shown routed to the site from the closest local power lines.

Remote monitoring thru a SCADA system is assumed for the Outlet Structure. Data is gathered at strategic locations and delivered digitally back to a central monitoring station, where the information system is managed. Key monitoring points include:

- Monitoring of stage and head across the Outlet Structure

The diversion channel approaching the drop Outlet Structure is 250 ft wide and contains a low flow channel. Both channels slope towards the Outlet Structure with the invert elevation of the low flow channel being 861.2 and the invert of the main channel bottom being 864.2. A fish passage is shown conceptually connecting the low flow channel to the Red River of the North via a series of riffles and pools. The channel will pass through a specially design section of the drop Outlet Structure wing walls and form their along the west side of the last section of the diversion connecting the project to the Red River.

The upstream channel contains 7H:1V side slopes up to the existing ground of approximately 880.0. Embankments consisting of the channel spoil line both sides of the upstream diversion channel, however, are offset approximately 50 ft before continuing with the 7H:1V slopes up to the top elevation of 889.7. The top width of the spoil pile is approximately 410 ft before back slopes extend back to existing ground of 880.0 at 10H:1V slopes. The spoil piles extend to the drop Outlet Structure where both upstream and downstream wing walls extend out and tie into the spoil pile crest.

The upstream end of the Outlet Structure is protected from scour by the wing walls with a top elevation of 889.7 (5 ft higher than the 500-yr flood elevation on the Red River of the North at this location) which extend out to the spoil pile and riprap placed on the channel bottom and channel side slopes for an approximate distance of 25 ft. The downstream structure also consists of wing walls at 889.7 that tie back into the spoil pile crest and riprap which is placed for a distance of 50 ft on the channel bottom and side slopes.

The downstream channel sits at elevation 843.9 and is 250 ft wide. The channel side slopes are 7H:1V and extend up to existing ground at approximately 880.0. During normal river conditions, approximately 9.5 ft of water will fill the downstream channel where water levels will equilibrate with the elevation of water in the Red River of the North. This is based on the median flow rate and stage in the Red River of the North. The presence of a permanent pool in this location will assist in energy dissipation of low flows exiting the diversion which will be almost constant given the amount of drainage intercepted by the diversion.

The spoil piles have a crest width of approximately 410 ft and sit 5 ft above the 500-yr flood level.

During the final design, modifications to the downstream channel will be considered to minimize the length of the downstream wing walls. Additional consideration will also be given to energy dissipation. A combination of baffle blocks, sill, plunge pool, and other energy dissipation will be reviewed to minimize the downstream energy and size of the stilling basin structure.

Topsoil stripping, replacing and site restoration is assumed to be required for all areas permanently acquired by the project as well as permanent easement areas.

For hydraulic structure feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

Future hydraulic structure civil site design efforts should address the following in greater detail:

- Detailed design must address hydraulics on ogee spillways and determination of design events.
- Detailed design must address the issue of scour at this structure. Phase 4 erosion protection is based on Phase 3 2-dimensional velocity modeling and is conceptual.
- Site design may need to be modified depending upon the nature of recreational features incorporated in final design.
- Projects of this scale will often include public art or other modifications to enhance the visual appeal of what is built. Civil site works may need to be modified to accommodate these features.
- Maintenance access to the areas upstream and downstream of all control structures and on both sides of the diversion channel should be provided in final design.

F5.6 Civil Design at Closure/Drainage Structures (Storage Area 1 Closure/Drainage Structure North, Storage Area 1 Closure/Drainage Structure East, Wolverton Creek Closure/Drainage Structure, Drain 14 Drop Structure)

The feasibility civil design methodology for Closure/Drainage Structures for Wolverton Creek (FCP) and on the north side (North Outlet) and east side (East Outlet) of Storage Area 1 (LPP) as well as the Drain 14 Drop Structure is described below. These civil designs are used to estimate quantities for feasibility cost estimates. For additional information on these structures refer to Appendix F Drawings S-411, S-418 through S-420, and S-430.

F5.6.1 Hydraulic Structure Micro-Siting

Each of the structures discussed in this section are sited simply at the location where an existing drainage channel is interrupted by either a levee or the diversion channel. Since all are being built in the existing channel with flows routed around them there is little room for moving them.

A permanent easement of 30 ft, offset from the extents of grading work, is assumed at each of the hydraulic structures. A temporary construction easement of 15 ft, offset from the edge of permanent easement is assumed at each hydraulic structure. This right-of-way is consistent with what is assumed for the channel portions of the project.

Future micro-siting efforts should address the following in greater detail:

- Coordinate how the Wolverton Creek closure structure will be sited with relation to the Clay County Road 60 (130th Ave South). It may be beneficial to rout the County road up onto the levee at this location and over the closure structure rather than constructing a separate culvert crossing.
- If Storage Area 1 is reconfigured the location of the north and east outlets will also be changed. At this time their location is relatively independent of all other factors since no roads are nearby either of them.
- Siting of the Drain 14 structure should be coordinated with nearby road embankments that may be needed to contain high flows from the diversion which could back flow into the surrounding area. This condition may be alleviated with the addition of backflow preventions gates on the Drain 14 structure itself if local flood peaks can be handled without threatening local structures.
- Consider eliminating Drain 14 drop structure entirely by routing Drain 14 north to the Maple River and combining this flow with that of the Maple River. This would make the Maple River structure substantially larger but result in an overall cost savings to the project.
- Consider dropping a base flow portion of Drain 14 into the diversion via a smaller more affordable drop structure at this location but carry higher flood flows north to the Maple River as discussed in the point above.

F5.6.2 Hydraulic Structure Civil Site Design

It is assumed that all four structures will be constructed in the existing channel, as flows are low enough that they can be diverted around the construction sites. The four structures are designed to allow flows in the respective channels to pass through levee or spoil pile embankments.

Since the preliminary designs for the three closure drainage structures are similar, the description below applies to all of them (SA1 North, SA1 East and Wolverton).

- Each structure will consist of parallel concrete retaining walls, 10 ft apart for the SA structures and 24 ft apart for the Wolverton structure, passing through the levee. The retaining walls will be supported by concrete footings sized to prevent overturning and sliding. Concerns with long term settlement will require pile foundations supporting the footings. The preliminary sizing of the retaining walls and footings is based on the design of similar structures on other projects. Appropriate drainage measures will be included behind the retaining walls to prevent the buildup of hydrostatic pressure behind the walls.

- A heated sluice gate will be used to control flow through each structure. Heated bulkhead slots on the downstream side of the sluice gate will be installed as a backup closure mechanism. It is assumed that the sluice gate will remain open under normal conditions and will be closed during major flood events. A bridge will span the structure along the top of the levee and provide access for operating the sluice gate and adding stop logs to the bulkhead. Wingwalls on the upstream side will funnel water into the structure, prevent seepage, and control erosion along with upstream riprap placed in the approach channel and side slopes.

The Drain 14 Structure is a drop structure only with no closure element. The structure is very similar to those proposed for the Rush and Lower Rush Rivers. Only basic structure sizing was perfomed during this round of work for the purposes of arriving at quantities for the cost estimate. Additional work related to the civil site around this structure will be needed during future phases of work.

Access roadways are located to provide maintenance (not public) access to each hydraulic structure. Access roads are assumed to have gravel surfacing and be 12 ft wide for servicing minimal amounts of traffic during observation or maintenance activities, during both flooding and non-flooding times. Fences and gates are assumed for limiting or prohibiting public access. Access to each hydraulic structure is provided:

- Access for the LPP Wolverton Creek Structure will be from a short driveway off of Clay County Road 60.
- Access for the Storage Area 1 structures will be from a road constructed along the top of the impoundment levee. For the North structure this road will be accessed from Cass CR 14. For the East structure this road will be accessed from Cass CR 21.

- Access for the Drain 14 Structure will be from the north off of 32nd Ave NW.

A maintenance building and small parking area is located near each structure for storage of materials, equipment and usage during flooding events. Power is brought to these sites. Remote monitoring thru a Supervisory Control and Data Acquisition (SCADA) system is assumed for these sites. Water and sanitary sewer service are not included.

The preliminary design dimensions for the levee embankments adjacent to all but Drain 14 are a 15 ft top width, 4:1 side slopes and a crest elevation of 927. Drain 14 is planned to pass through the spoil pile on the west side of the diversion channel where the top width is currently shown at 100 ft with 7:1 side slopes on the spoil pile and a crest elevation of 905 is planned.

Topsoil stripping, replacing and site restoration is assumed to be required for all areas permanently acquired by the project as well as permanent easement areas.

For hydraulic structure feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

Future hydraulic structure civil site design efforts should address the following in greater detail:

- Consider altering the shape and size of the spoil pile immediately adjacent to the diversion in the vicinity of Drain 14. It should be possible to significantly shorten the spoil pile retaining walls.
- Consider if the Drain 14 structure can be eliminated in its entirety by routing drainage along the west side of the diversion embankment and dropping it into the Maple River.
- Site design around each of these features may need to be modified depending upon the nature of recreational facilities incorporated in final design.
- Maintenance access to the areas upstream and downstream of all control structures and on both sides of the diversion channel should be considered in final design.
- The hydraulic opening will need to be refined based on contributing drainage area, elected design storm event, and draw-down time requirements after major floods. Preliminary sizing is based on the contributing drainage area and approximate sizes of existing drainage culverts for the respective channels. A uniform size was selected for all three structures to facilitate cost estimating. It is assumed that refinements to the size of the hydraulic opening will not have a significant effect on the overall cost of each structure.

- Energy dissipation will be necessary downstream of each structure. This could take the form of a stilling basin, baffle blocks, sill, plunge pool, or other appropriate methods. The energy dissipation design will depend largely on the design flows passing through the control structure.
- Effective seepage control will be critical for levee stability. It is assumed that wing walls will help to prevent seepage and a combination of granular filters and drains will be used to safely control seepage minimizing the potential for piping and providing long term stability of the embankment near structures.
- Long term settlement concerns will need to be addressed for the levee embankment and each structure.

F5.7 Civil Design at Storage Area 1

The feasibility civil site design methodology of Storage Area 1 is described below. These civil site designs are used to estimate quantities for feasibility cost estimates.

F5.7.1 Storage Area 1 Micro-Siting

Storage Area 1 is a 4360-acre area located on the north side of the Diversion Channel between the Wild Rice River and the Sheyenne River in North Dakota. For additional information on the levee embankments see Exhibit E of Appendix F and drawing S-418.

Storage Area 1 will affect several existing roads. In general, County Roads will be maintained through the storage area and minor roads will be interrupted when they intersect levee embankments. There are three locations where County Roads will be brought up and over the Storage Area 1 levees. These are County Road 16 over the South and East Levees, and County Road 21 over the East Levee. It is assumed that the area inside Storage Area 1 will be usable as cropland when not in use for flood protection.

A permanent easement of 30 ft, offset from the extents of grading work, is assumed around the north, west and east sides of the Storage Area 1 facility. A temporary construction easement of 15 ft, offset from the edge of permanent easement is also assumed.

Future micro-siting efforts should address the following in greater detail:

- Storage Area 1 is needed to contain a significant amount of storage. Micro-siting of this facility pertains to the exact location of the containment levees. This should be coordinated with local officials during final design to the extent reasonable to vary the location of the containment levee as needed to maximize storage while minimizing impacts to local existing structures, utilities and transportation facilities.
- Storage Area 1 siting needs to be coordinated with the location of the diversion inlet and the location of the north and east outlets.

- Note that the interior area of Storage Area 1 is permanently acquired as part of the project. Detailed design and planning of this area is not included at this feasibility stage.

F5.7.2 Storage Area 1 Civil Site Design

Storage Area 1 will be surrounded by a levee at elevation 927, which is nearly 2 ft above the maximum water surface elevation during the SPF (see Section F2.2.18). Over 55,000 acre-feet of storage will be provided during the 100-yr and 500-yr flood events. The levee top width is 15 ft and the side slopes are 4:1 H:V. Existing ground elevations generally range from 908 to 915.The South Levee will be constructed from spoils from the diversion channel. The East, North, and West Levees will be constructed primarily from borrow trenches within the storage area. The area is very large and access to it will be from many local and regional roads.

There will be three openings in the levees surrounding Storage Area 1. The Inlet-Outlet Opening will be a 1400 ft gap in the South Levee near the Wild Rice River Control Structure. The North Outlet and East Outlet will both be gated structures that are left open during non-flood conditions, and closed during major flood events.

The Inlet-Outlet Opening is hydraulically connected with the rest of the upstream staging area. The width of the opening is large enough that flow velocities into and out of the storage area will be low. In general, existing grade will be maintained at the inlet. It is assumed that no armoring of the horizontal ground plane at the inlet will be necessary. Permanent vegetation will be required over this area.

Internal drainage within the storage area will occur through existing drainage ditches. The borrow trenches for the embankments will slope to the naturally occurring existing North and East Outlets. These outlets will be constructed in-line with the ditches draining the two main drainage areas within Storage Area 1. The drainage area for the East Outlet will be approximately 1500-acres. The drainage area for the North Outlet will be approximately 2810-acres.

Both the North and East Outlets will be gated structures as discussed above in the previous section. During normal (non-flood) conditions, the North and East Outlet gates will remain open to allow for natural drainage of the storage area.

During flood events, the North and East Outlet gates will be closed. Once flood elevations rise above elevation 910, floodwaters will enter the storage area through the Inlet-Outlet Opening. Elevations within the storage area will equalize with the rest of the upstream staging area. After the flood peak has passed and the upstream staging level drops, water in Storage Area 1 will leave the storage area through the Inlet-Outlet Opening. Once floodwaters recede, areas within Storage Area 1 that are below elevation 910 will still be inundated. At that point, the North and East Outlets will be opened to allow those low-lying areas to drain.

Topsoil stripping, replacing and site restoration is assumed to be required only for those areas disturbed for containment levee construction and borrow and not all areas for which easements are acquired for this facility.

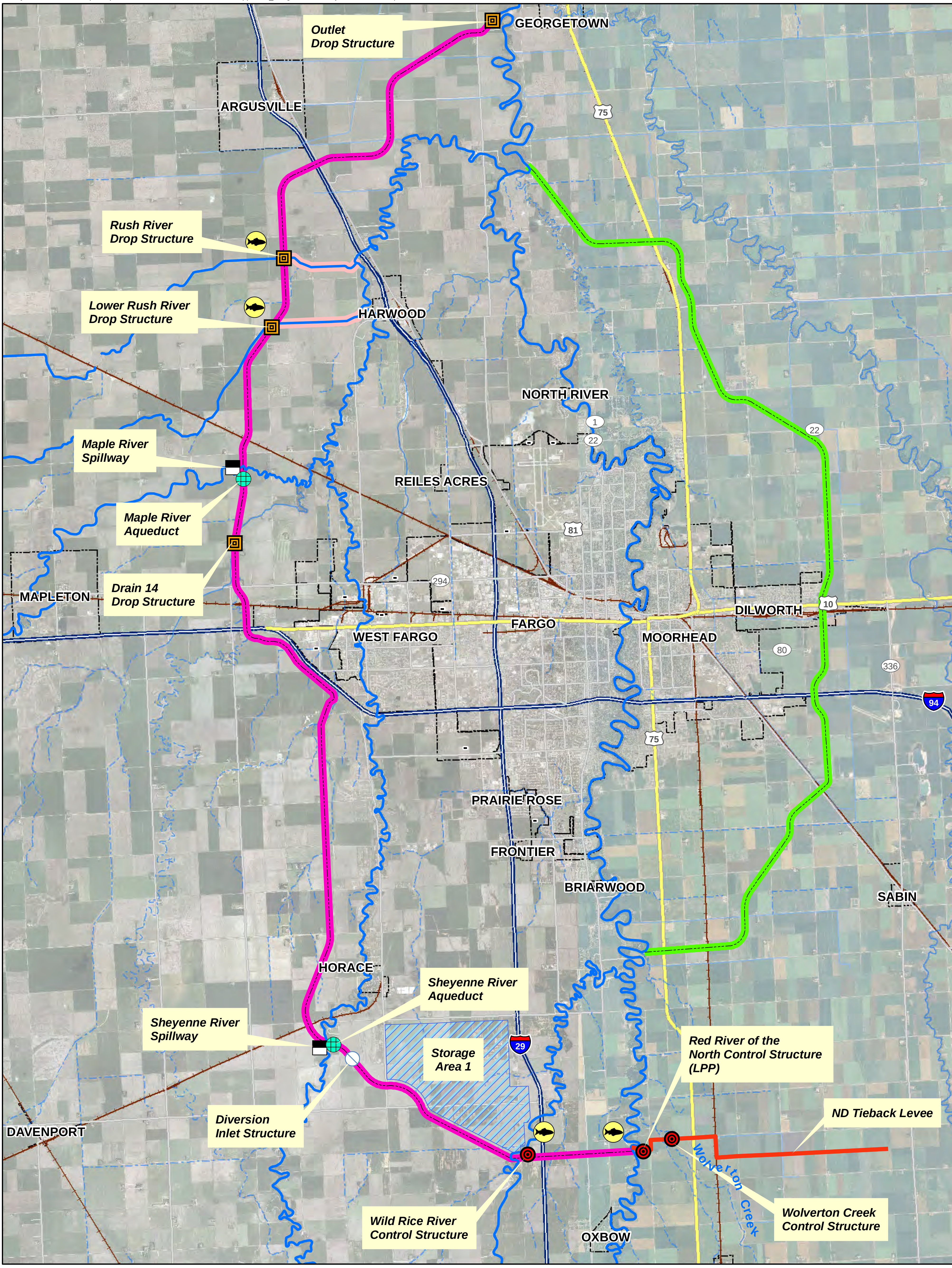
For Storage Area 1 feasibility civil site design, certain features are not shown on the drawings presented in Appendix F (as coordinated with USACE), including:

- Ecological mitigation areas (off-site or on-site)
- Locations of relocated utilities
- Recreation features
- Landscaping

The following issues will need further consideration during final design:

- Material sources for levee construction – As the cut/fill quantities are refined it will be worth assessing what is the most economical way to use spoils from the excavation of the Diversion Channel.
- Internal drainage – Make sure the levees do not create ponding areas that do not drain to one of the outlets.
- Size outlet structures to provide the necessary capacity for the design rainfall event, as well as meet drawdown time requirements after flood events.
- Detailed design must address issues and requirements related to the Levee System Evaluation for the National Flood Insurance Program. Detailed design should address operation and maintenance concerns associated with regular evaluation and certification under this program. For example, developing a comprehensive strategy related to instrumentation, monitoring, inspections, etc. and incorporating project features to facilitate system evaluation.

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F01 - Project Overview Map.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- ND Tieback Levee
- North Dakota Diversion Locally Preferred Plan (LPP)
- Minnesota Diversion Federally Comparable Plan (FCP)
- Channel Reclamation Reaches
- Storage Area 1

Figure F01

PROJECT OVERVIEW

Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

Figure F02 Longitudinal Profile of LPP Diversion Channel

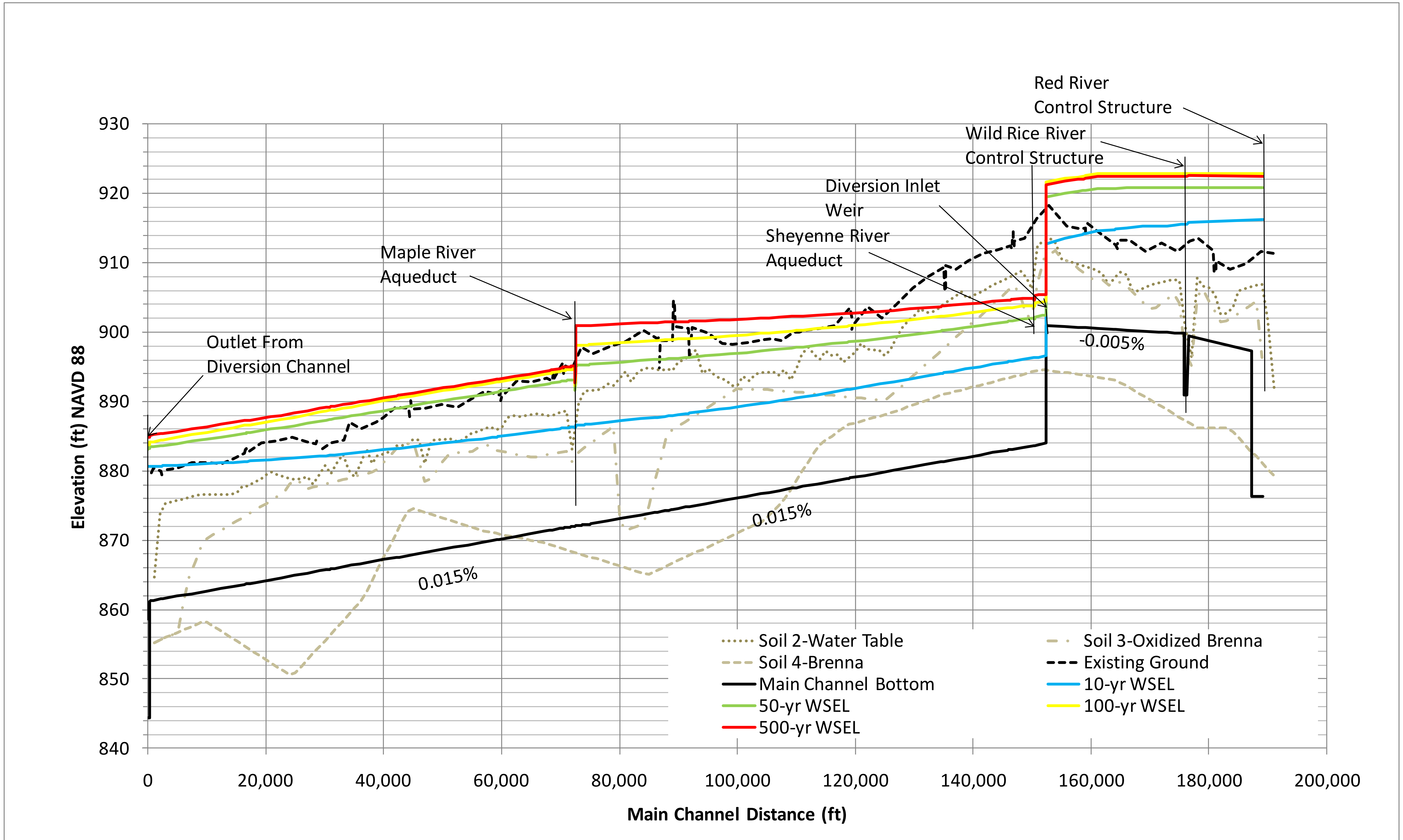


Figure F03 Storage Elevation Curves for Upstream Staging Area and Storage Area 1

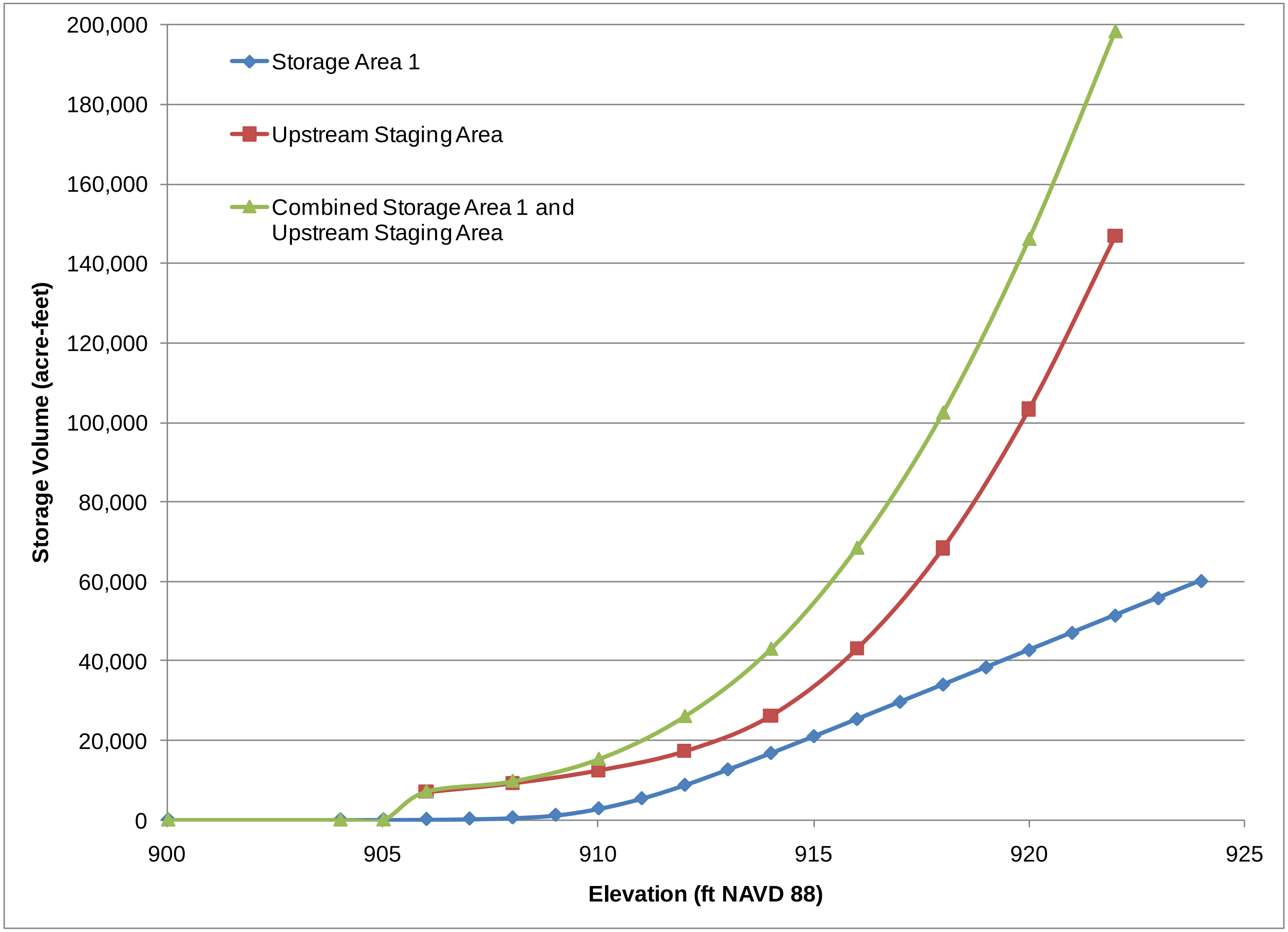
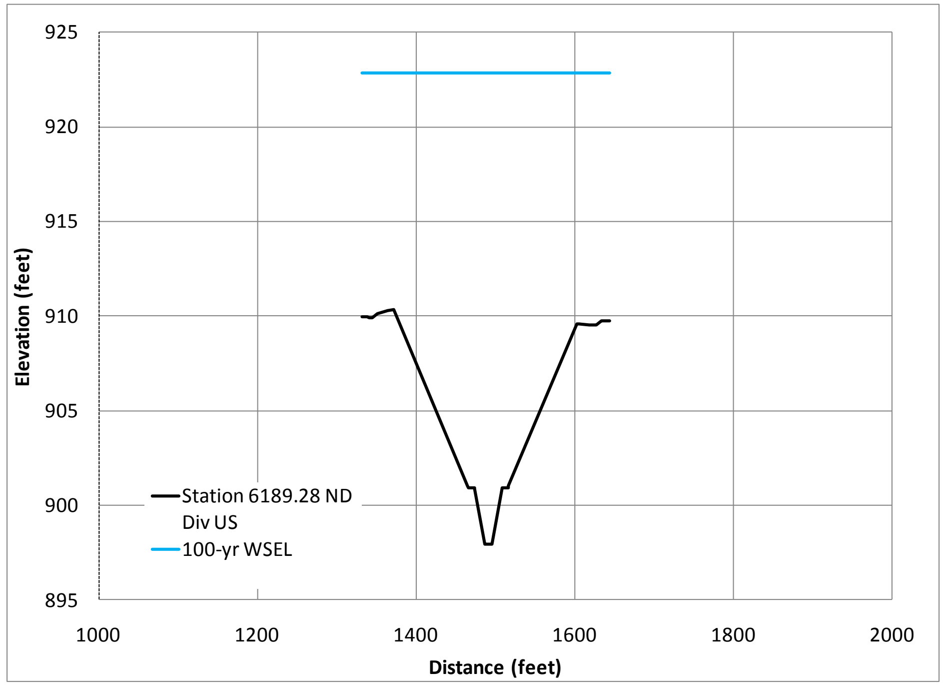
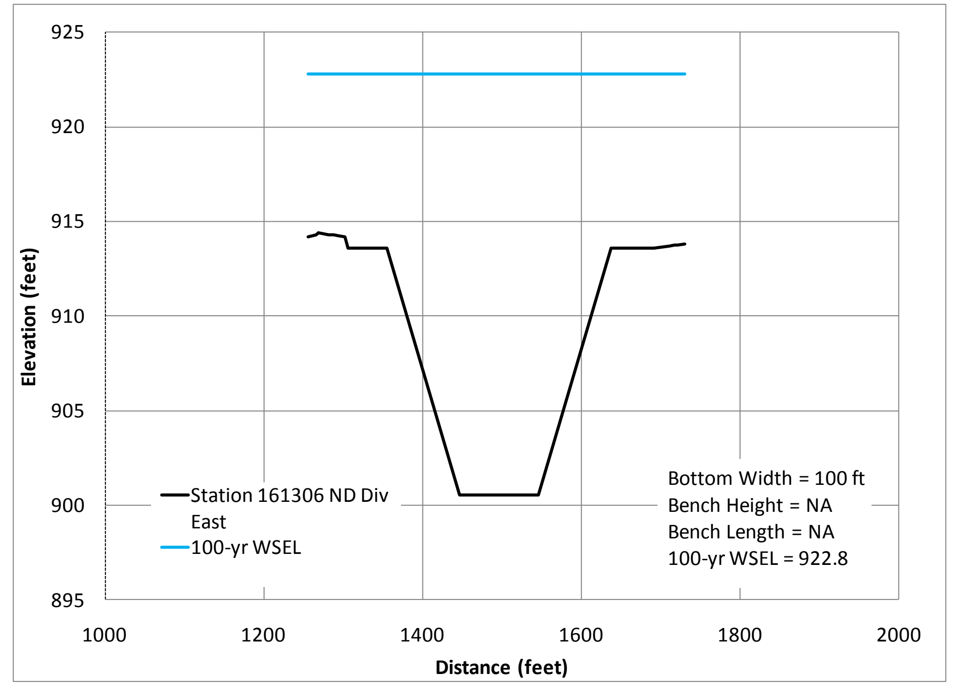


Figure F04 Typical Cross Sections of LPP Diversion Channel – See Note 1

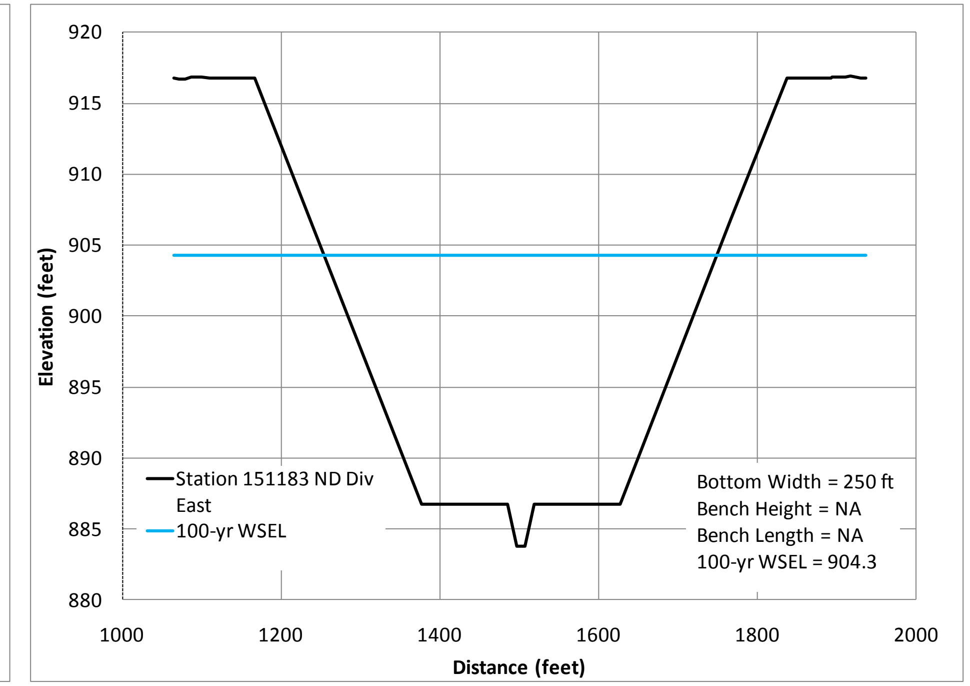
Station 6189.28 LPP ND Div US
Located Between Red River and the Wild Rice River



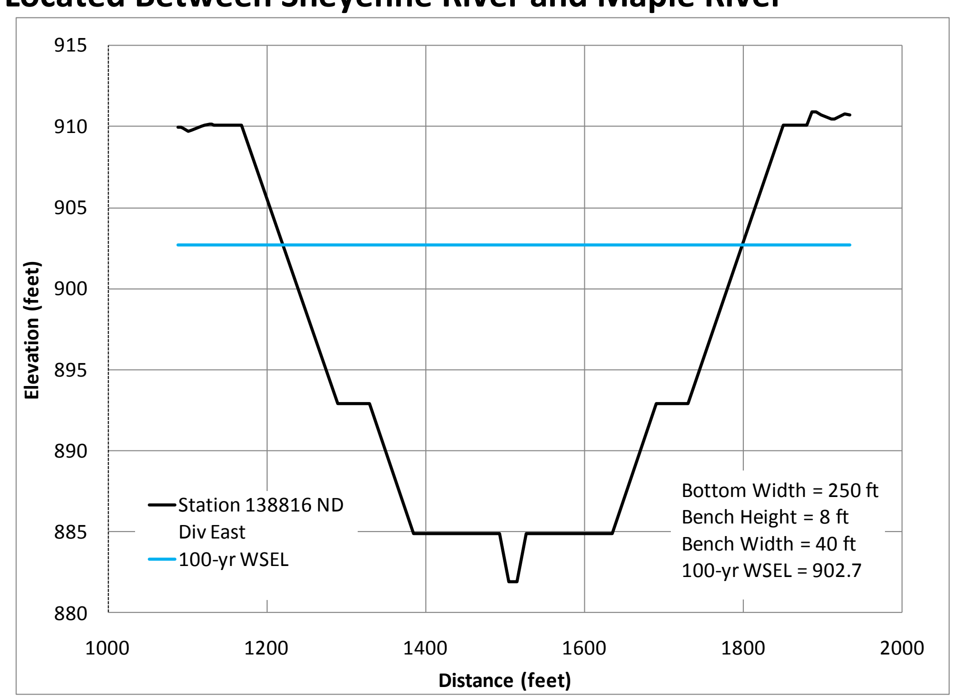
Station 161306 LPP ND Div East
Located Between Wild Rice River and Main Inlet Weir



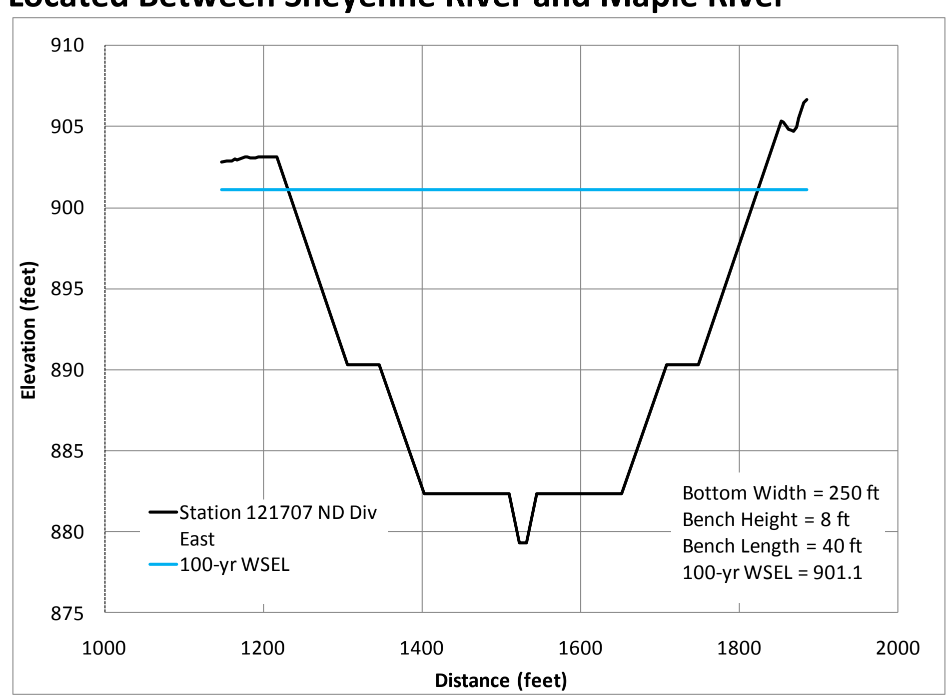
Station 151183 LPP ND Div East
Located Between Main Inlet Weir and Sheyenne River



Station 138816 LPP ND Div East
Located Between Sheyenne River and Maple River



Station 121707 LPP ND Div East
Located Between Sheyenne River and Maple River



Station 101591 LPP ND Div East
Located Between Sheyenne River and Maple River

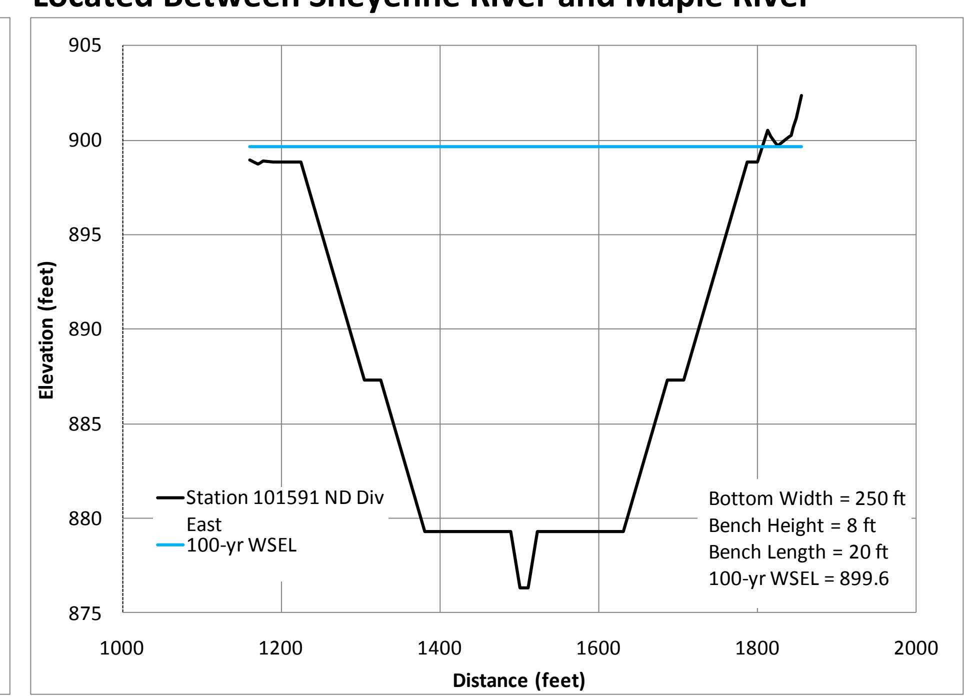
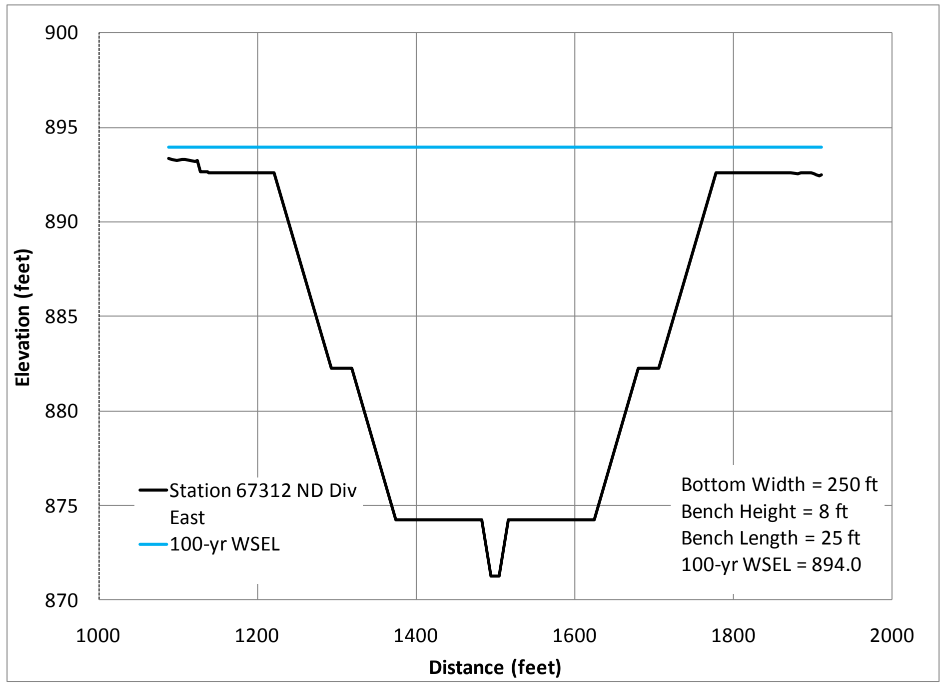
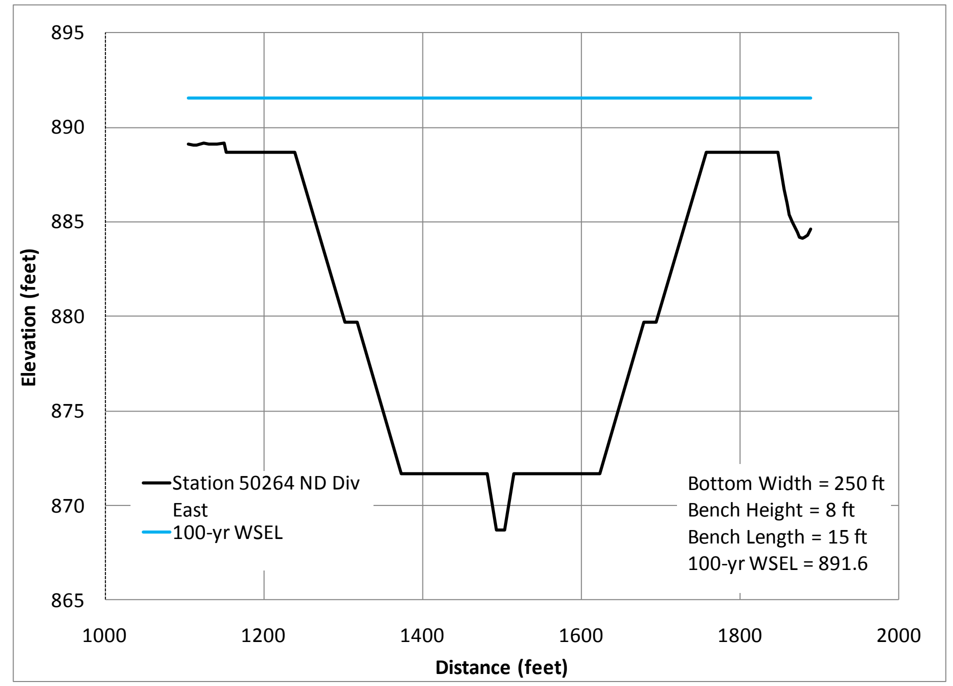


Figure F04 Typical Cross Sections of LPP Diversion Channel – See Note 1 (continued)

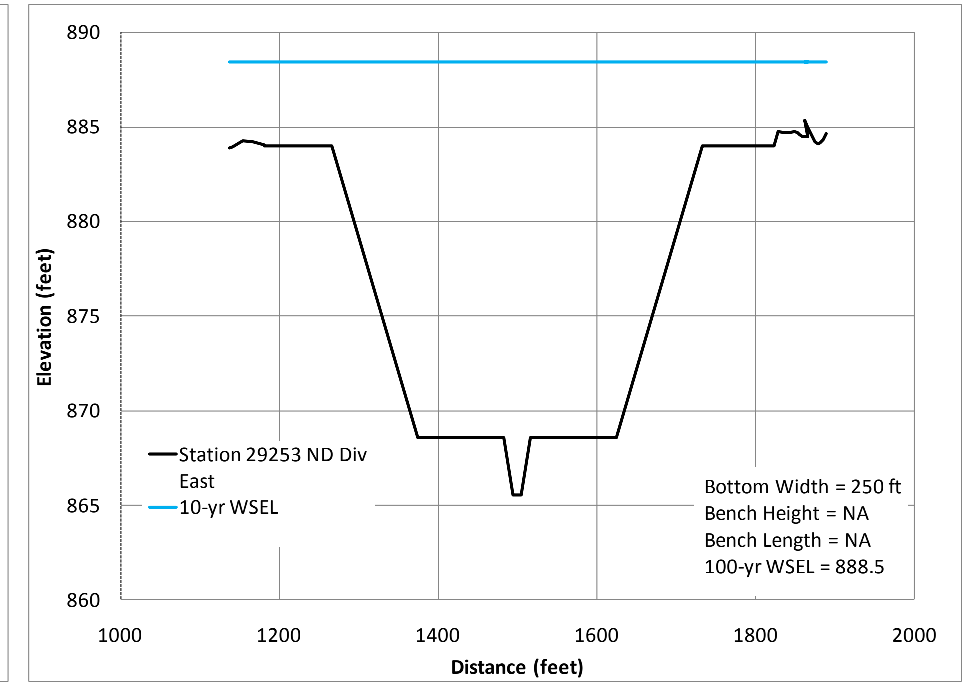
Station 67312 LPP ND Div East
Located Between Maple River and Lower Rush River



Station 50264 LPP ND Div East
Located Between Lower Rush River and Rush River

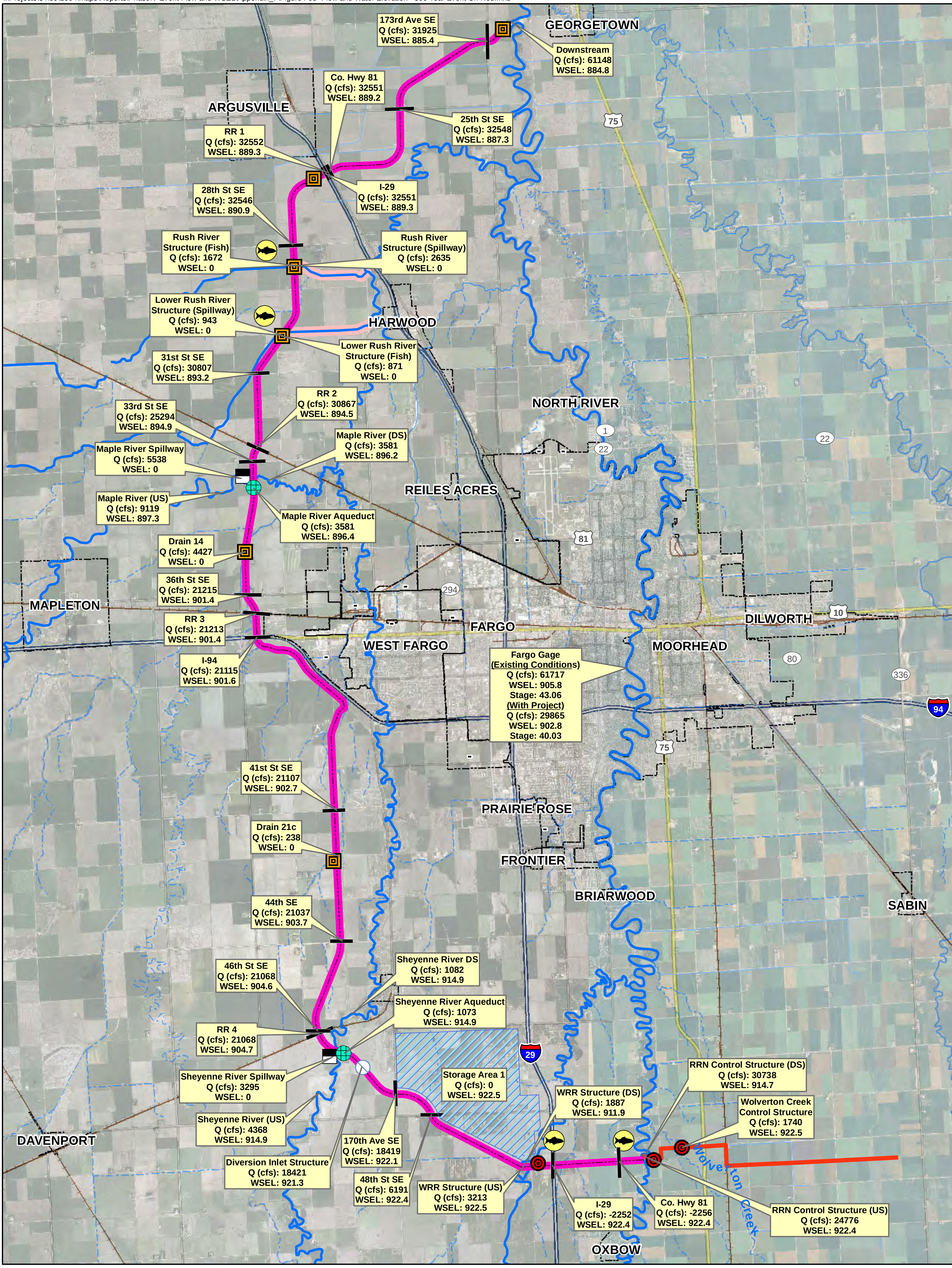


Station 29253 LPP ND Div East
Located Between Rush River and Diversion Channel Outlet



Note 1: The cross sections shown in this figure include only the area excavated.
Spoils and/or levees will contain water up to the 500-yr water surface elevation with a freeboard allowance of 3 feet.

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F05 -Flow and Water Elevation - 500 Year Event On Red.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

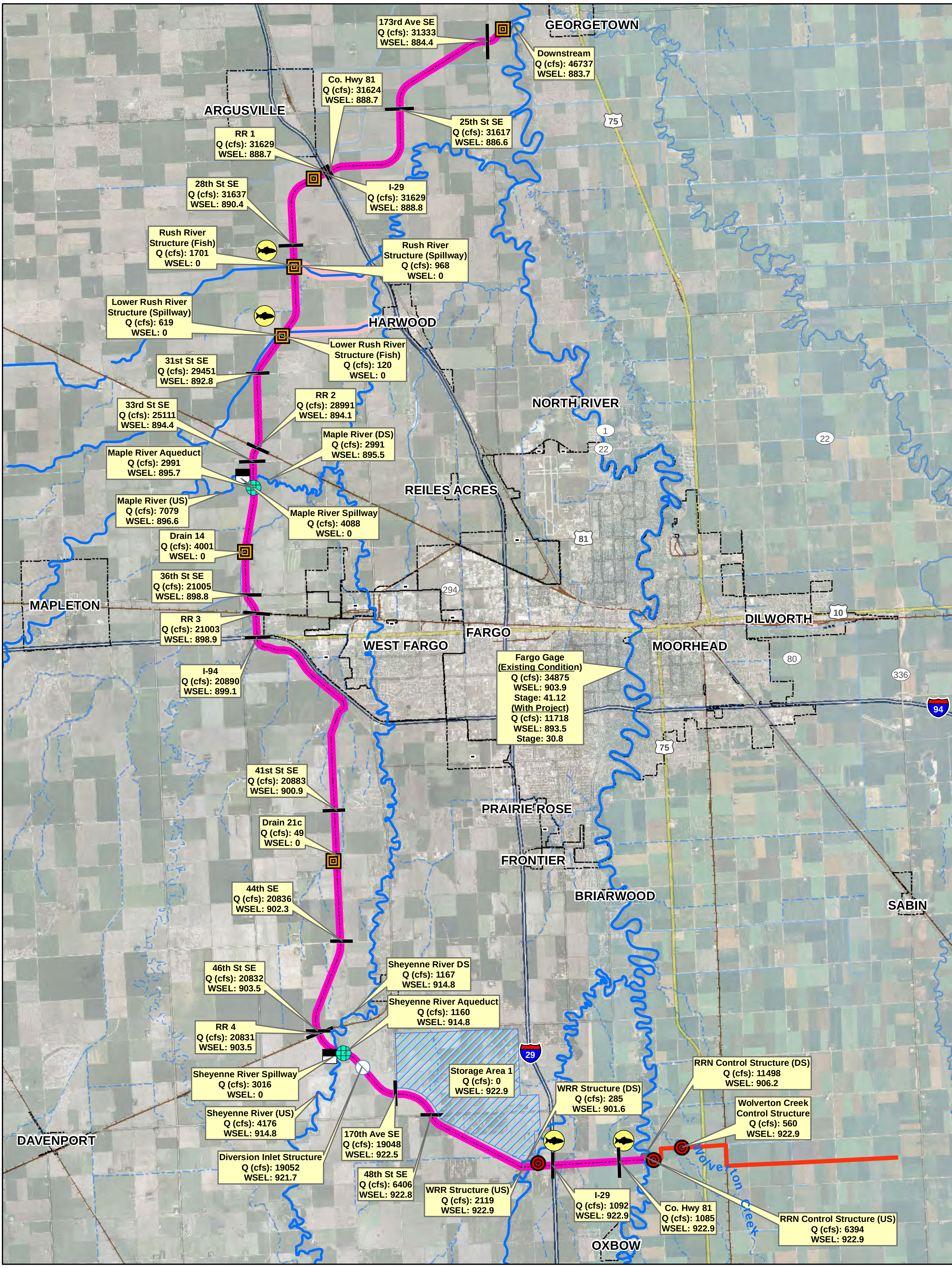
Figure F05

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 0.2-PERCENT CHANCE EVENT IN RED RIVER OF THE NORTH (AND COINCIDENTAL EVENT IN ND TRIBUTARIES)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F06 -Flow and Water Elevation - 100 Year Event On Red.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

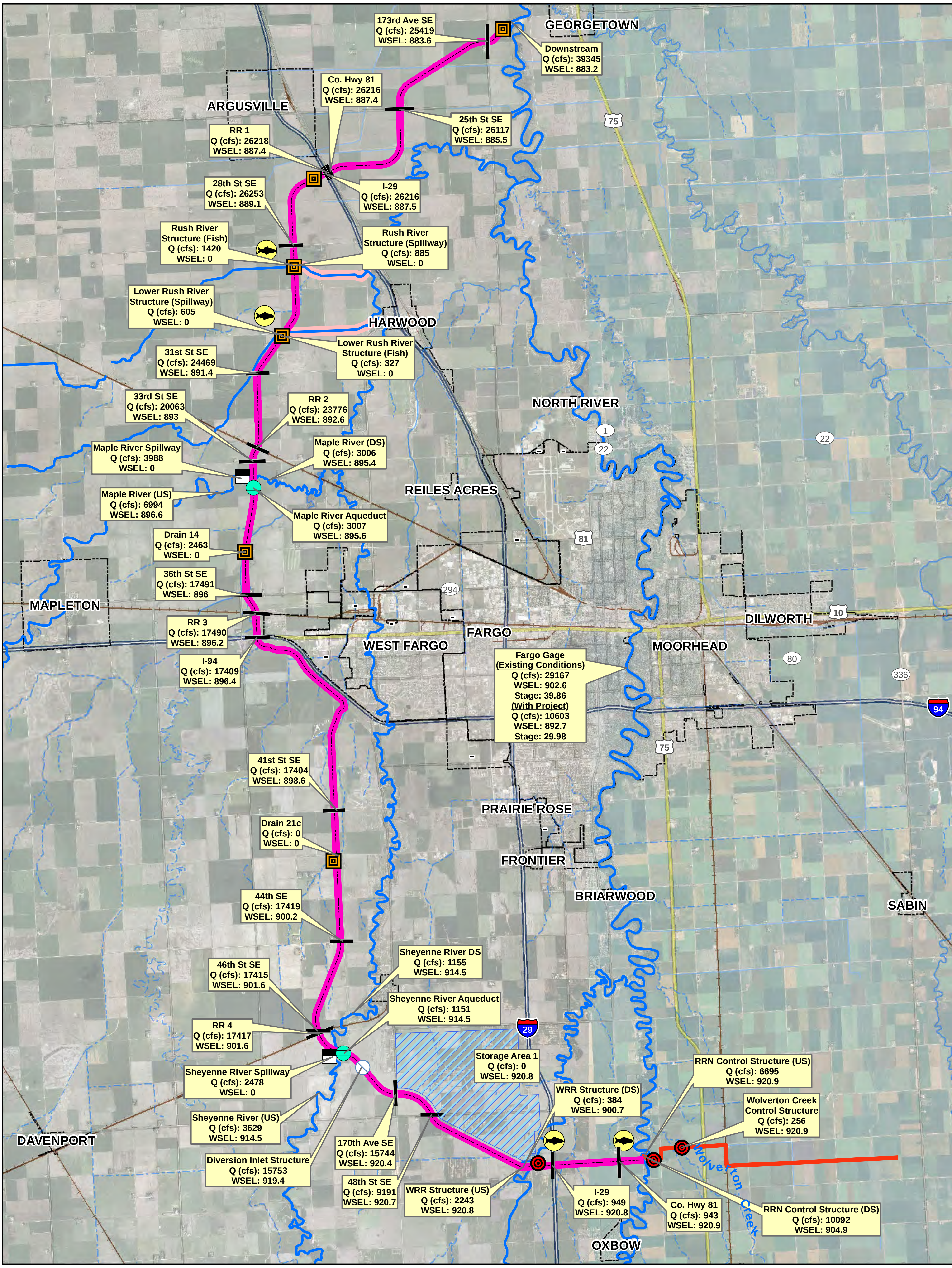
Figure F06

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 1-PERCENT CHANCE EVENT IN RED RIVER OF THE NORTH (AND COINCIDENTAL EVENT IN ND TRIBUTARIES)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F07 -Flow and Water Elevation - 50 Year Event On Red.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only. Flows in overbanks/floodplain are not reported.

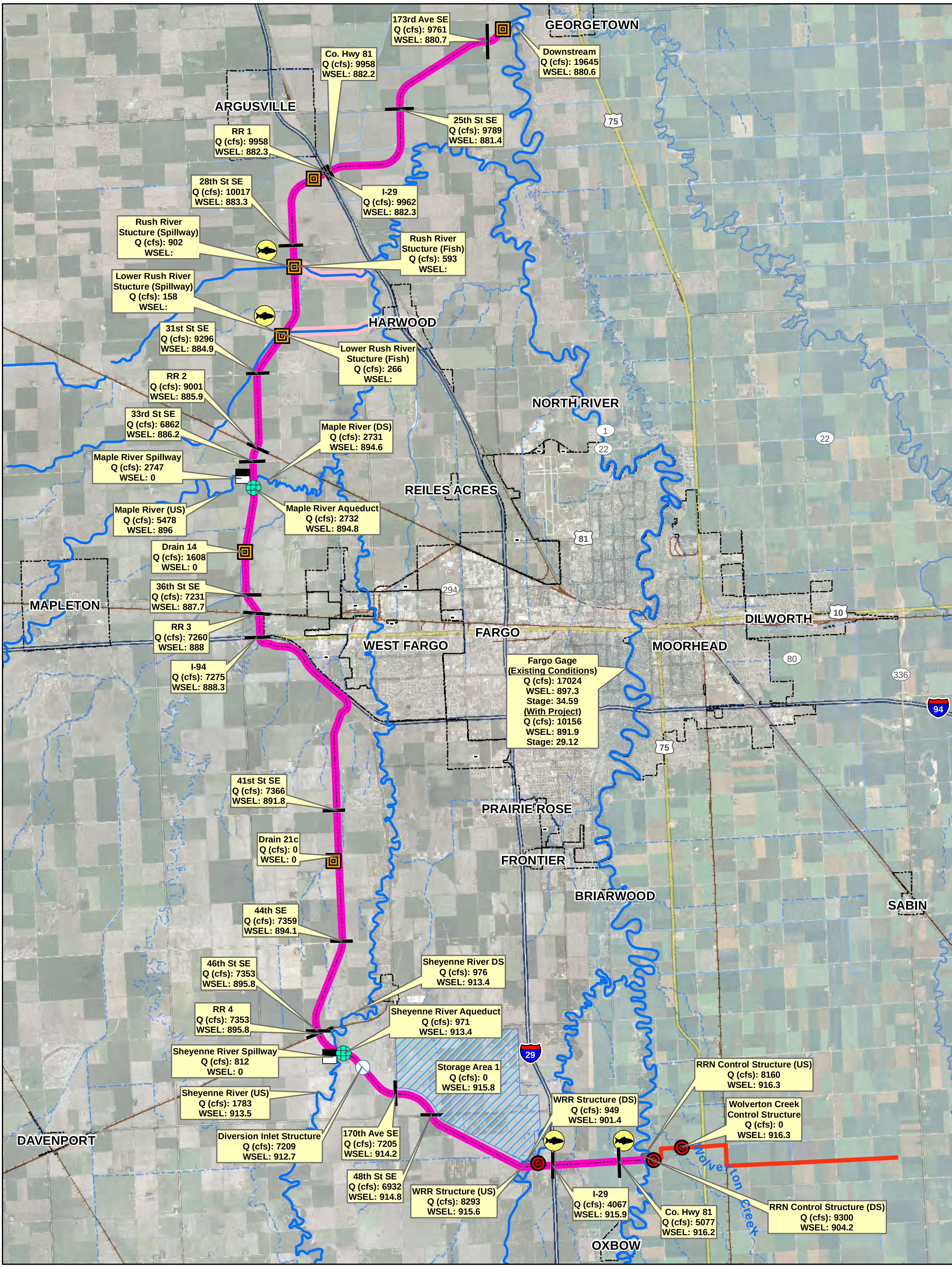
Figure F07

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 2-PERCENT CHANCE EVENT IN RED RIVER OF THE NORTH (AND COINCIDENTAL EVENT IN ND TRIBUTARIES)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F08 -Flow and Water Elevation - 10 Year Event On Red.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only. Flows in overbanks/floodplain are not reported.

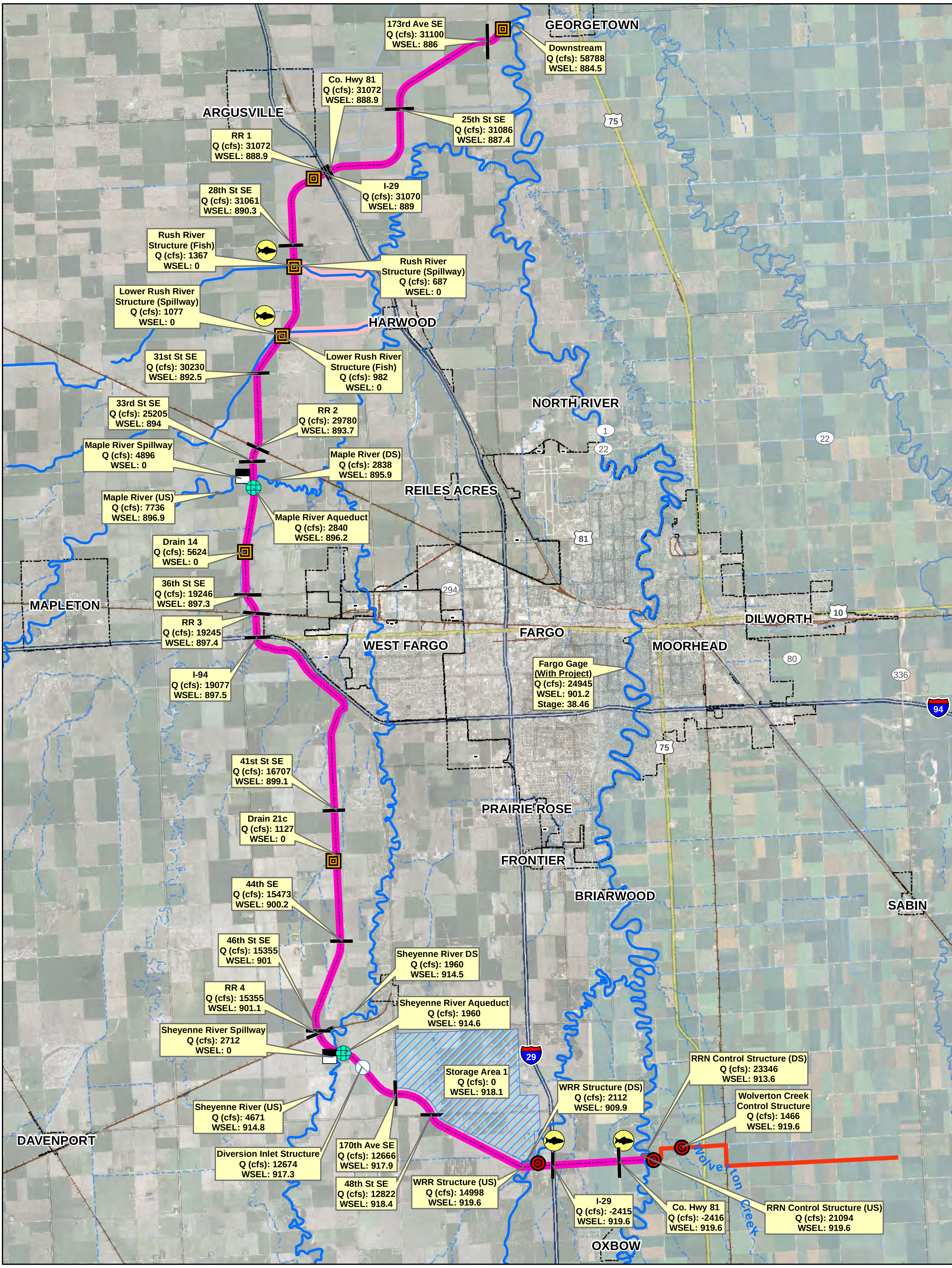
Figure F08

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 10-PERCENT CHANCE EVENT IN RED RIVER OF THE NORTH (AND COINCIDENTAL EVENT IN ND TRIBUTARIES)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F09 -Flow and Water Elevation - 500 Year Event On Tribs.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

North Dakota Diversion
Locally Preferred Plan (LPP)
ND Tieback Levee
Channel Reclamation Reaches
Bridge Reconstruction
Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

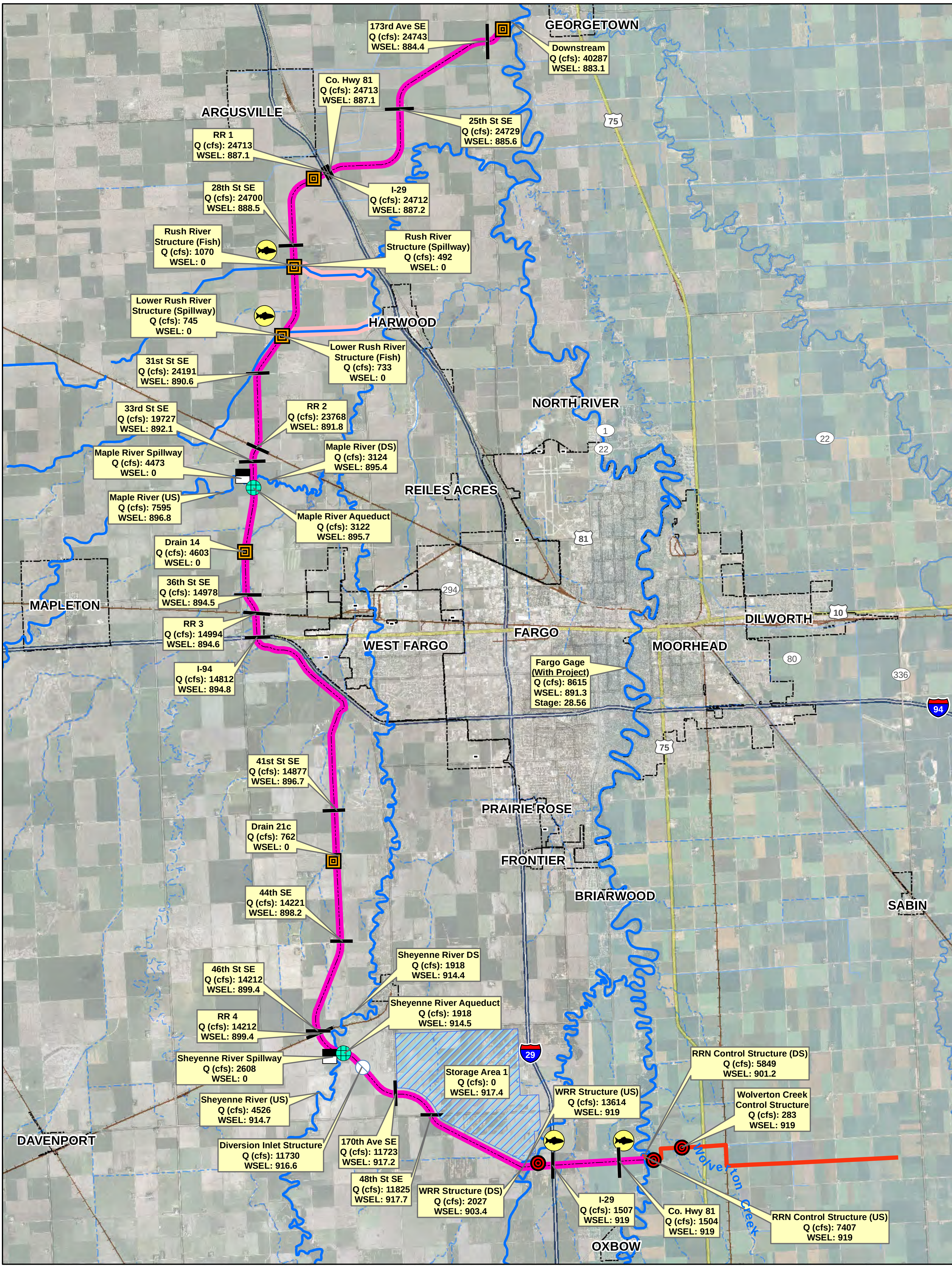
Figure F09

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 0.2-PERCENT CHANCE EVENT IN ND TRIBUTARIES (AND COINCIDENTAL EVENT IN RED RIVER OF THE NORTH)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4
Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F10 -Flow and Water Elevation - 100 Year Event On Tribs.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only. Flows in overbanks/floodplain are not reported.

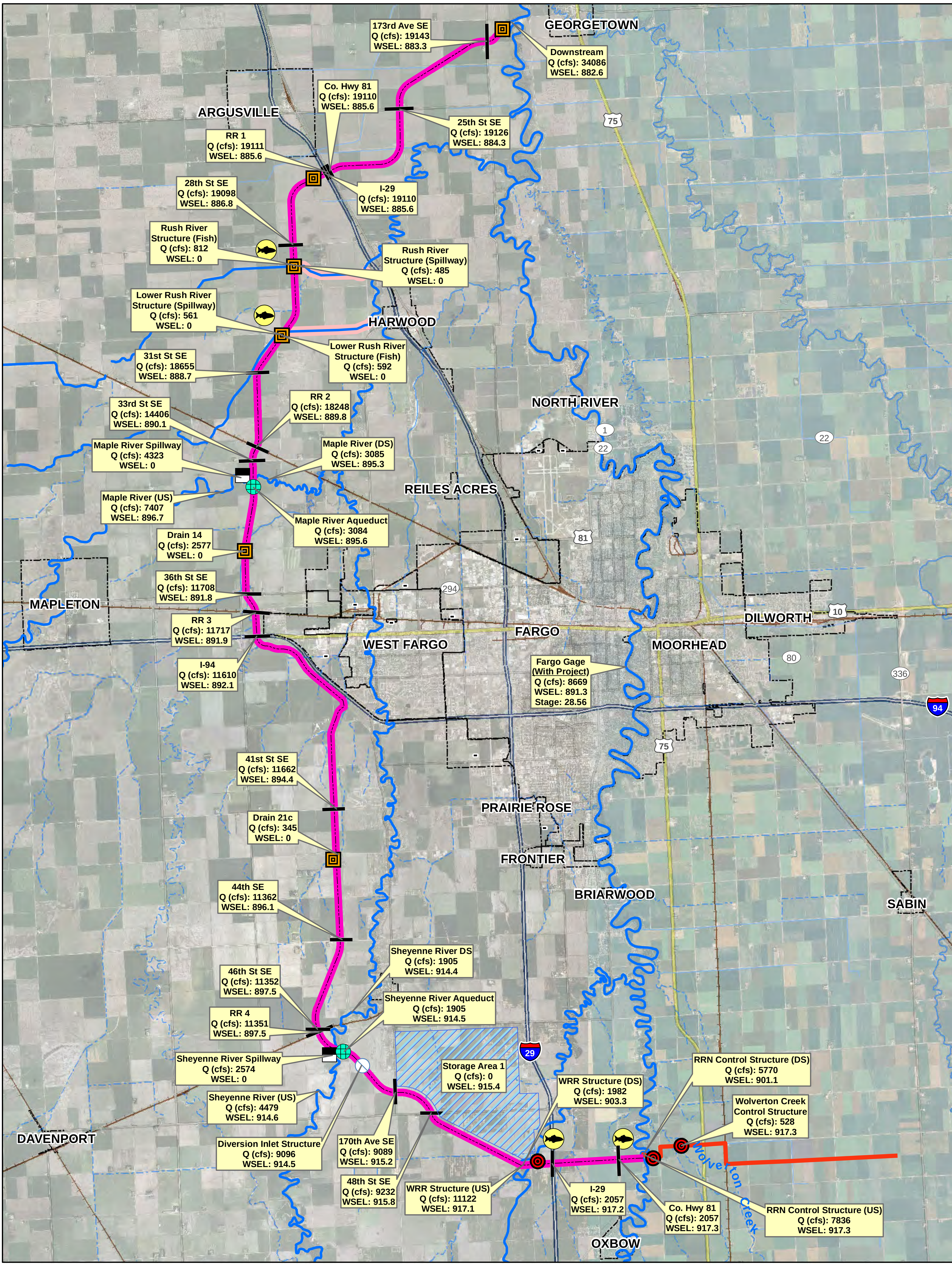
Figure F10

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 1-PERCENT CHANCE EVENT IN ND TRIBUTARIES (AND COINCIDENTAL EVENT IN RED RIVER OF THE NORTH)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F11 -Flow and Water Elevation - 50 Year Event On Tribs.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

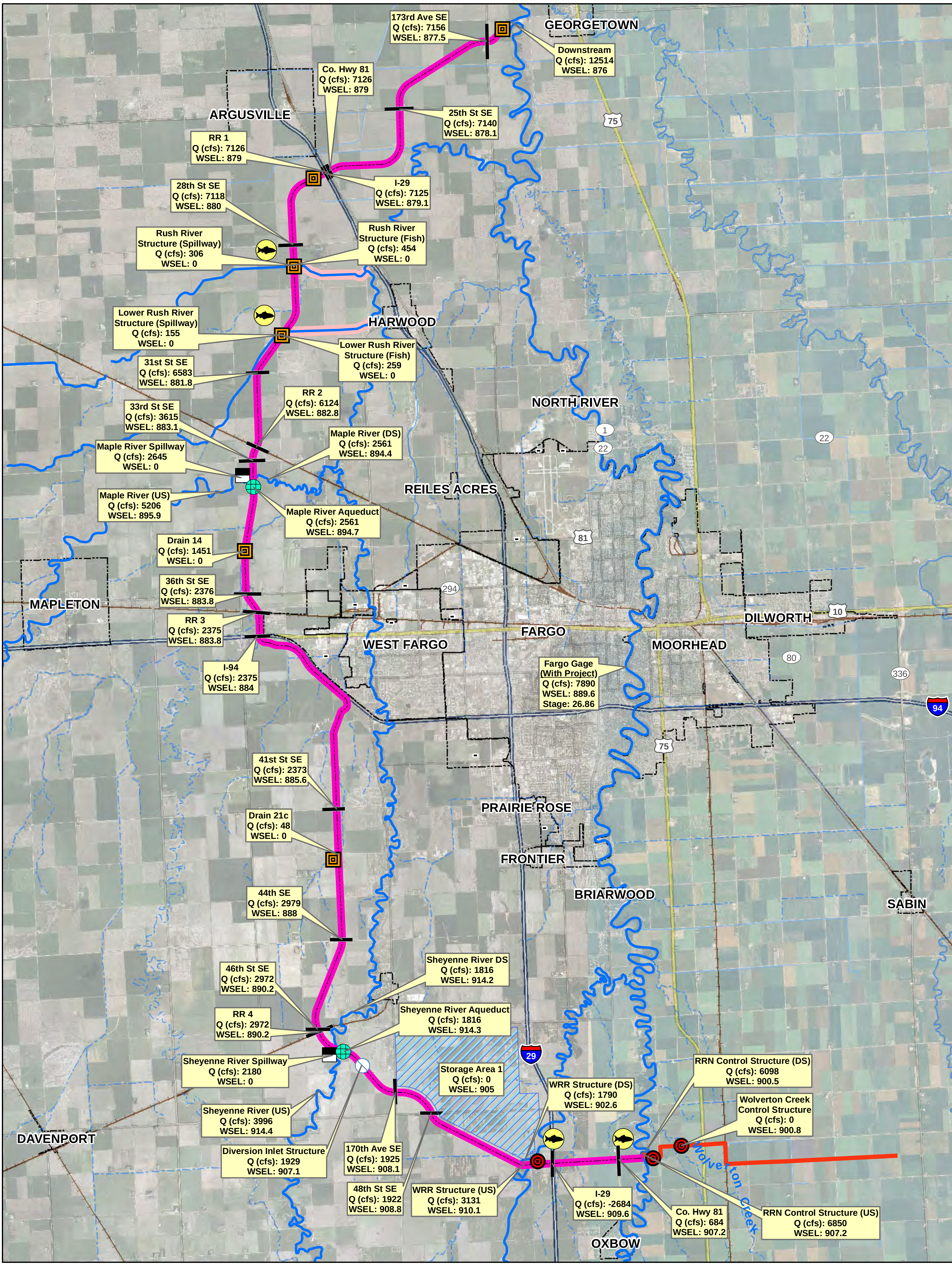
Figure F11

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 2-PERCENT CHANCE EVENT IN ND TRIBUTARIES (AND COINCIDENTAL EVENT IN RED RIVER OF THE NORTH)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F12 -Flow and Water Elevation - 10 Year Event On Tribs.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

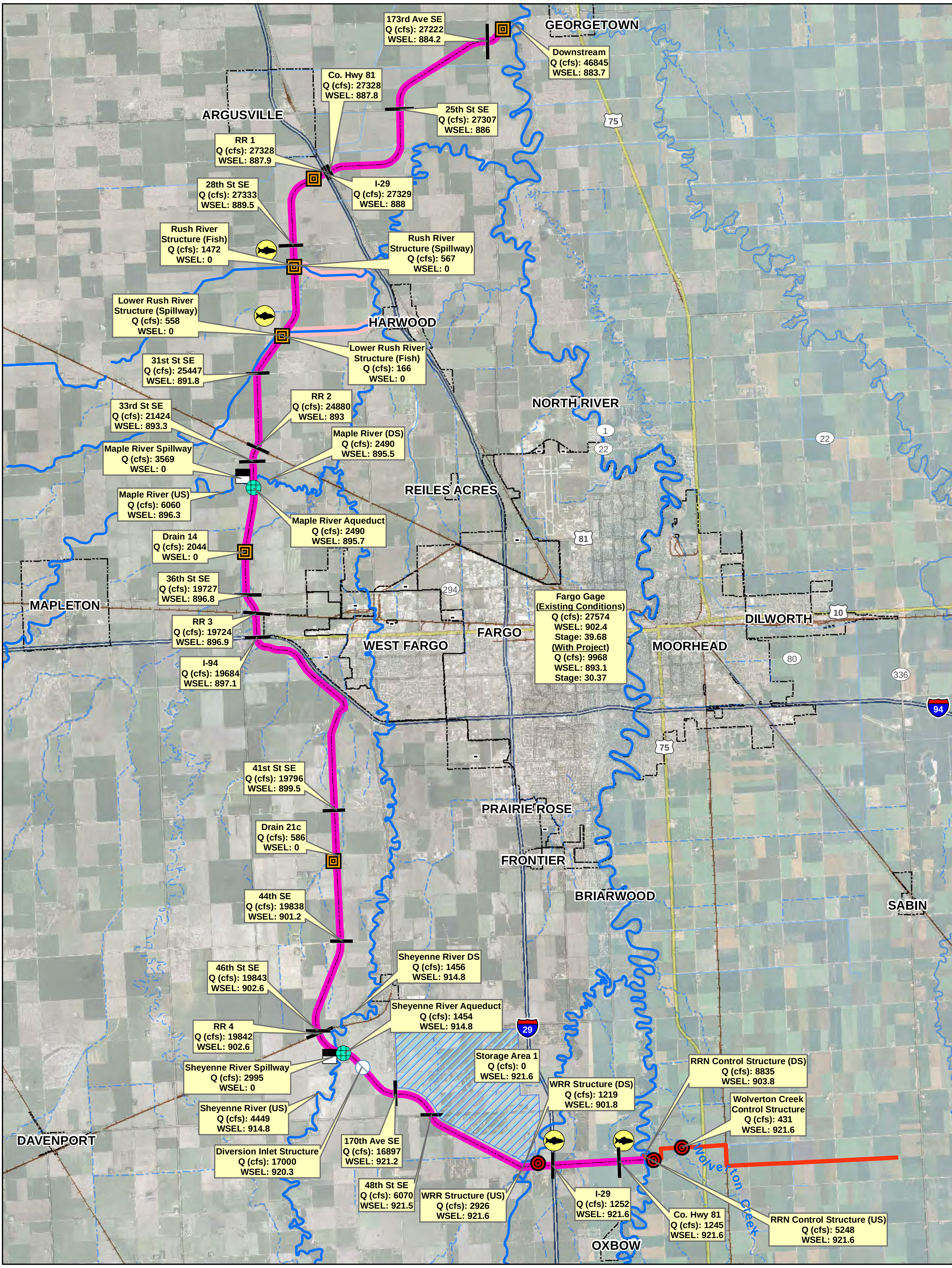
Figure F12

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR 10-PERCENT CHANCE EVENT IN ND TRIBUTARIES (AND COINCIDENTAL EVENT IN RED RIVER OF THE NORTH)
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F13 -Flow and Water Elevation - 1997 Event.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

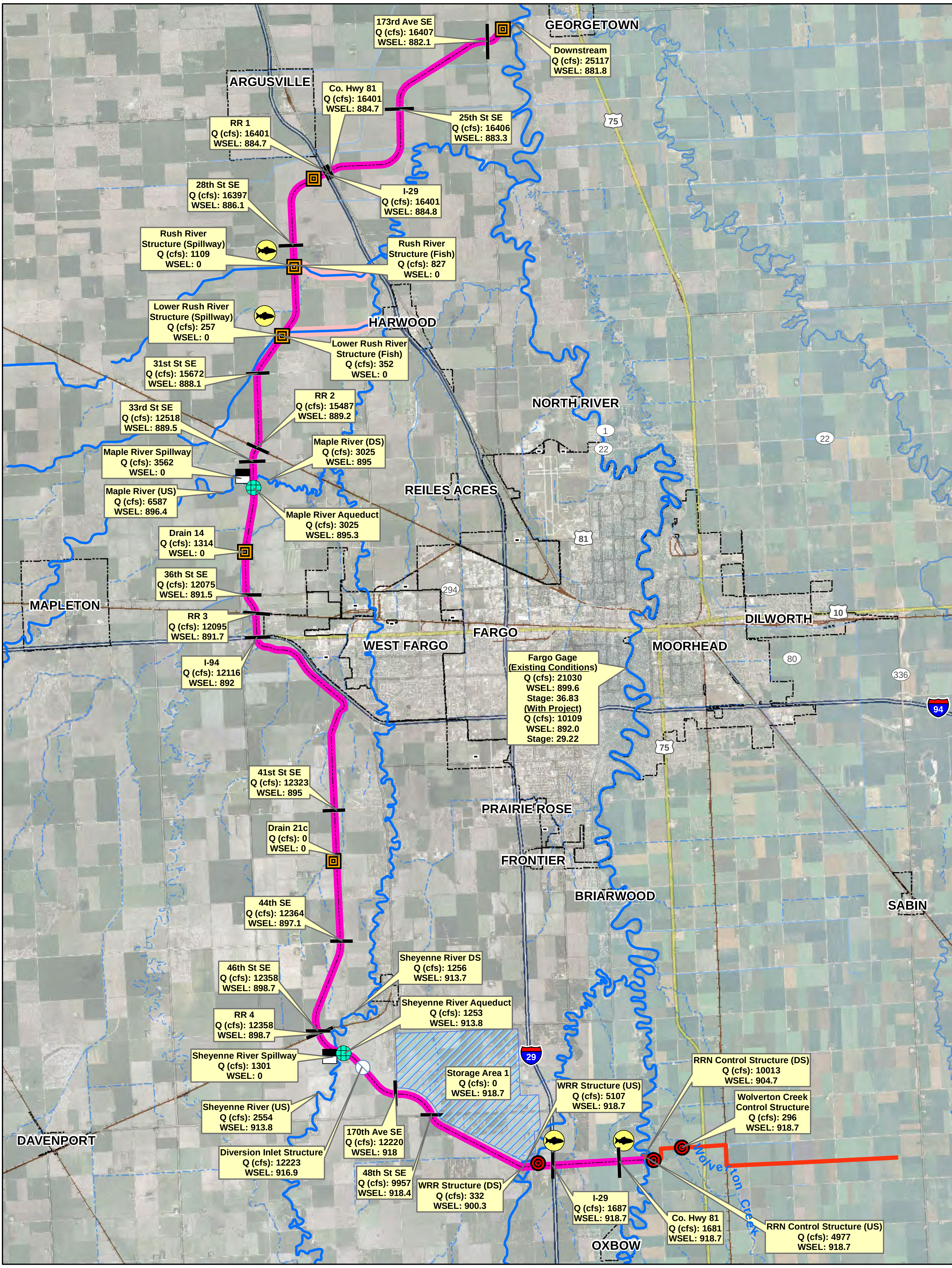
Figure F13

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR THE 1997 FLOOD
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F14 -Flow and Water Elevation - 2006 Event.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

Note: Flows in rivers (US) are in main channel only. Flows in overbanks/floodplain are not reported.

Figure F14

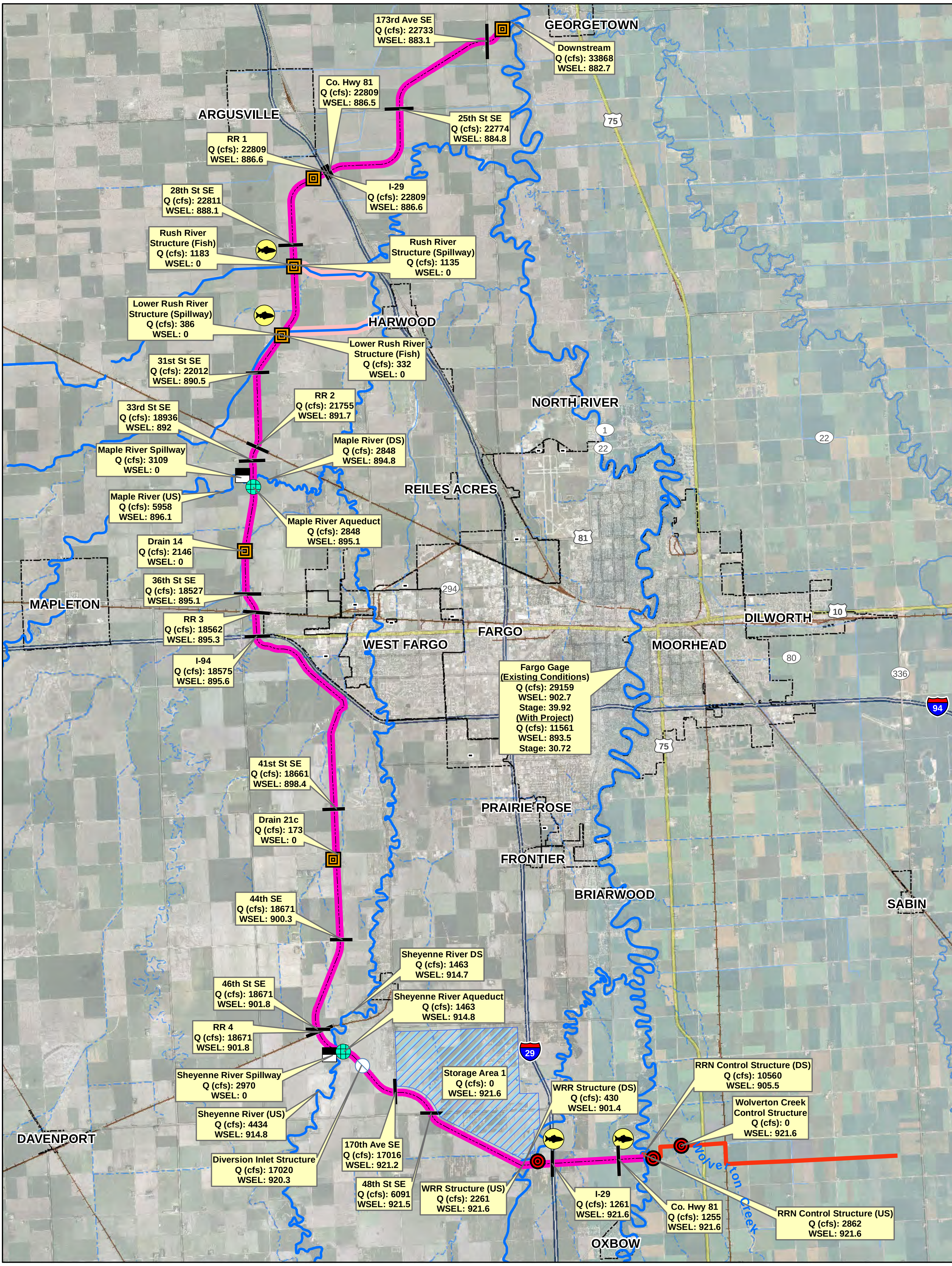
FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR THE 2006 FLOOD

Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F15 -Flow and Water Elevation - 2009 Event.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

North Dakota Diversion Locally Preferred Plan (LPP)

ND Tieback Levee

Channel Reclamation Reaches

Bridge Reconstruction

Storage Area 1

Note: Flows in rivers (US) are in main channel only. Flows in overbanks/floodplain are not reported.

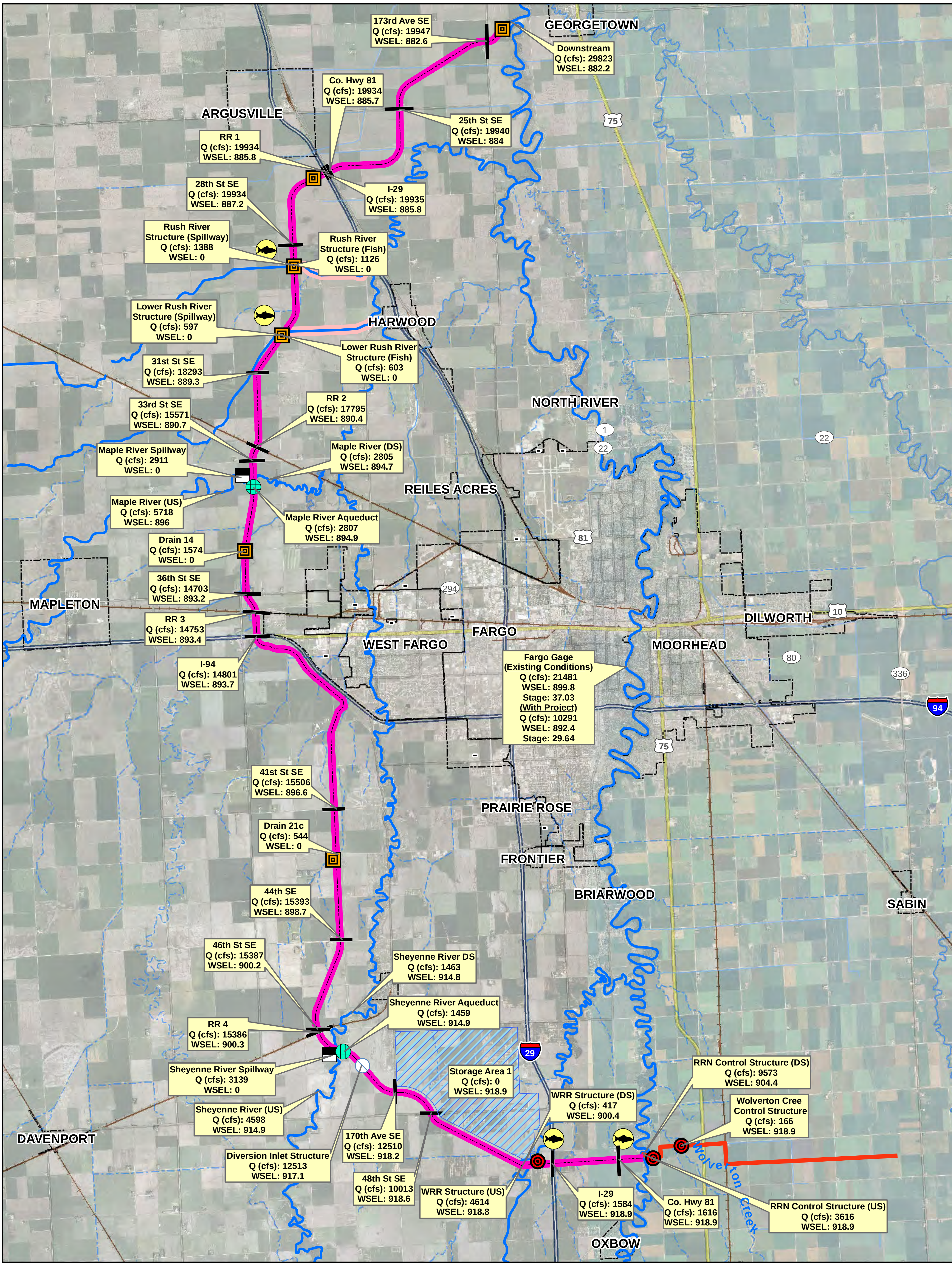
Figure F15

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR THE 2009 FLOOD
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F16 -Flow and Water Elevation - 2010 Event.mxd



Hydraulic Structures

- Weir
- Aqueduct
- Control Structure
- Drop Structure
- Spillway
- Fish Passageway

North Dakota Diversion
Locally Preferred Plan (LPP)

ND Tieback Levee

Channel Reclamation Reaches

Bridge Reconstruction

Storage Area 1

Note: Flows in rivers (US) are in main channel only.
Flows in overbanks/floodplain are not reported.

Figure F16

FLOWS AND WATER SURFACE ELEVATIONS AT MAIN PROJECT FEATURES FOR THE 2010 FLOOD
Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

I:\Projects\34\0910041\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F17 - Flows at Red River (FCP).mxd

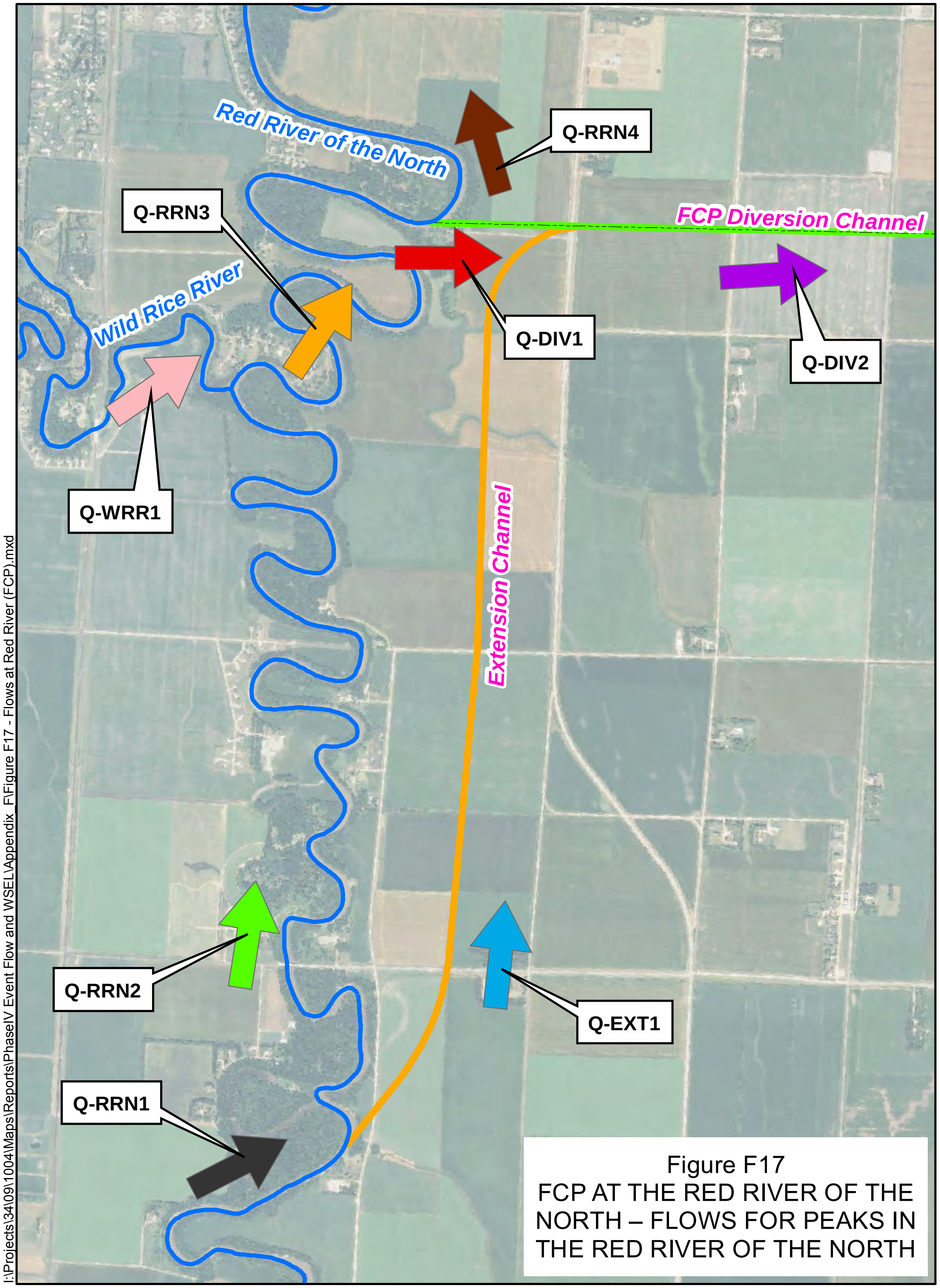


Figure F17
FCP AT THE RED RIVER OF THE NORTH – FLOWS FOR PEAKS IN THE RED RIVER OF THE NORTH

Year 0 Hydrology*

Event Return Interval	Q-RRN1 (cfs)	Q-EXT1 (cfs)	Q-RRN2 (cfs)	Q-WRR1 (cfs)	Q-RRN3 (cfs)	Q-DIV1 (cfs)	Q-DIV2 (cfs)	Q-RRN4 (cfs)
2-year	4,166	0	4,166	1,434	5,600	0	0	5,600
5-year	8,952	867	8,085	3,198	11,283	1,642	2,510	9,640
10-year	10,791	2,667	8,124	6,209	14,333	4,679	7,346	9,654
20-year	13,453	4,278	9,175	8,547	17,722	7,963	12,241	9,759
50-year	17,872	6,319	11,553	11,428	22,981	13,178	19,497	9,803
100-year	21,196	7,679	13,517	13,504	27,021	16,093	23,771	10,929
200-year	30,252	10,576	19,676	15,948	35,624	19,257	29,833	16,367
500-year	42,385	14,602	27,783	19,315	47,098	20,160	34,762	26,938

Year 25 Hydrology*

Event Return Interval	Q-RRN1 (cfs)	Q-EXT1 (cfs)	Q-RRN2 (cfs)	Q-WRR1 (cfs)	Q-RRN3 (cfs)	Q-DIV1 (cfs)	Q-DIV2 (cfs)	Q-RRN4 (cfs)
2-year	3,198	0	3,198	1,154	4,352	0	0	4,352
5-year	7,772	120	7,652	2,836	10,488	847	968	9,640
10-year	9,722	2,014	7,708	5,672	13,380	3,733	5,747	9,647
20-year	12,388	3,754	8,634	7,957	16,591	6,850	10,604	9,741
50-year	16,680	5,797	10,883	10,761	21,644	11,799	17,596	9,845
100-year	20,055	7,230	12,825	12,866	25,691	15,772	23,003	9,918
200-year	27,549	9,640	17,909	14,693	32,602	18,393	28,033	14,209
500-year	39,492	13,598	25,894	18,149	44,043	21,357	34,955	22,686

Year 50 Hydrology*

Event Return Interval	Q-RRN1 (cfs)	Q-EXT1 (cfs)	Q-RRN2 (cfs)	Q-WRR1 (cfs)	Q-RRN3 (cfs)	Q-DIV1 (cfs)	Q-DIV2 (cfs)	Q-RRN4 (cfs)
2-year	2,548	0	2,548	958	3,506	0	0	3,506
5-year	6,358	0	6,358	2,803	9,161	0	0	9,161
10-year	8,774	1,456	7,318	5,191	12,509	2,963	4,419	9,546
20-year	11,434	3,292	8,142	7,421	15,563	5,975	9,267	9,588
50-year	15,609	5,383	10,226	10,155	20,381	10,732	16,115	9,649
100-year	19,019	6,853	12,166	12,285	24,451	14,275	21,128	10,176
200-year	25,243	8,858	16,385	13,544	29,929	17,859	26,716	12,071
500-year	36,927	12,851	24,076	17,107	41,183	21,578	34,429	19,605

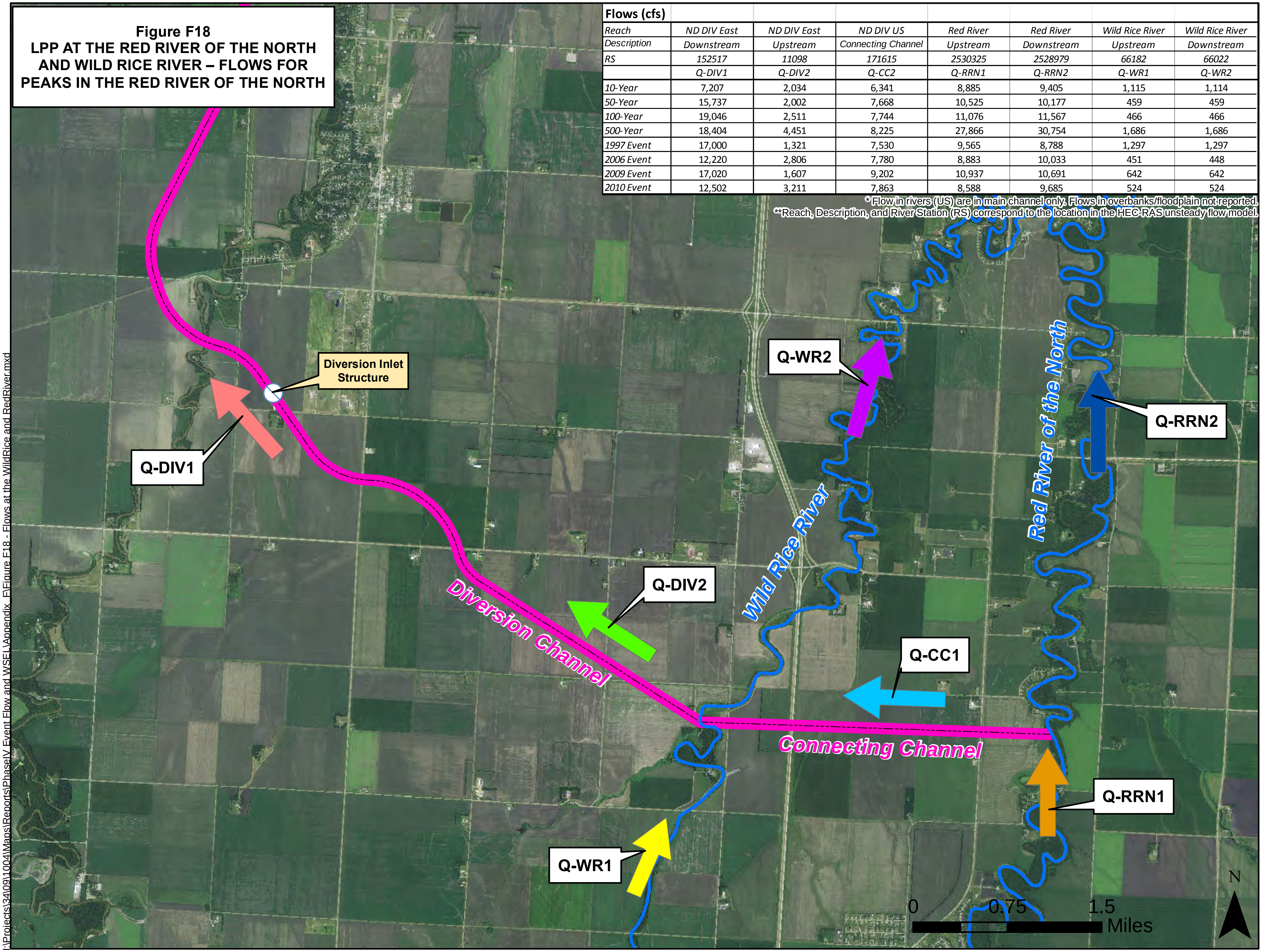
* Flows from Phase 3 HEC-RAS steady flow model

Figure F18
LPP AT THE RED RIVER OF THE NORTH AND WILD RICE RIVER – FLOWS FOR PEAKS IN THE RED RIVER OF THE NORTH

Flows (cfs)

Reach	ND DIV East	ND DIV East	ND DIV US	Red River	Red River	Wild Rice River	Wild Rice River
Description	Downstream	Upstream	Connecting Channel	Upstream	Downstream	Upstream	Downstream
RS	152517	11098	171615	2530325	2528979	66182	66022
	Q-DIV1	Q-DIV2	Q-CC2	Q-RRN1	Q-RRN2	Q-WR1	Q-WR2
10-Year	7,207	2,034	6,341	8,885	9,405	1,115	1,114
50-Year	15,737	2,002	7,668	10,525	10,177	459	459
100-Year	19,046	2,511	7,744	11,076	11,567	466	466
500-Year	18,404	4,451	8,225	27,866	30,754	1,686	1,686
1997 Event	17,000	1,321	7,530	9,565	8,788	1,297	1,297
2006 Event	12,220	2,806	7,780	8,883	10,033	451	448
2009 Event	17,020	1,607	9,202	10,937	10,691	642	642
2010 Event	12,502	3,211	7,863	8,588	9,685	524	524

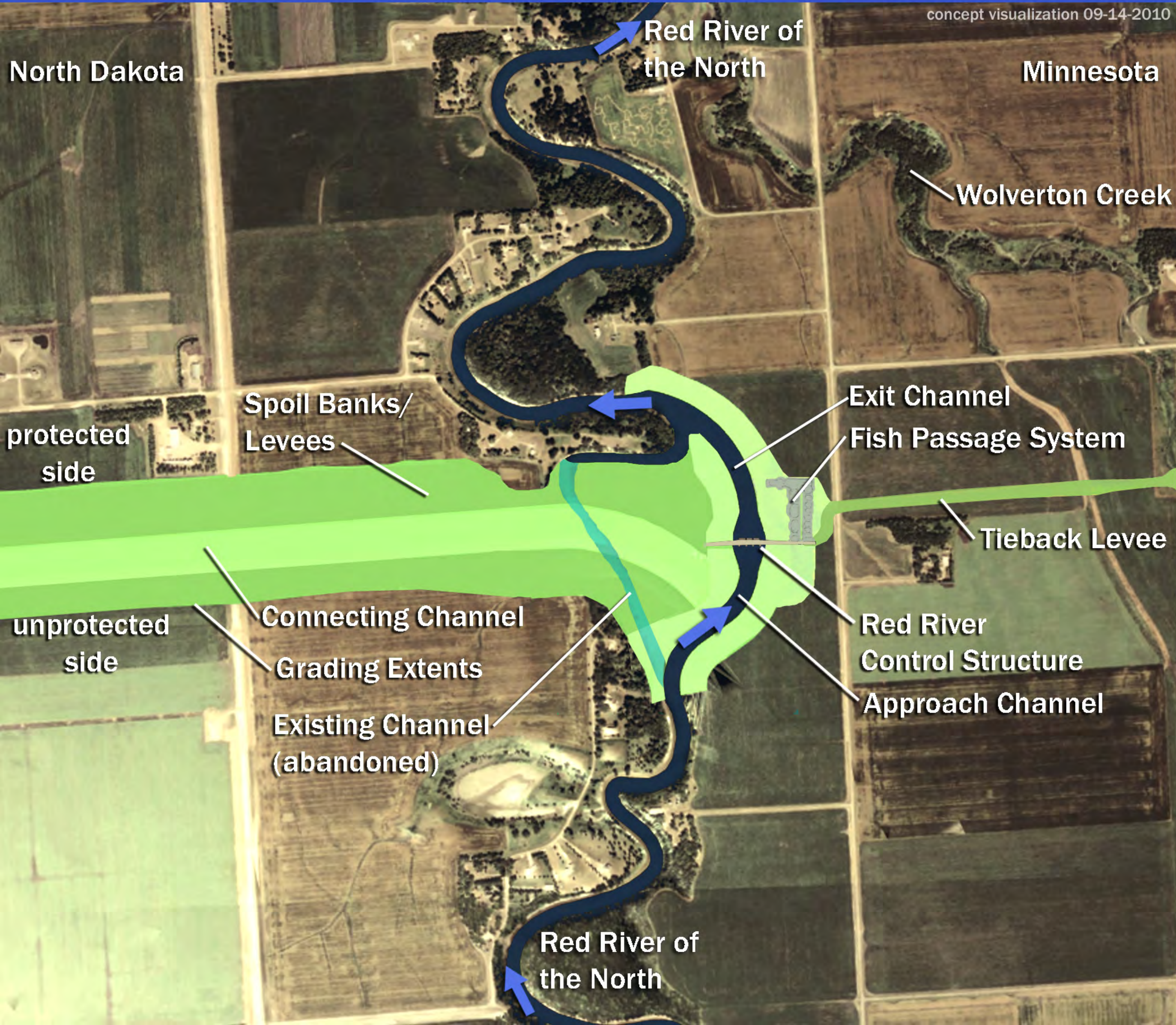
* Flow in rivers (US) are in main channel only. Flows in overbanks/floodplain not reported.
**Reach, Description, and River Station (RS) correspond to the location in the HEC-RAS unsteady flow model.



I:\Projects\34\09\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F18 - Flows at the WildRice and RedRiver.mxd

Red River Control Structure - Flow Scenario 1

BARR



LPP ND East 35K
Flow Scenario 1:
average flow

Figure F19

Red River Control Structure - Flow Scenario 2

LPP ND East 35K
Flow Scenario 2:
2-year event

Figure F20

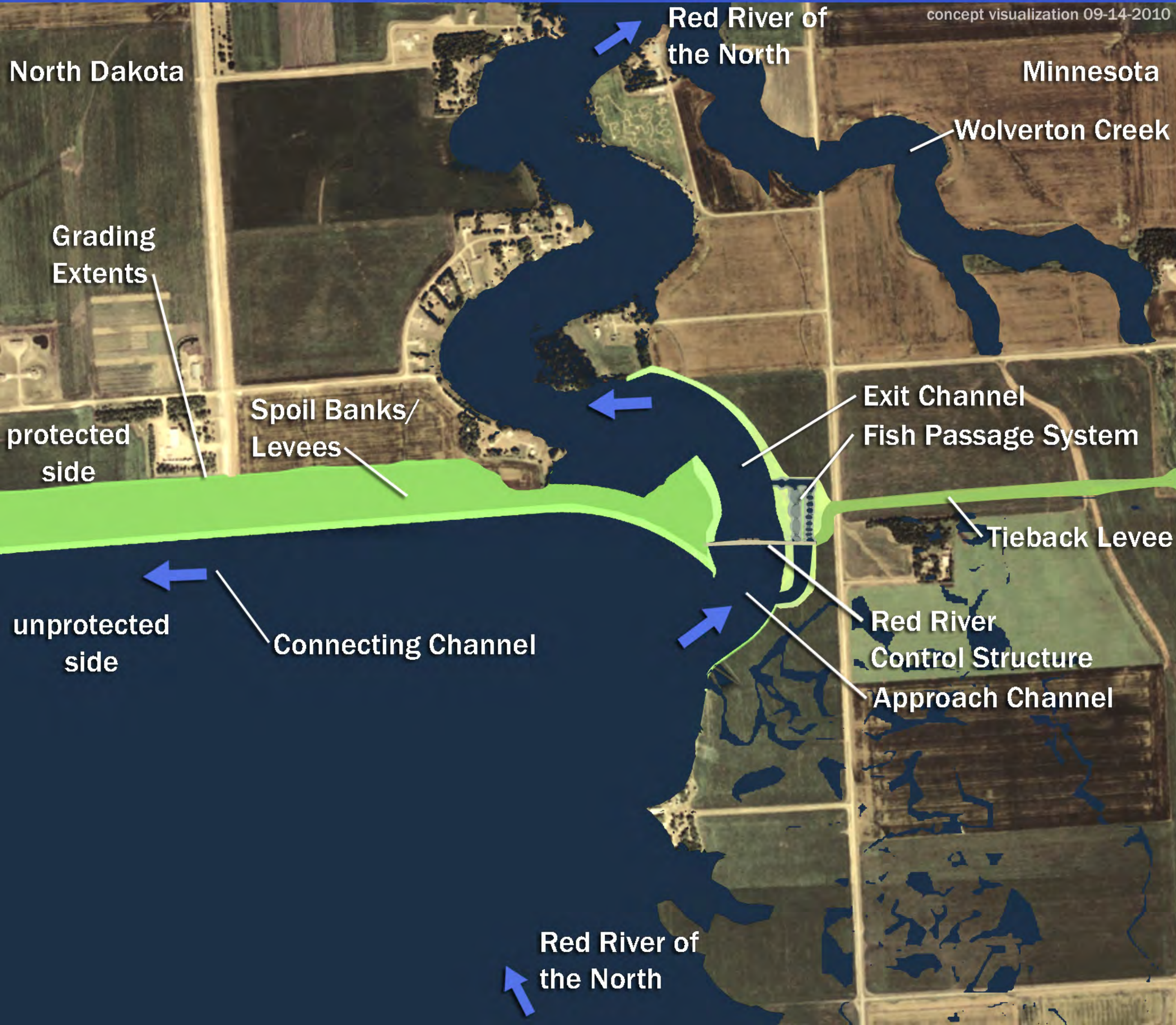
Red River Control Structure - Flow Scenario 3

LPP ND East 35K
Flow Scenario 3:
10-year event

Figure F21

Red River Control Structure - Flow Scenario 4

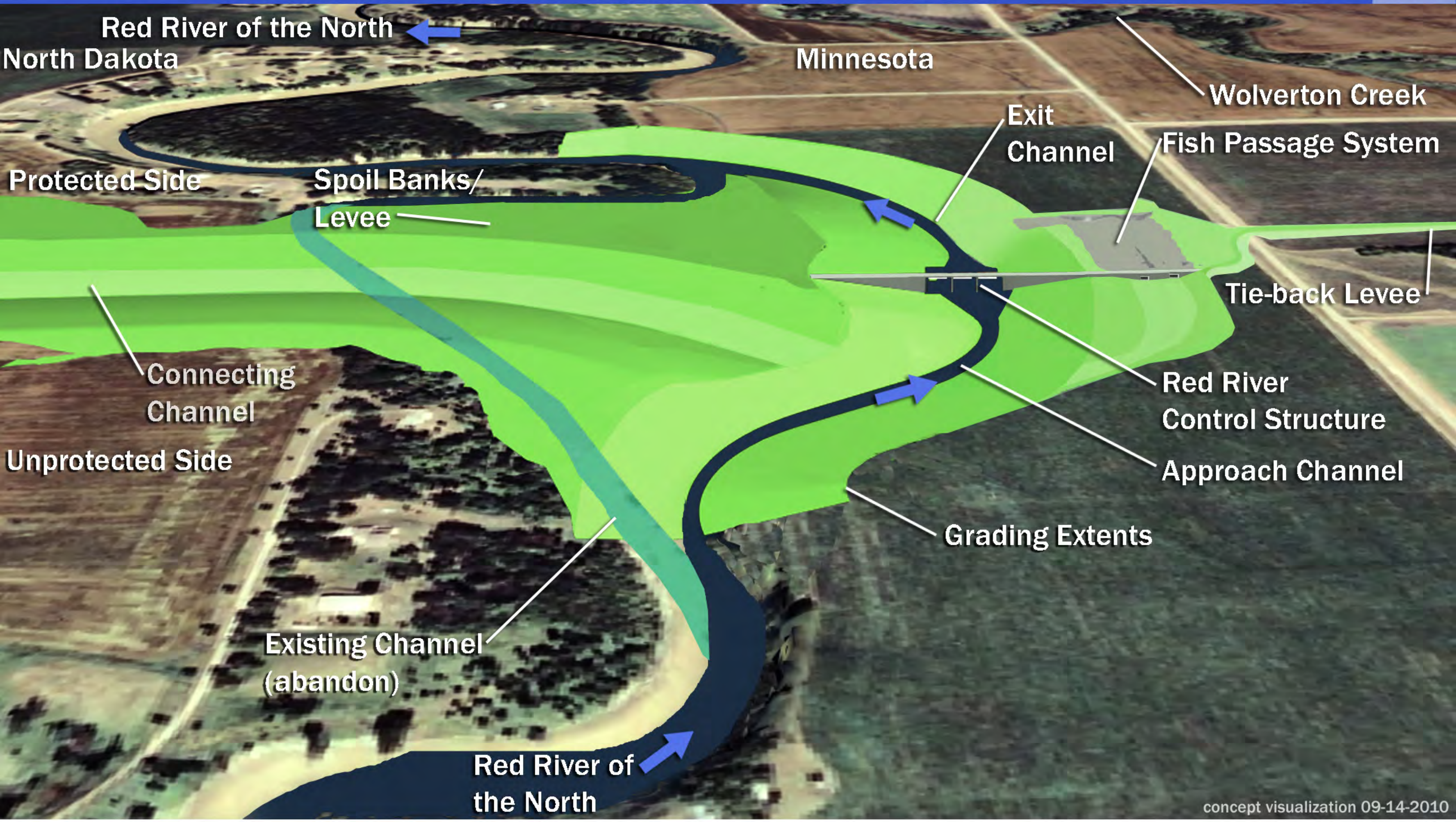
BARR



LPP ND East 35K Flow Scenario 4: 100-year event

Figure F22

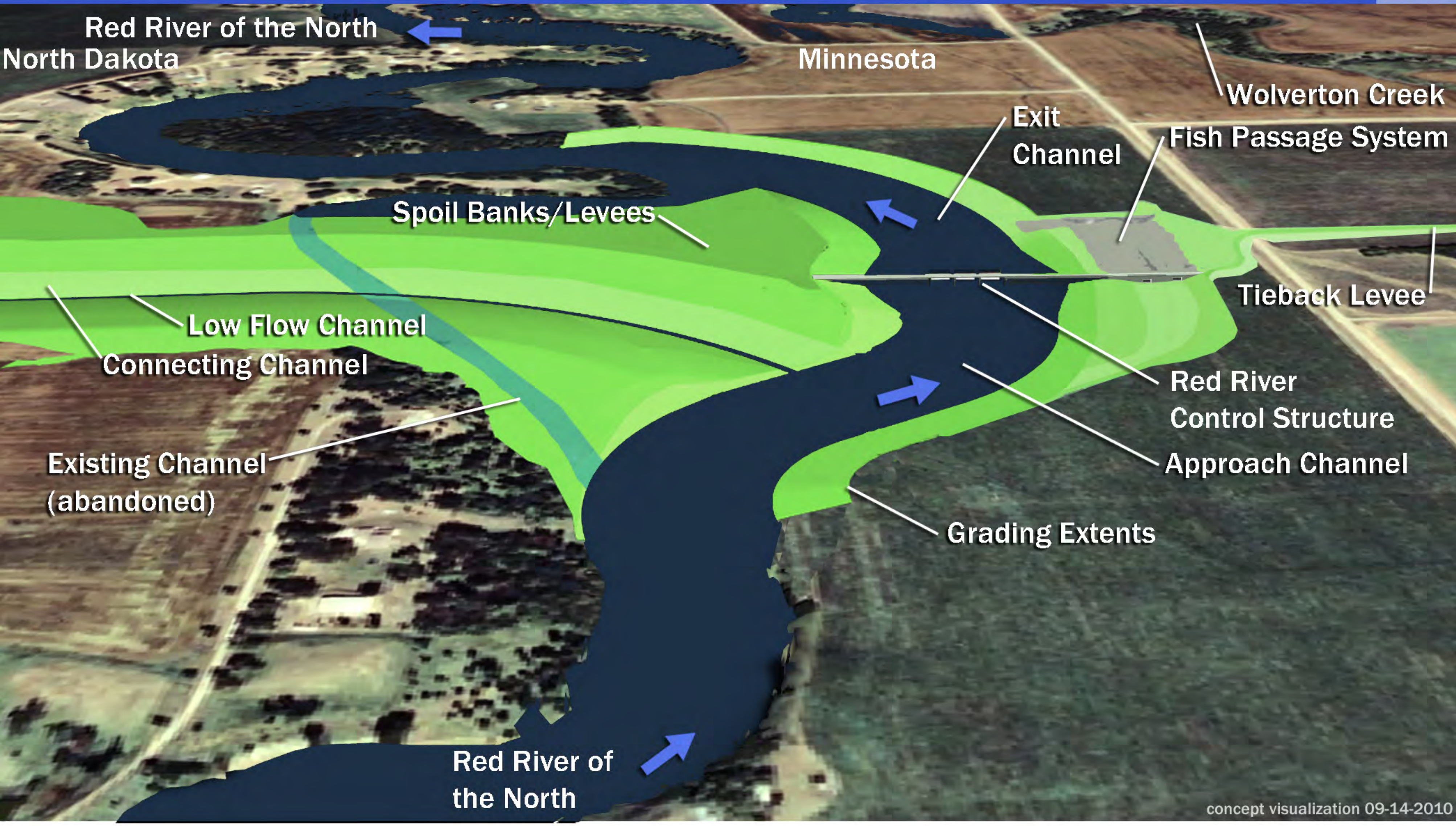
Red River Control Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1:
average flow

Figure F23

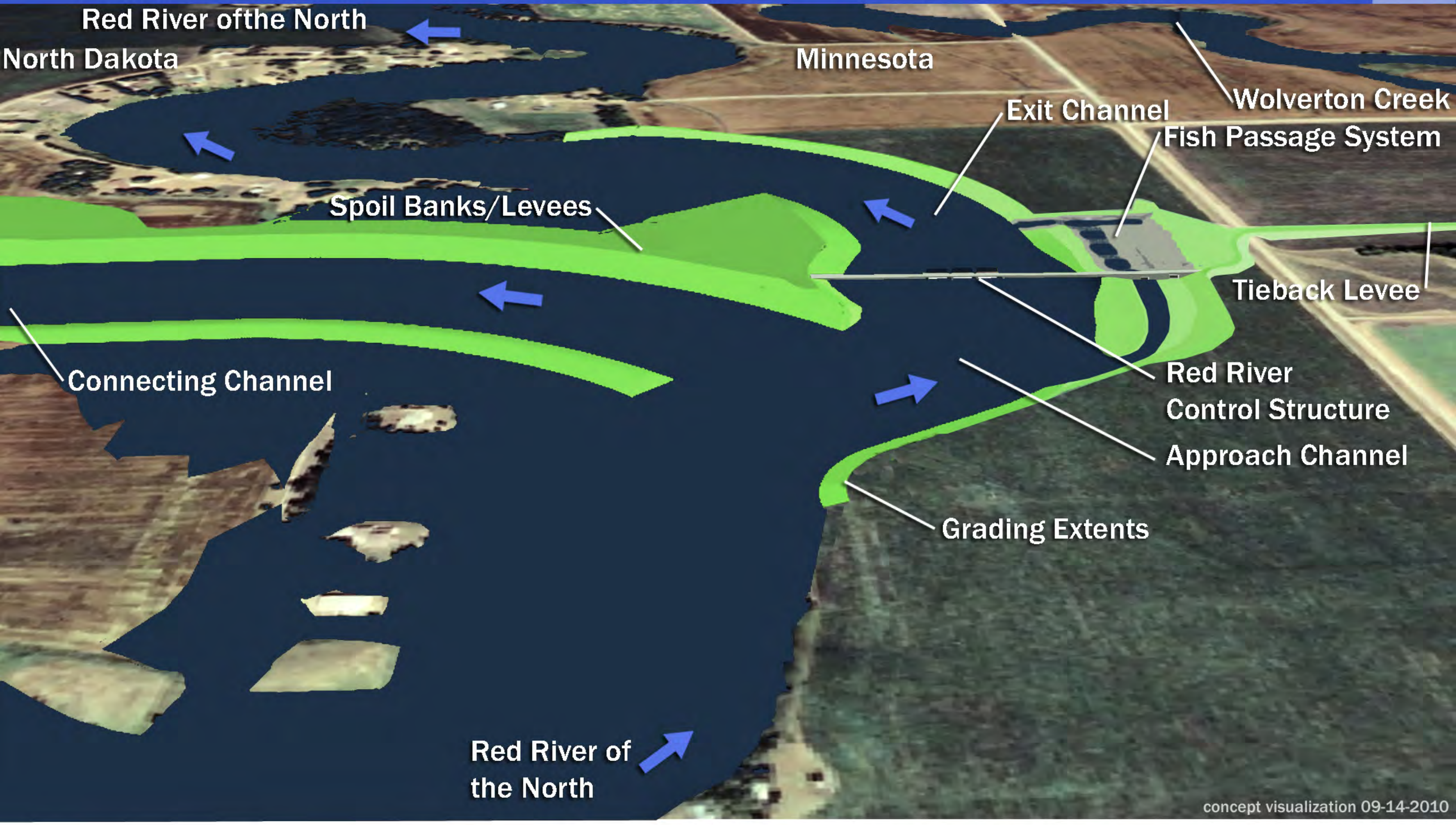
Red River Control Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2:
2-year event

Figure F24

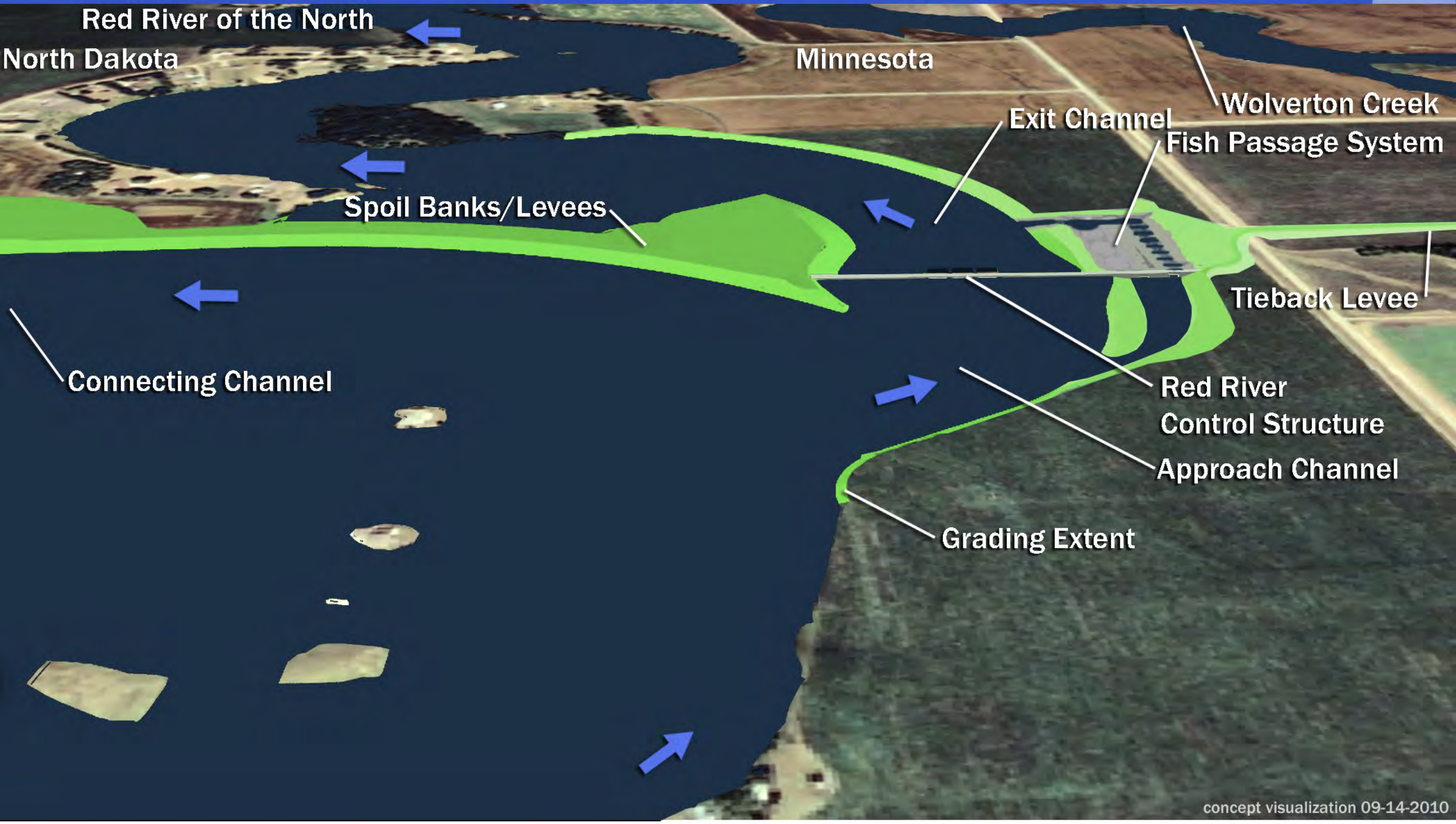
Red River Control Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3:
10-year event

Figure F25

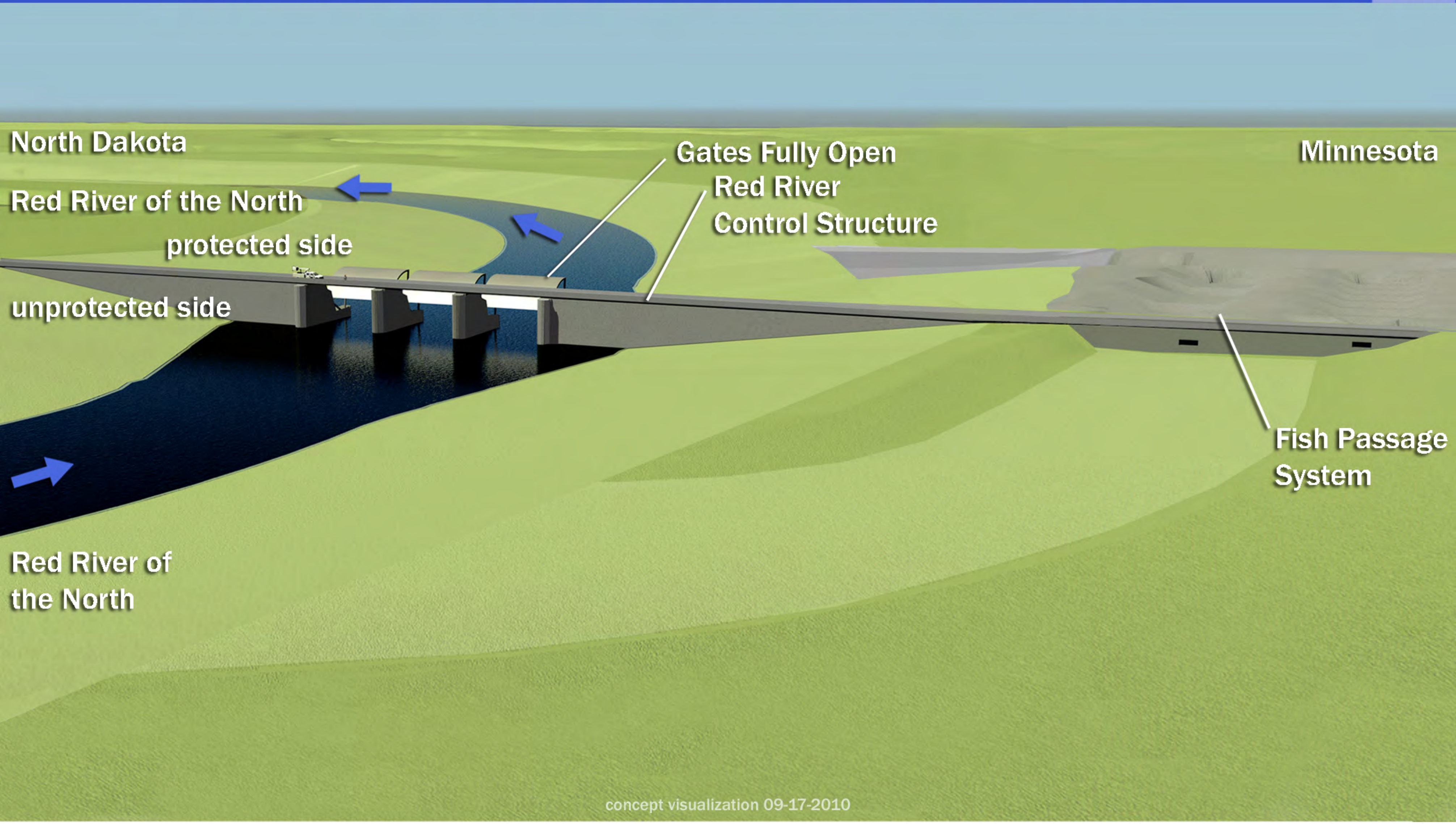
Red River Control Structure - Flow Scenario 4



LPP ND East 35K
Flow Scenario 4:
100-year event

Figure F26

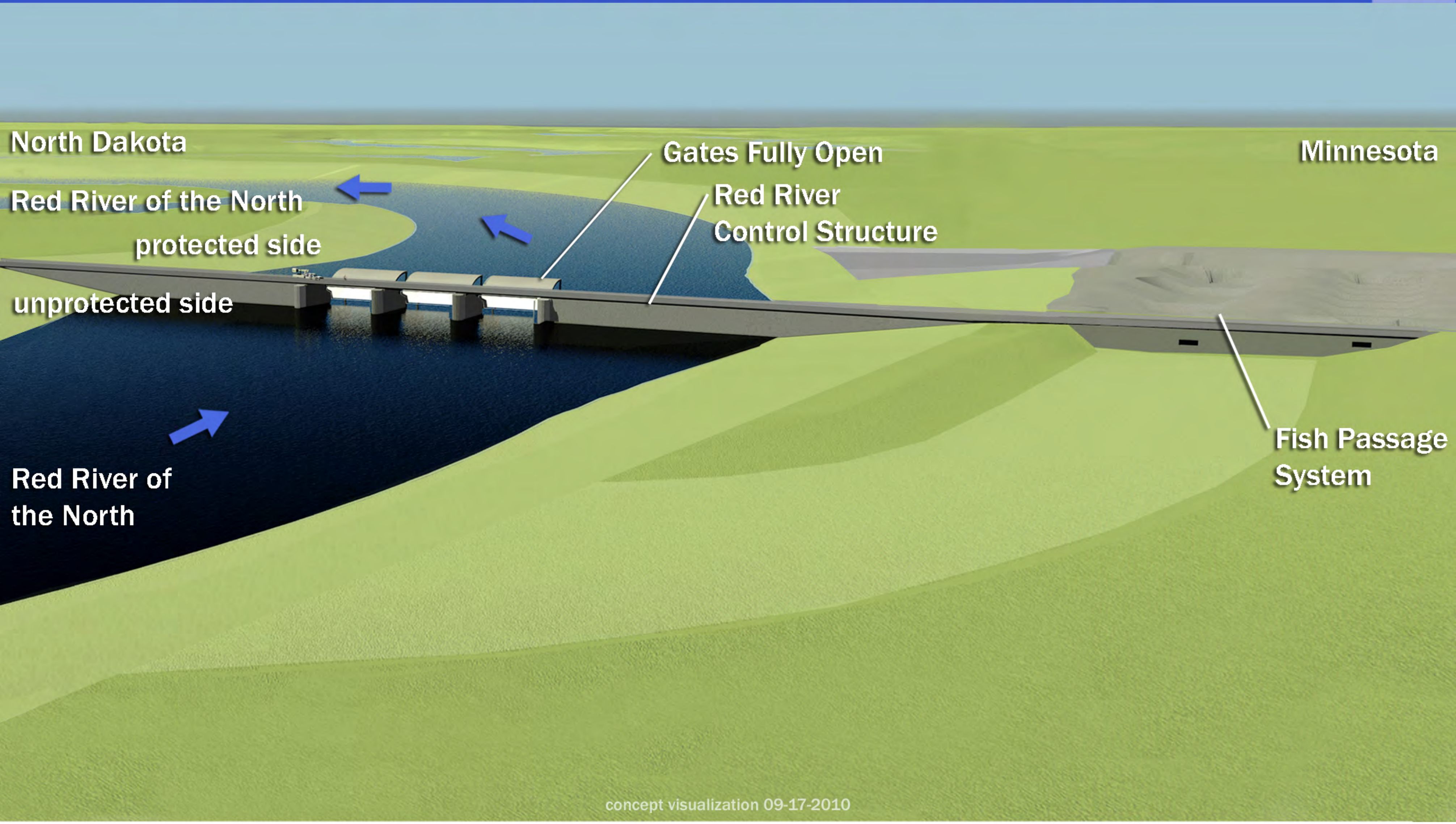
Red River Control Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1
average flow

Figure F27

Red River Control Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2
2-year flow

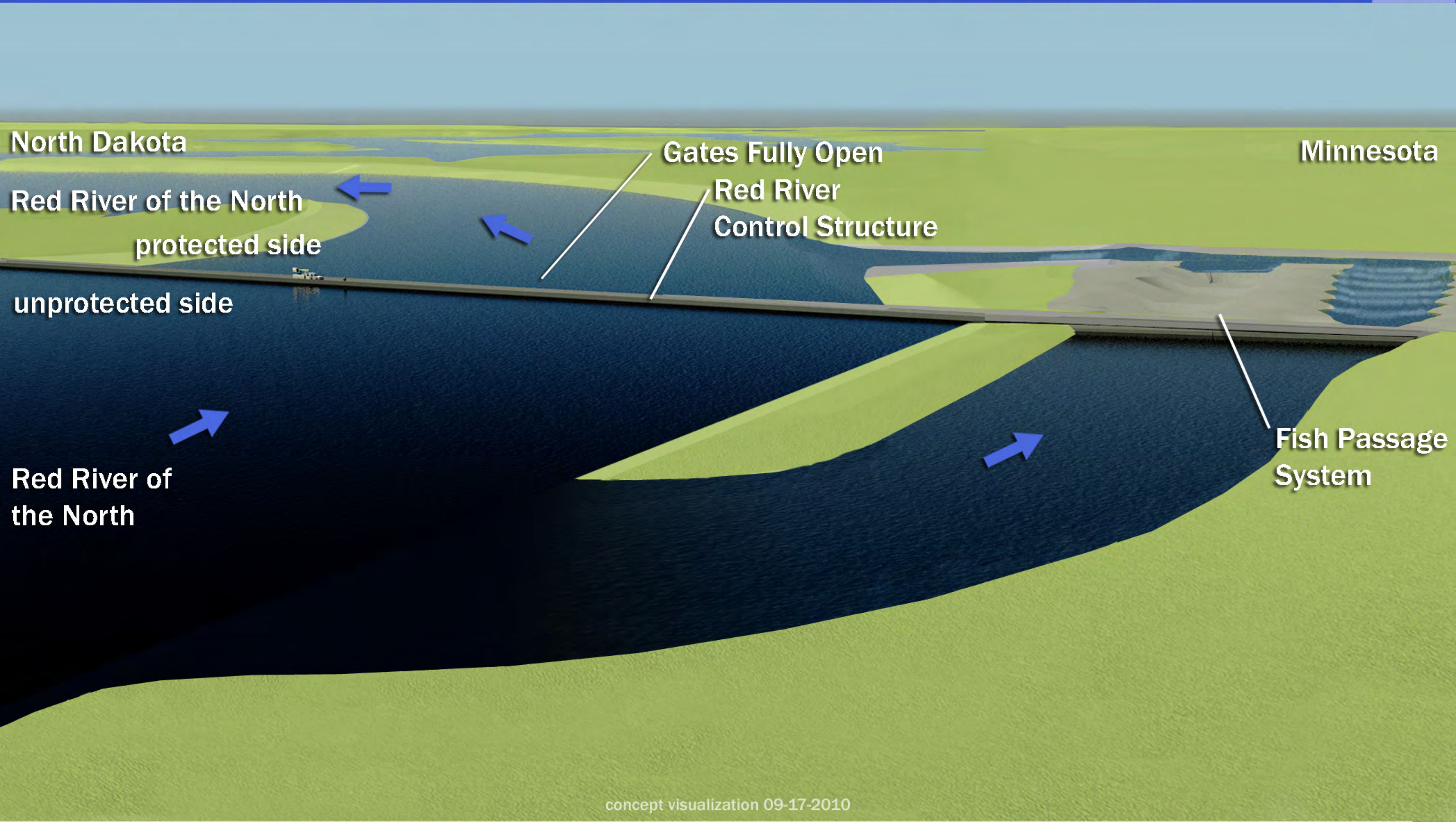
Figure F28

Red River Control Structure - Flow Scenario 3

LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F29

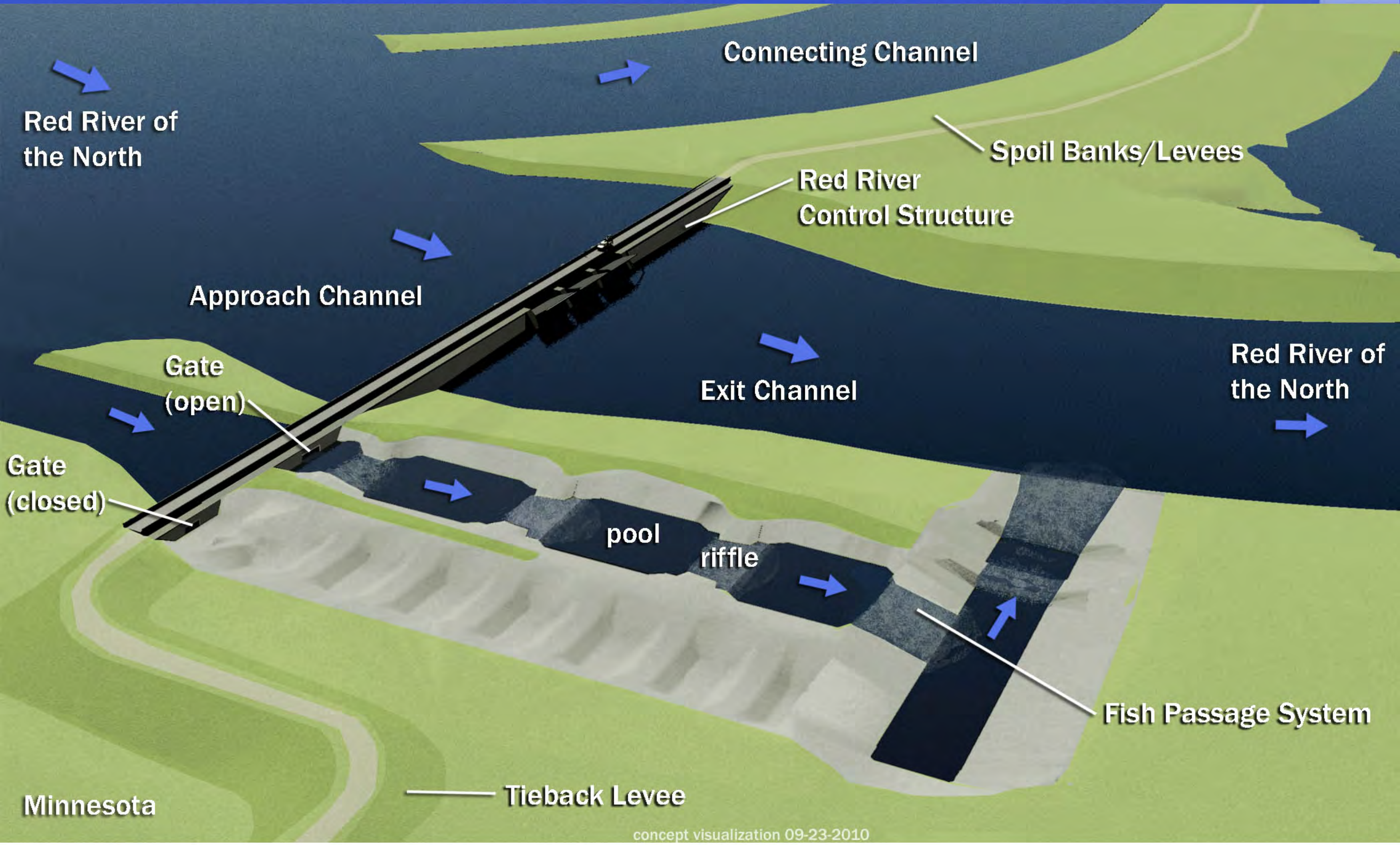
Red River Control Structure - Flow Scenario 4



LPP ND East 35K
Flow Scenario 3
100-year flow

Figure F30

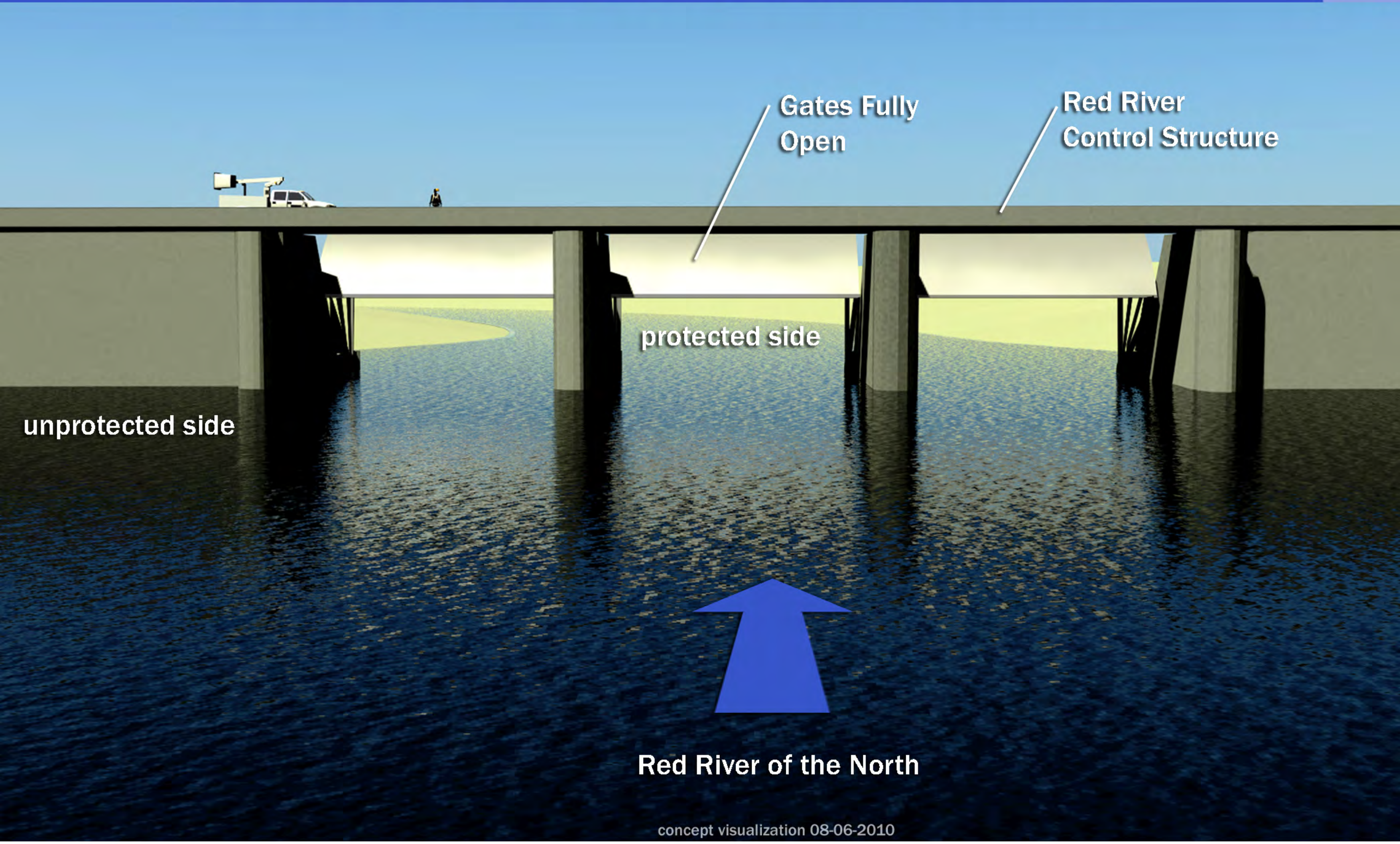
Red River Control Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F31

Red River Control Structure - Flow Scenario 1

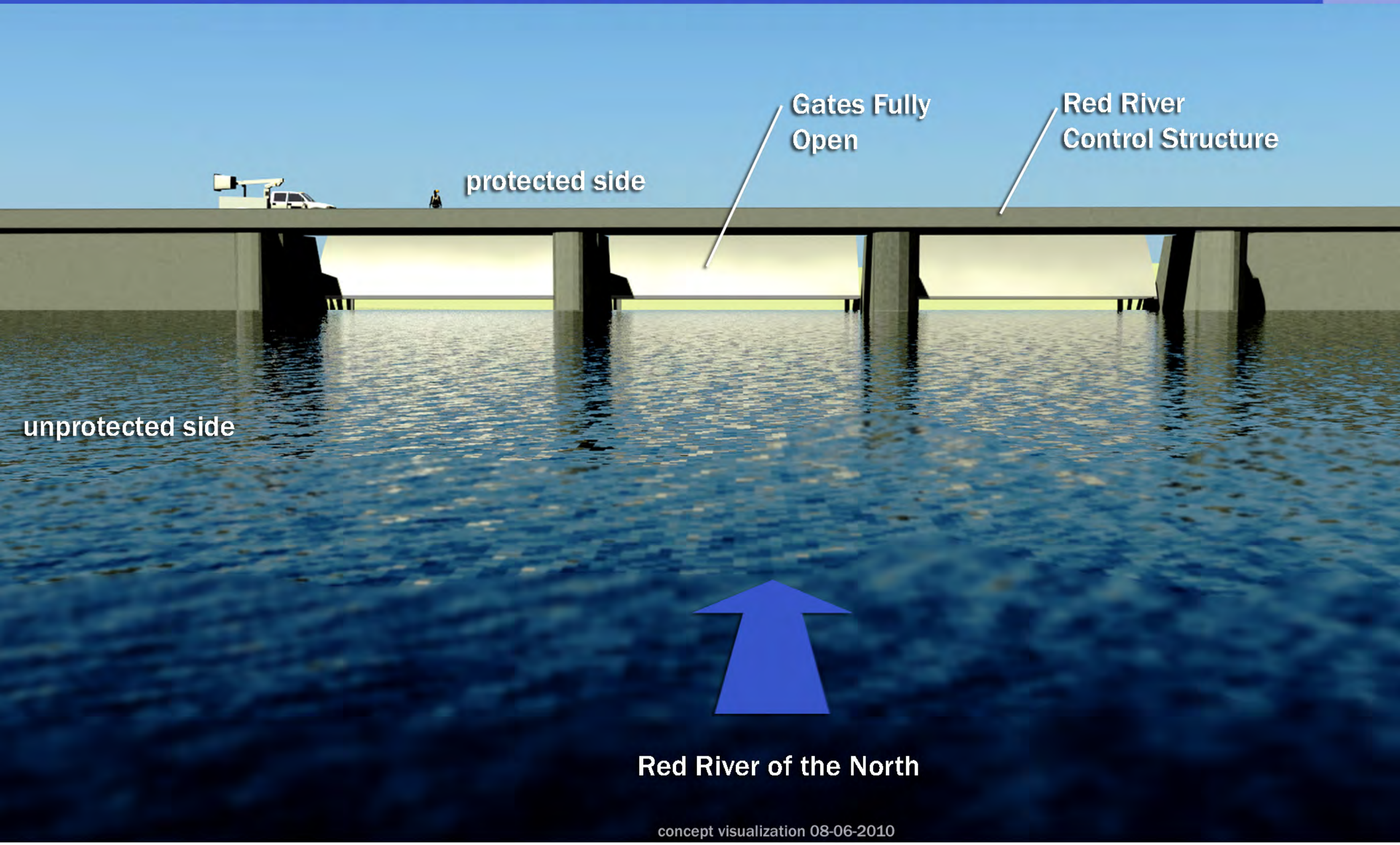


LPP ND East 35K
Flow Scenario 1:
average flow

Figure F32

Red River Control Structure - Flow Scenario 2

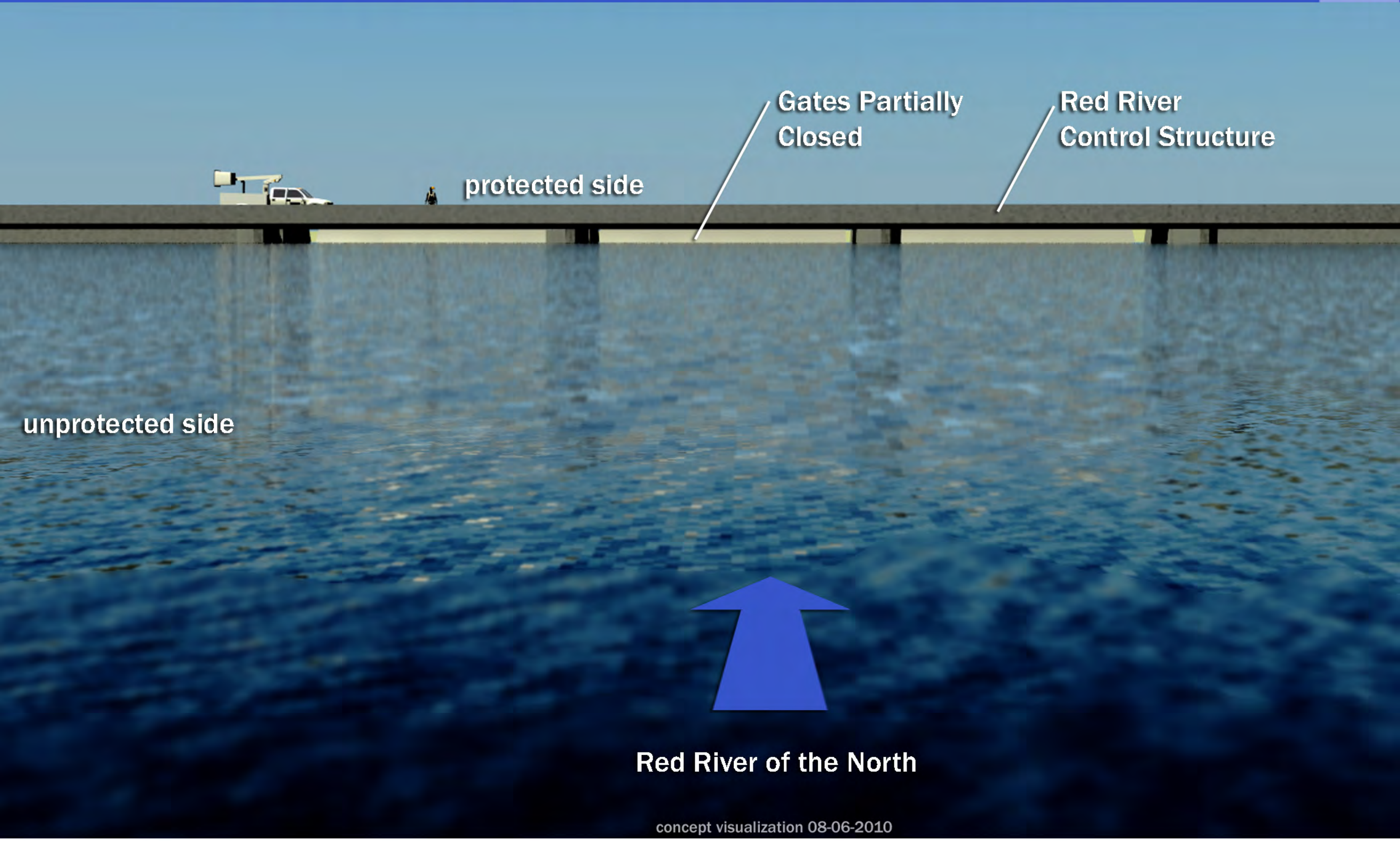
BARR



LPP ND East 35K
Flow Scenario 2
2-year flow

Figure F33

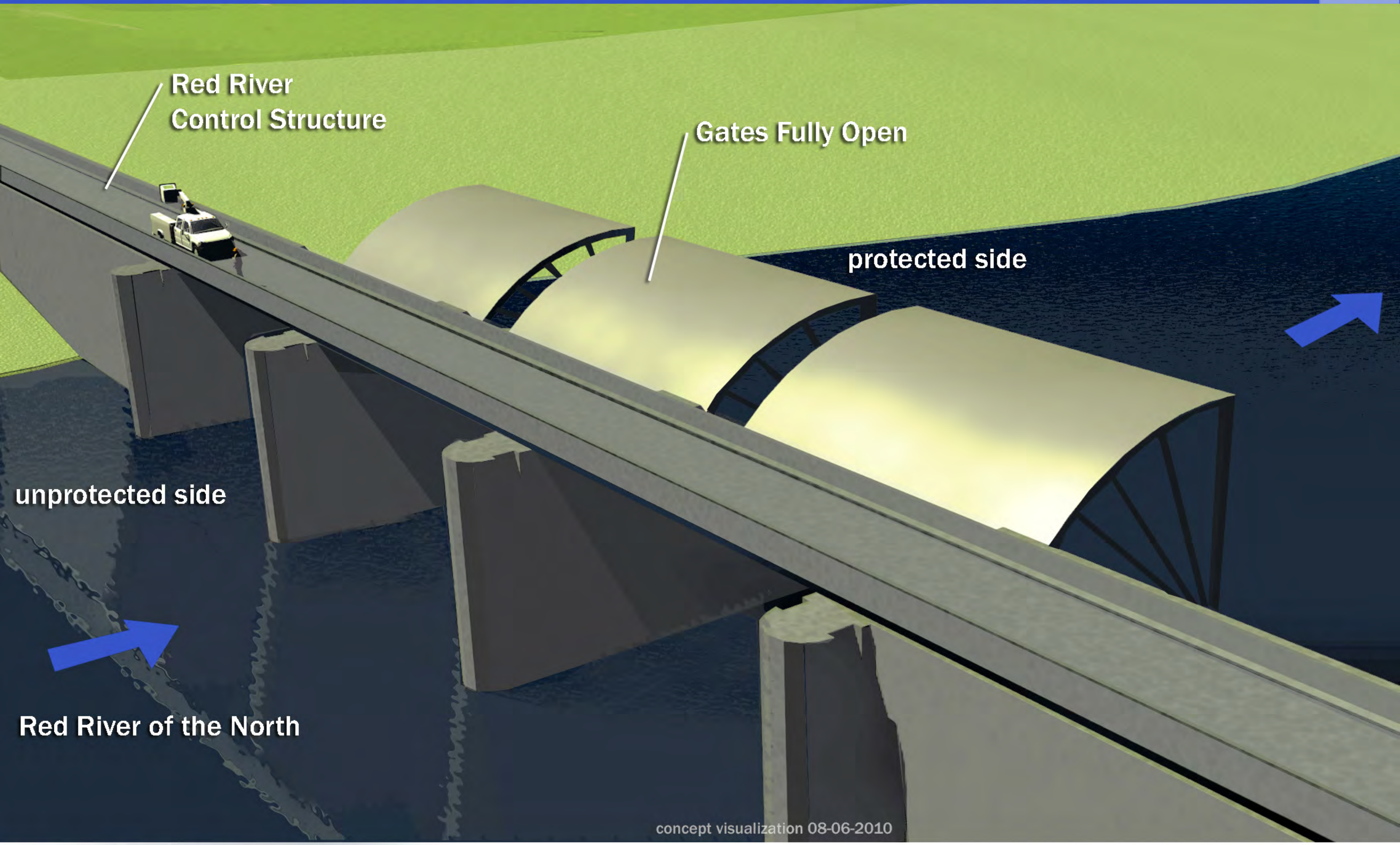
Red River Control Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F34

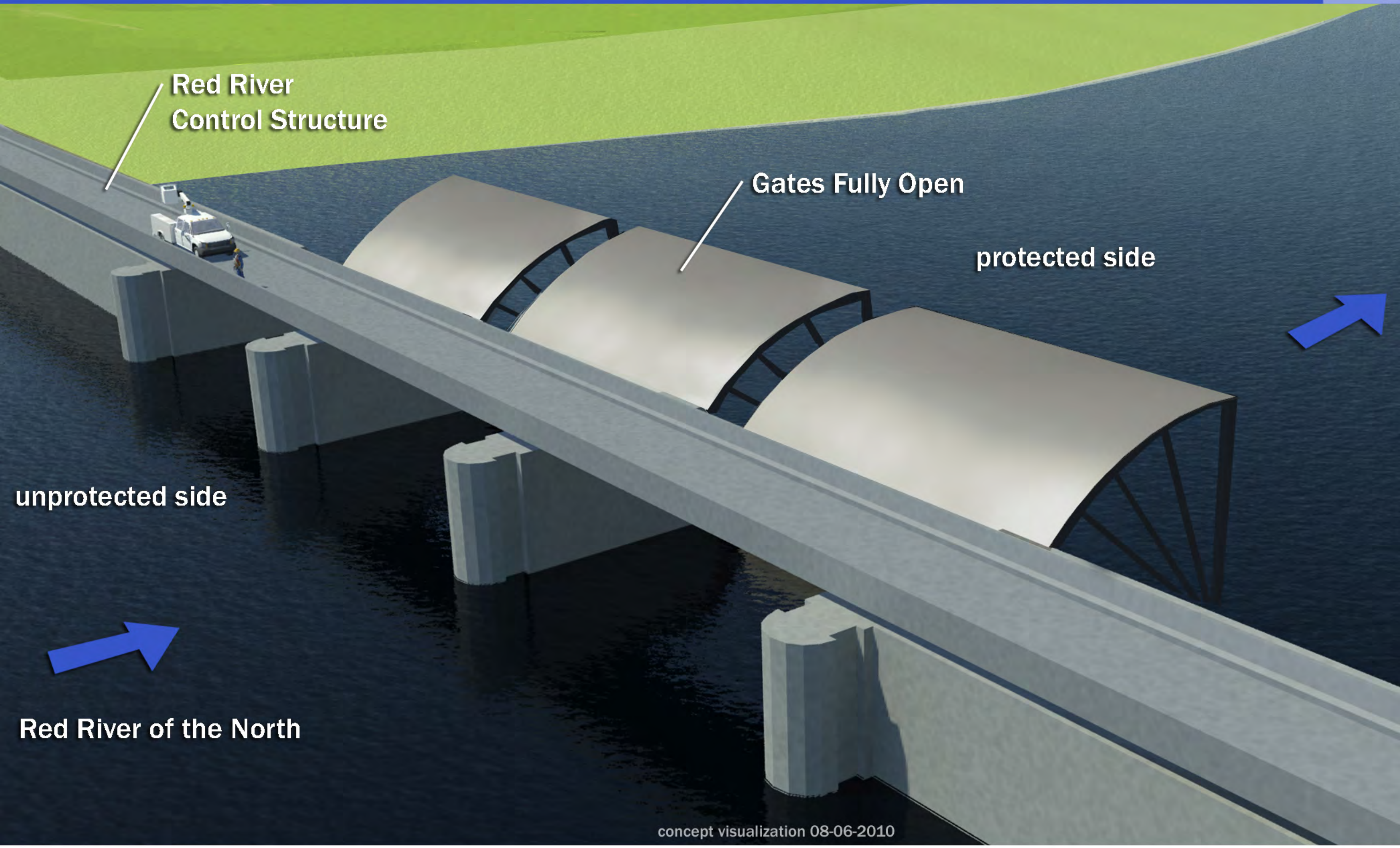
Red River Control Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1:
average flow

Figure F35

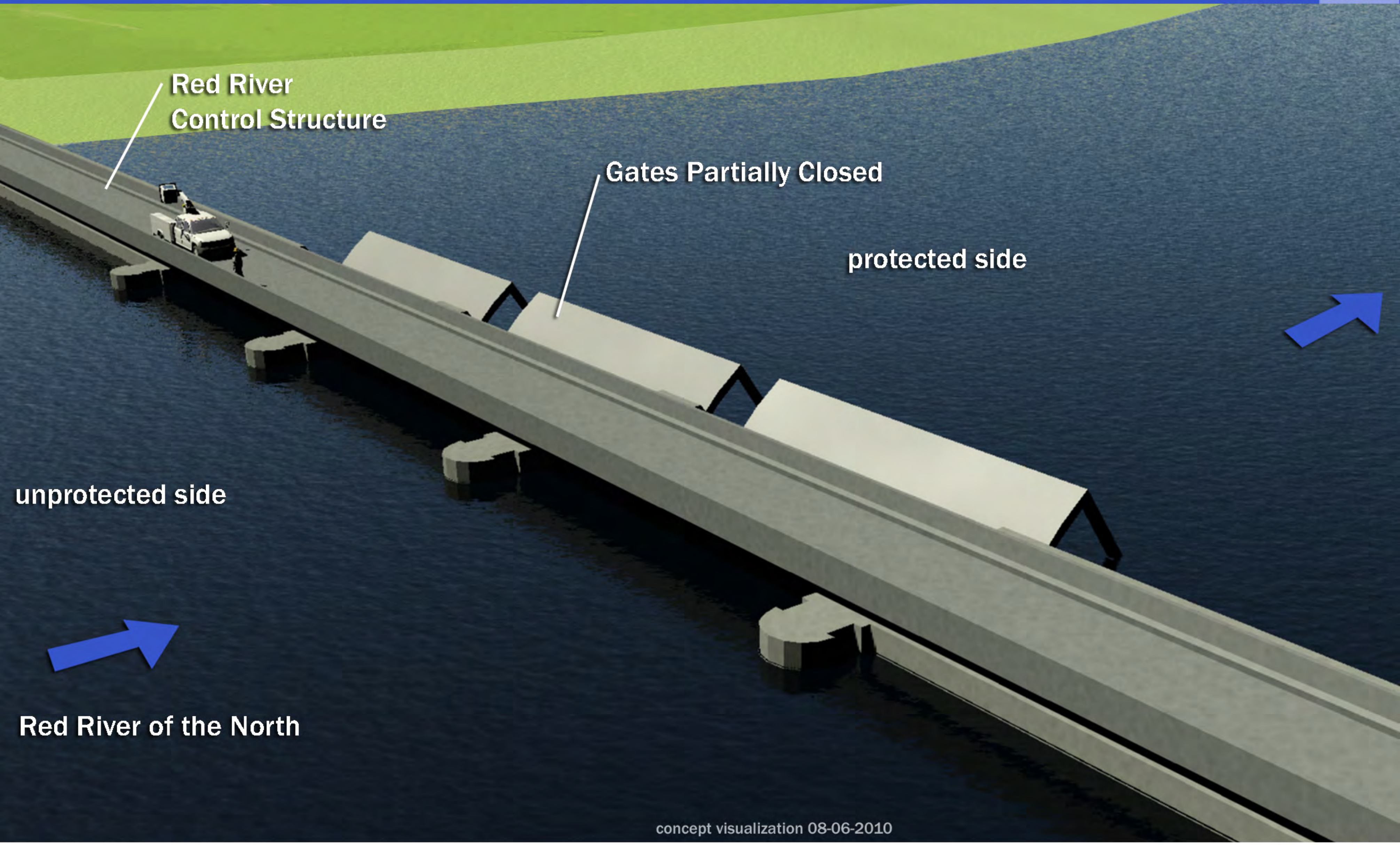
Red River Control Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2:
2-year flow

Figure F36

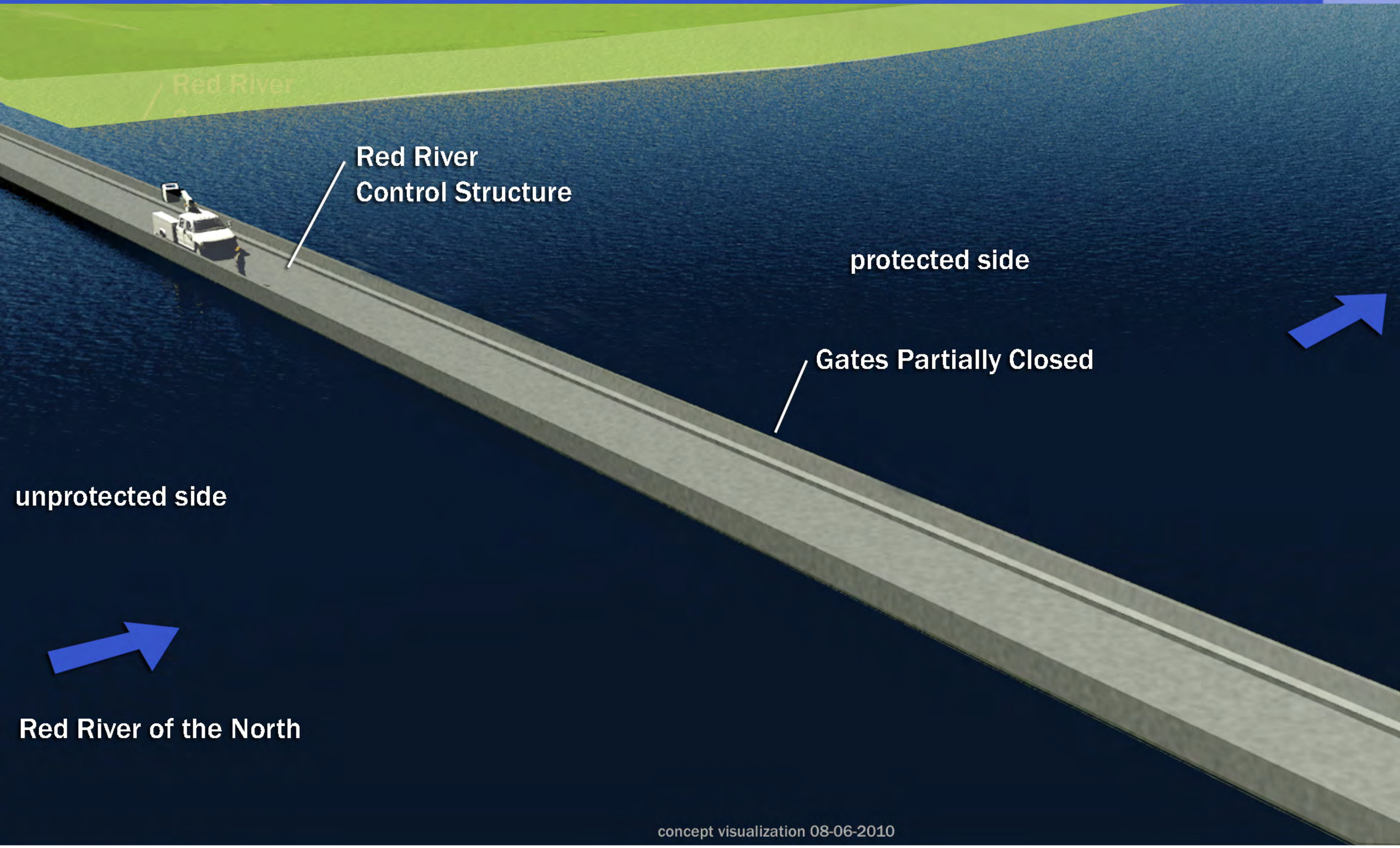
Red River Control Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3:
10-year flow

Figure F37

Red River Control Structure - Flow Scenario 4



LPP ND East 35K
Flow Scenario 4:
100-year flow

Figure F38

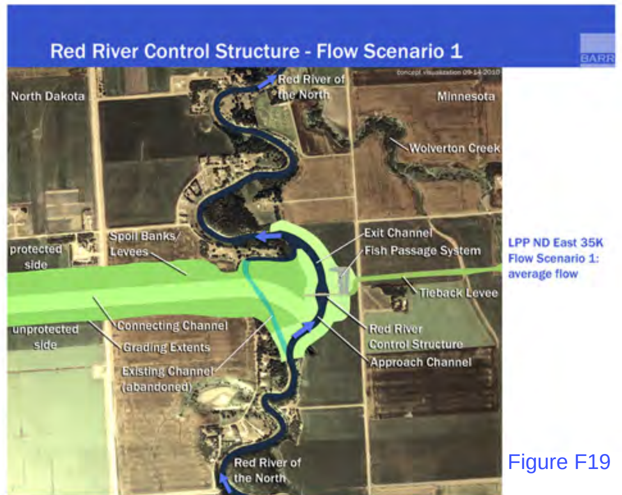


Figure F19

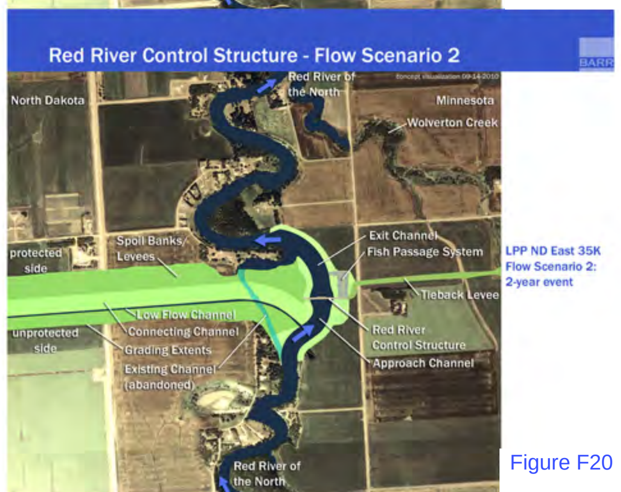


Figure F20

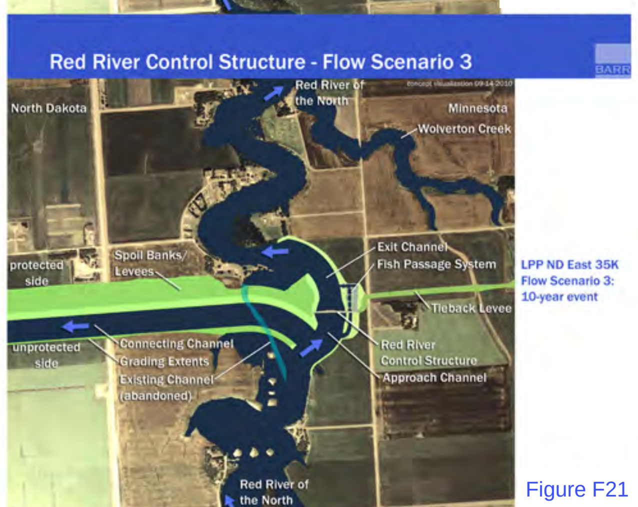


Figure F21

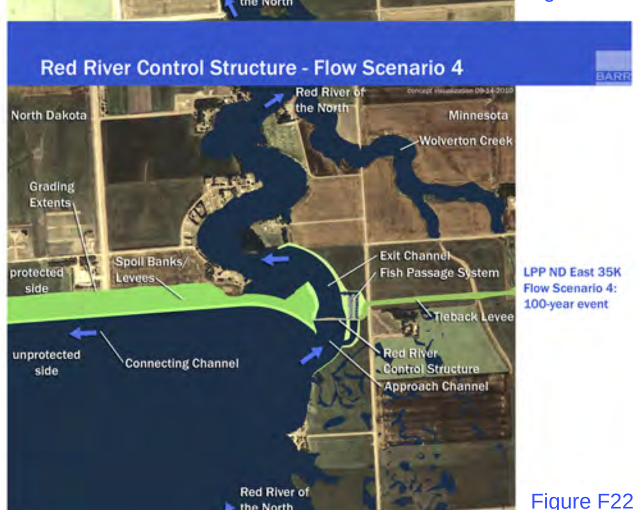
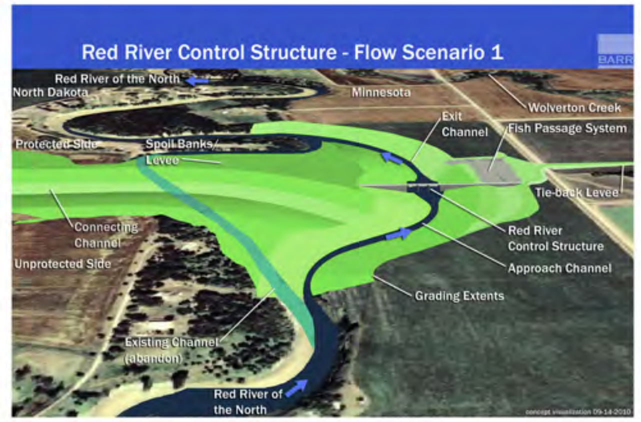
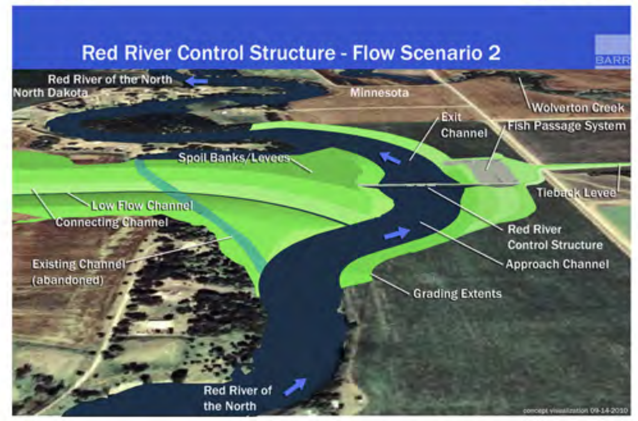


Figure F22



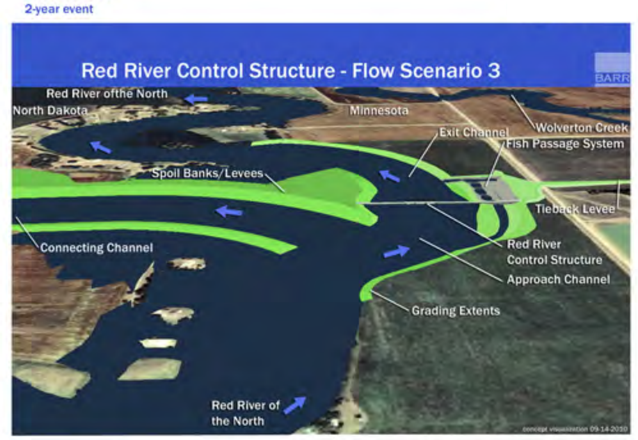
LPP ND East 35K
Flow Scenario 1:
average flow

Figure F23



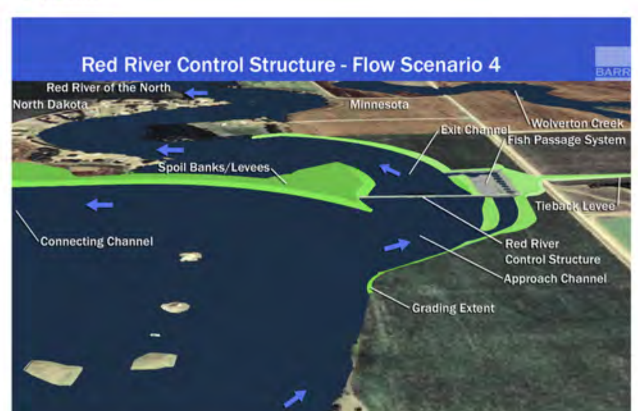
LPP ND East 35K
Flow Scenario 2:
2-year event

Figure F24



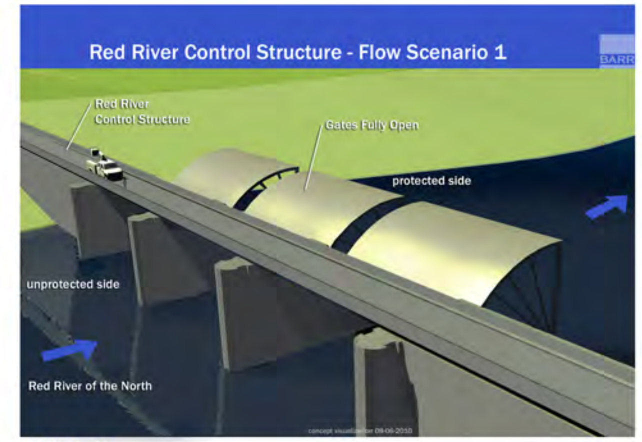
LPP ND East 35K
Flow Scenario 3:
10-year event

Figure F25



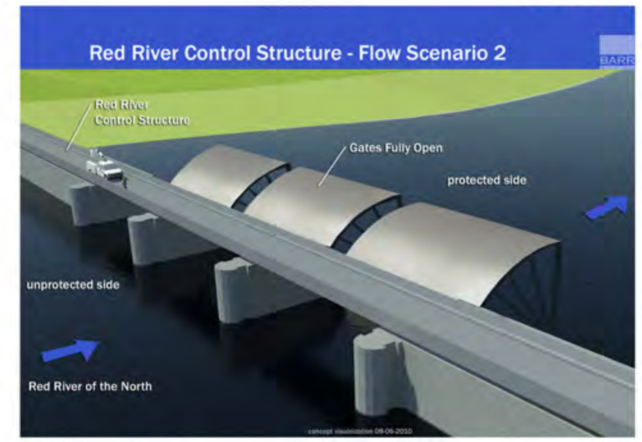
LPP ND East 35K
Flow Scenario 4:
100-year event

Figure F26



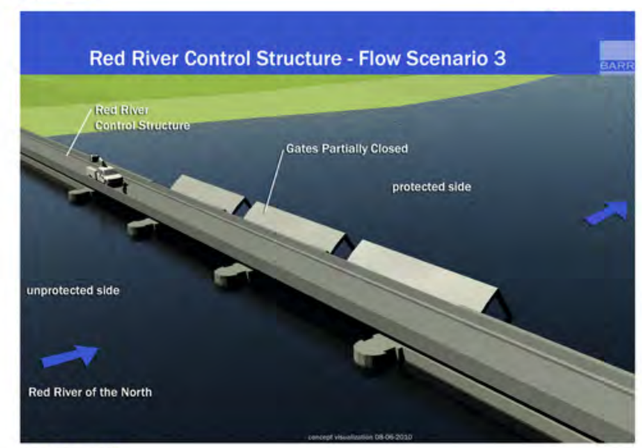
LPP ND East 35K
Flow Scenario 1:
average flow

Figure F35



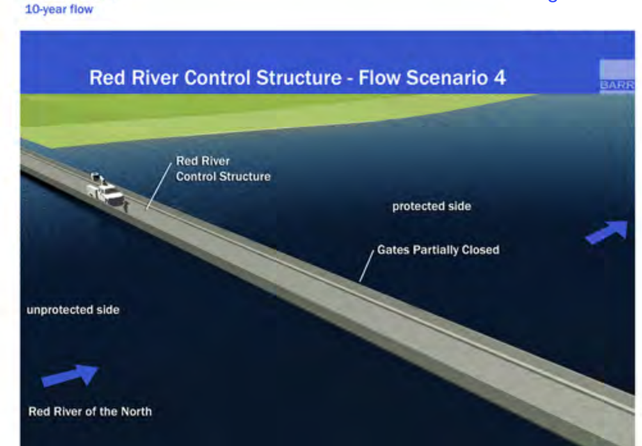
LPP ND East 35K
Flow Scenario 2:
2-year flow

Figure F36



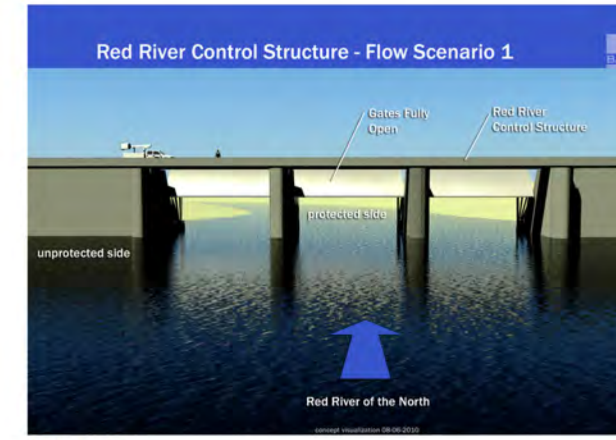
LPP ND East 35K
Flow Scenario 3:
10-year flow

Figure F37



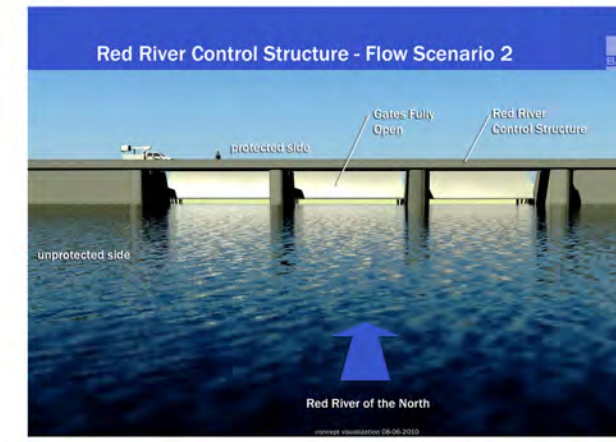
LPP ND East 35K
Flow Scenario 4:
100-year flow

Figure F38



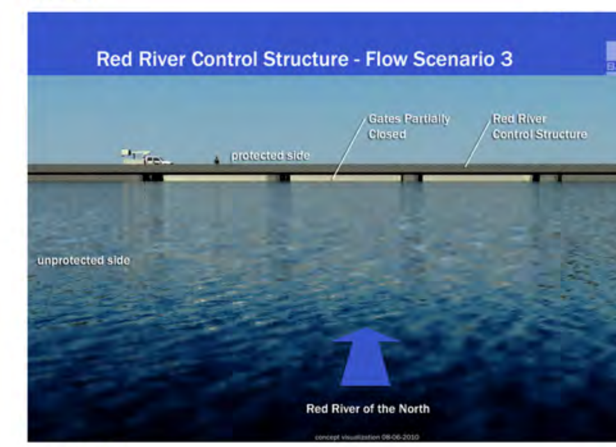
LPP ND East 35K
Flow Scenario 1:
average flow

Figure F32



LPP ND East 35K
Flow Scenario 2
2-year flow

Figure F33

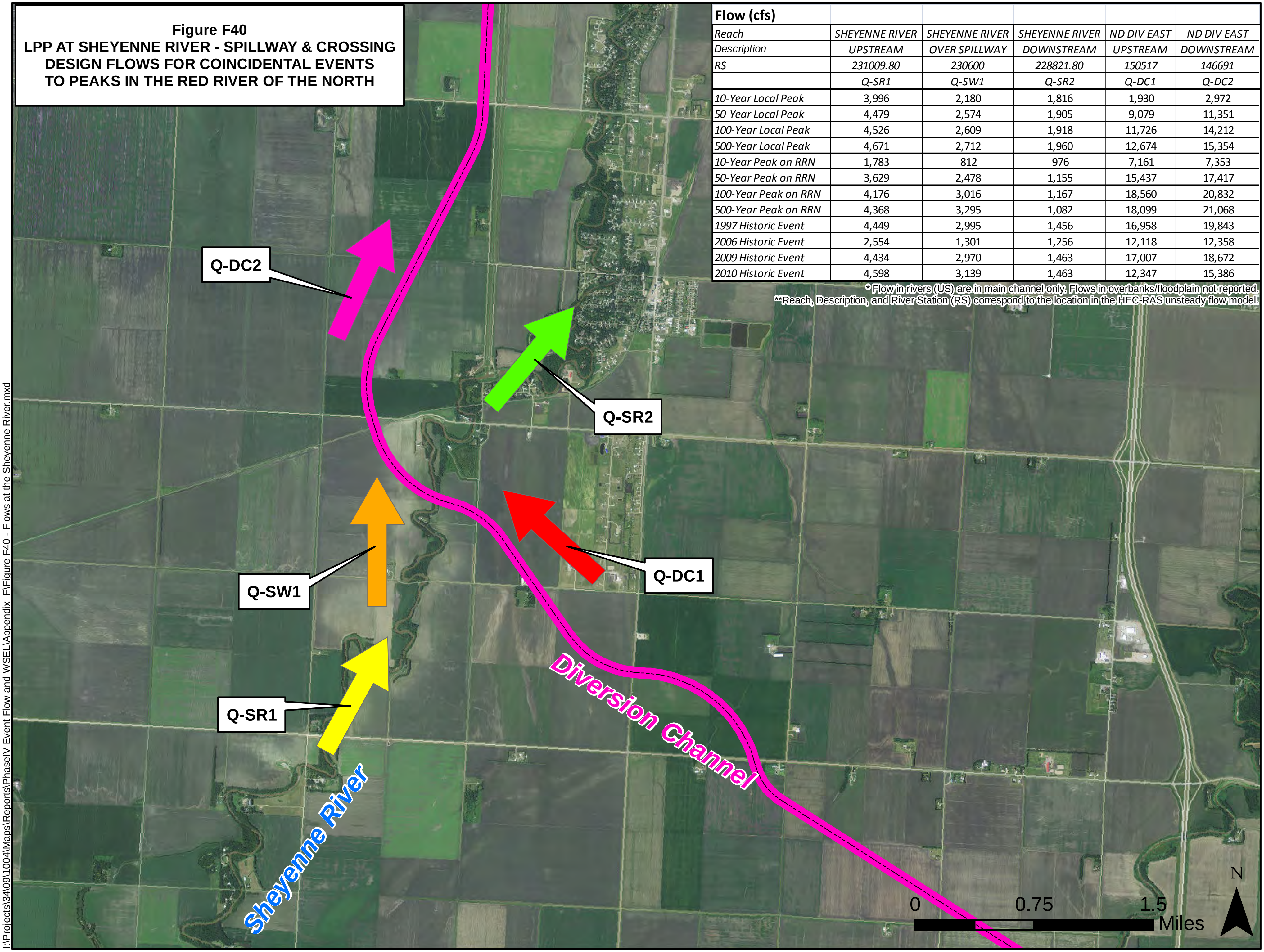


LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F34

Figure F39
MOSAIC OF RENDERINGS OF
RED RIVER LPP CONTROL STRUCTURE

Figure F40
LPP AT SHEYENNE RIVER - SPILLWAY & CROSSING
DESIGN FLOWS FOR COINCIDENTAL EVENTS
TO PEAKS IN THE RED RIVER OF THE NORTH



Flow (cfs)					
Reach	*SHEYENNE RIVER*	*SHEYENNE RIVER*	*SHEYENNE RIVER*	*ND DIV EAST*	*ND DIV EAST*
Description	*UPSTREAM*	*OVER SPILLWAY*	*DOWNSTREAM*	*UPSTREAM*	*DOWNSTREAM*
RS	*231009.80*	*230600*	*228821.80*	*150517*	*146691*
	Q-SR1	*Q-SW1*	*Q-SR2*	*Q-DC1*	*Q-DC2*
10-Year Local Peak	3,996	2,180	1,816	1,930	2,972
50-Year Local Peak	4,479	2,574	1,905	9,079	11,351
100-Year Local Peak	4,526	2,609	1,918	11,726	14,212
500-Year Local Peak	4,671	2,712	1,960	12,674	15,354
10-Year Peak on RRN	1,783	812	976	7,161	7,353
50-Year Peak on RRN	3,629	2,478	1,155	15,437	17,417
100-Year Peak on RRN	4,176	3,016	1,167	18,560	20,832
500-Year Peak on RRN	4,368	3,295	1,082	18,099	21,068
1997 Historic Event	4,449	2,995	1,456	16,958	19,843
2006 Historic Event	2,554	1,301	1,256	12,118	12,358
2009 Historic Event	4,434	2,970	1,463	17,007	18,672
2010 Historic Event	4,598	3,139	1,463	12,347	15,386

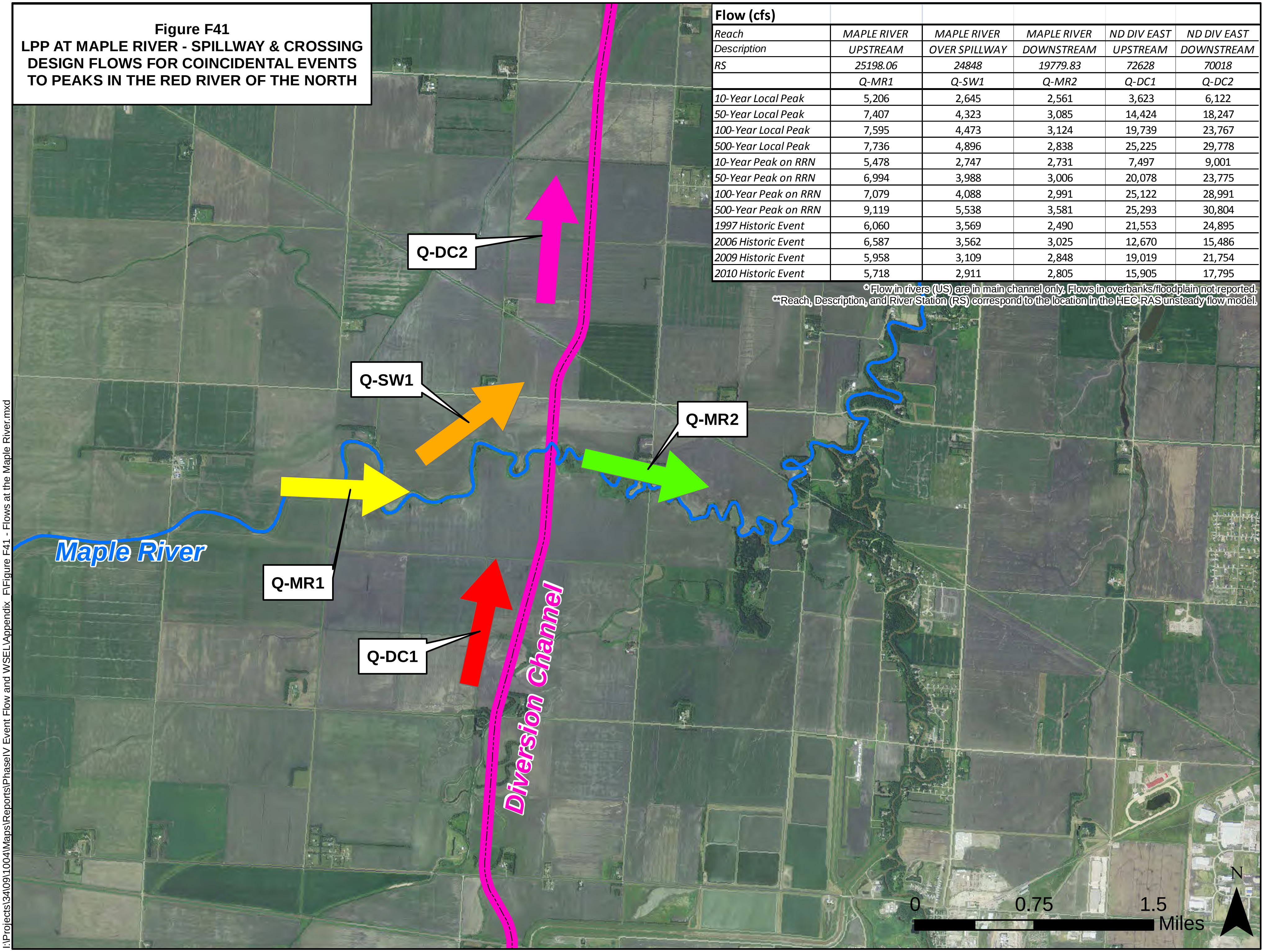
* Flow in rivers (US) are in main channel only. Flows in overbanks/floodplain not reported.
**Reach, Description, and River Station (RS) correspond to the location in the HEC-RAS unsteady flow model.

I:\Projects\3409\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F40 - Flows at the Sheyenne River.mxd

Figure F41
LPP AT MAPLE RIVER - SPILLWAY & CROSSING DESIGN FLOWS FOR COINCIDENTAL EVENTS TO PEAKS IN THE RED RIVER OF THE NORTH

Flow (cfs)					
Reach	*MAPLE RIVER*	*MAPLE RIVER*	*MAPLE RIVER*	*ND DIV EAST*	*ND DIV EAST*
Description	*UPSTREAM*	*OVER SPILLWAY*	*DOWNSTREAM*	*UPSTREAM*	*DOWNSTREAM*
RS	*25198.06*	*24848*	*19779.83*	*72628*	*70018*
	Q-MR1	*Q-SW1*	*Q-MR2*	*Q-DC1*	*Q-DC2*
10-Year Local Peak	5,206	2,645	2,561	3,623	6,122
50-Year Local Peak	7,407	4,323	3,085	14,424	18,247
100-Year Local Peak	7,595	4,473	3,124	19,739	23,767
500-Year Local Peak	7,736	4,896	2,838	25,225	29,778
10-Year Peak on RRN	5,478	2,747	2,731	7,497	9,001
50-Year Peak on RRN	6,994	3,988	3,006	20,078	23,775
100-Year Peak on RRN	7,079	4,088	2,991	25,122	28,991
500-Year Peak on RRN	9,119	5,538	3,581	25,293	30,804
1997 Historic Event	6,060	3,569	2,490	21,553	24,895
2006 Historic Event	6,587	3,562	3,025	12,670	15,486
2009 Historic Event	5,958	3,109	2,848	19,019	21,754
2010 Historic Event	5,718	2,911	2,805	15,905	17,795

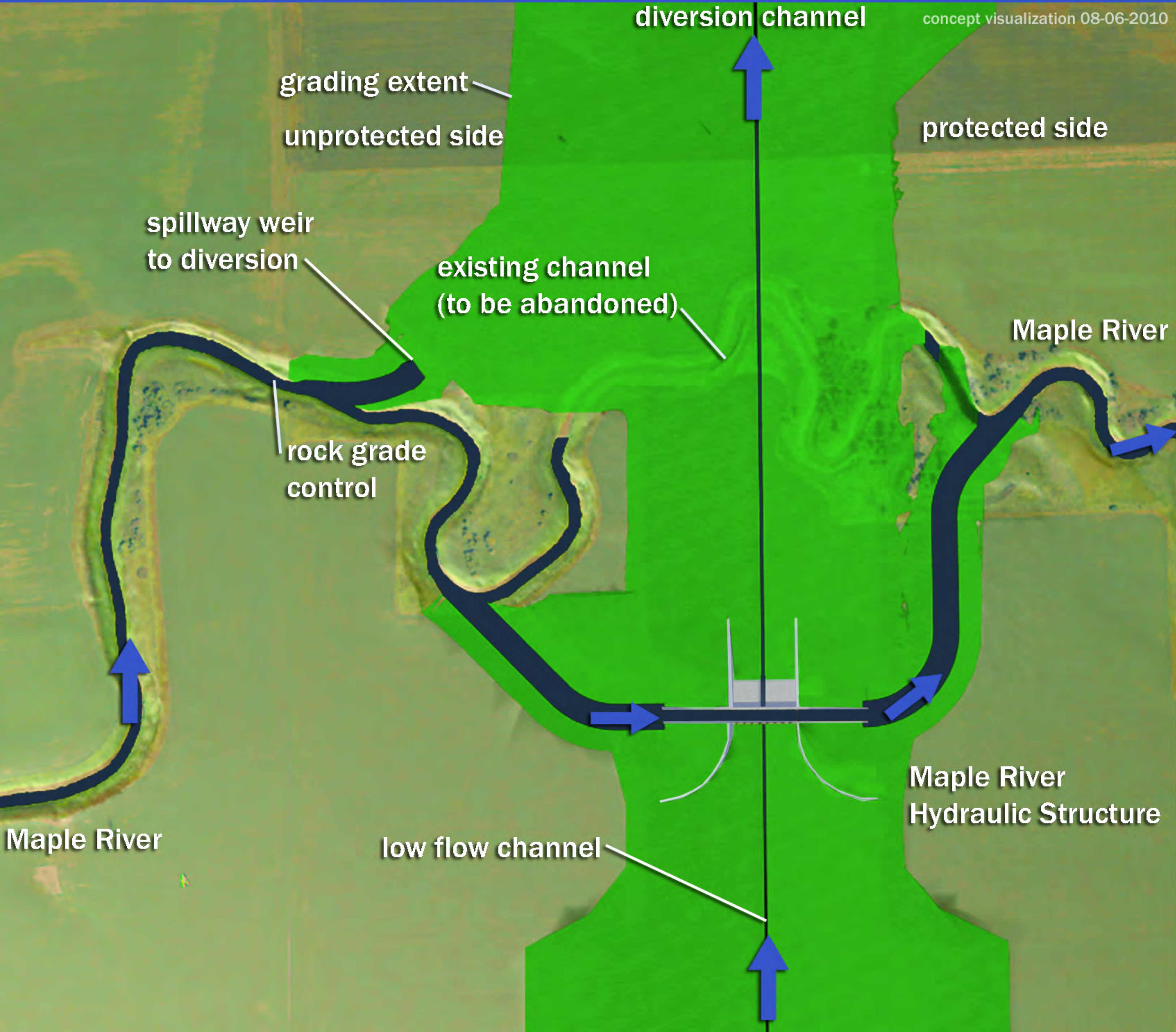
*Flow in rivers (US) are in main channel only. Flows in overbanks/floodplain not reported.
**Reach, Description, and River Station (RS) correspond to the location in the HEC-RAS unsteady flow model.



I:\Projects\3409\1004\Maps\Reports\PhaseIV Event Flow and WSEL\Appendix_F\Figure F41 - Flows at the Maple River.mxd

Maple River Hydraulic Structure - Flow Scenario 1

BARR

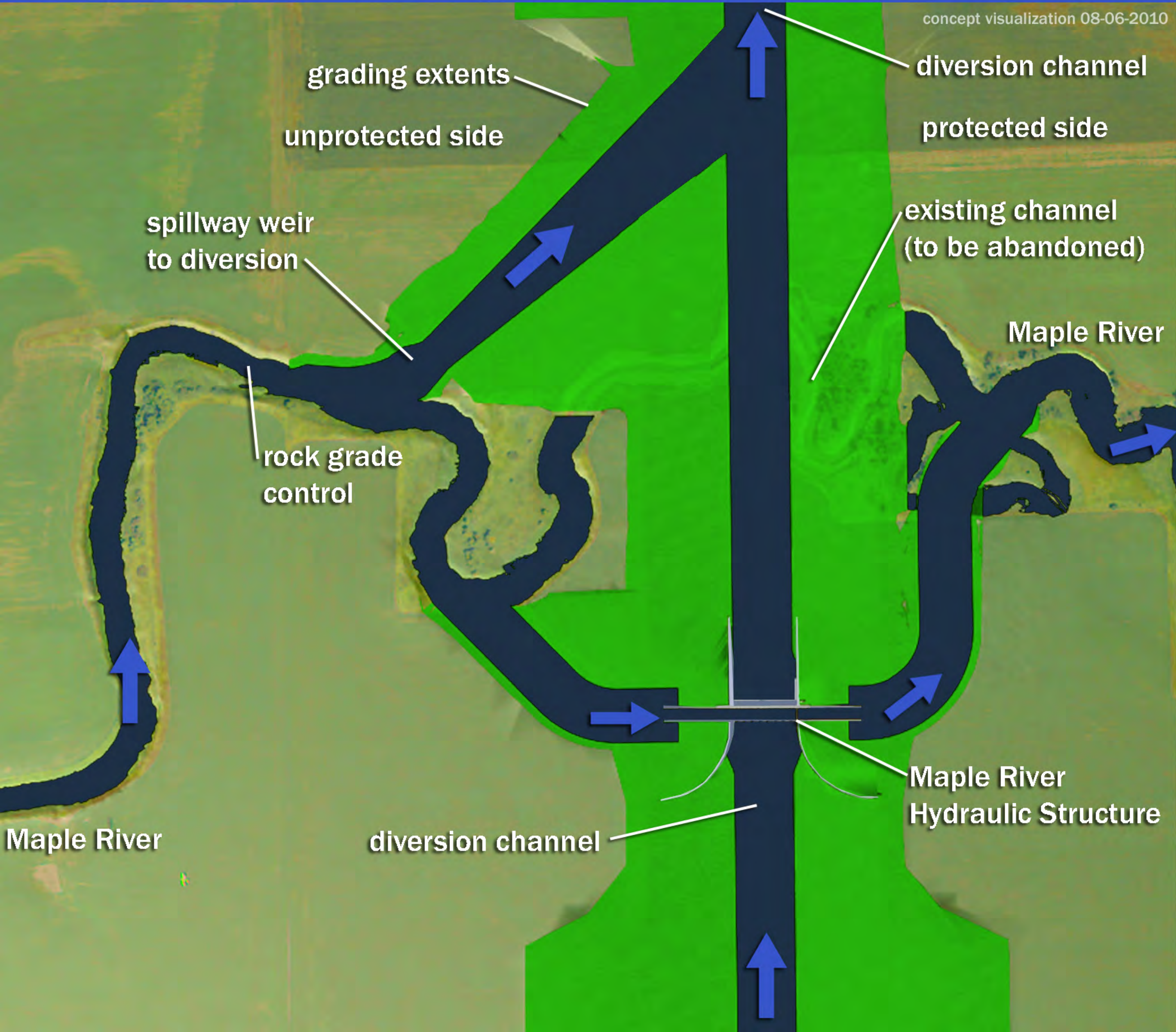


LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted
over spillway

Figure F42

Maple River Hydraulic Structure - Flow Scenario 2

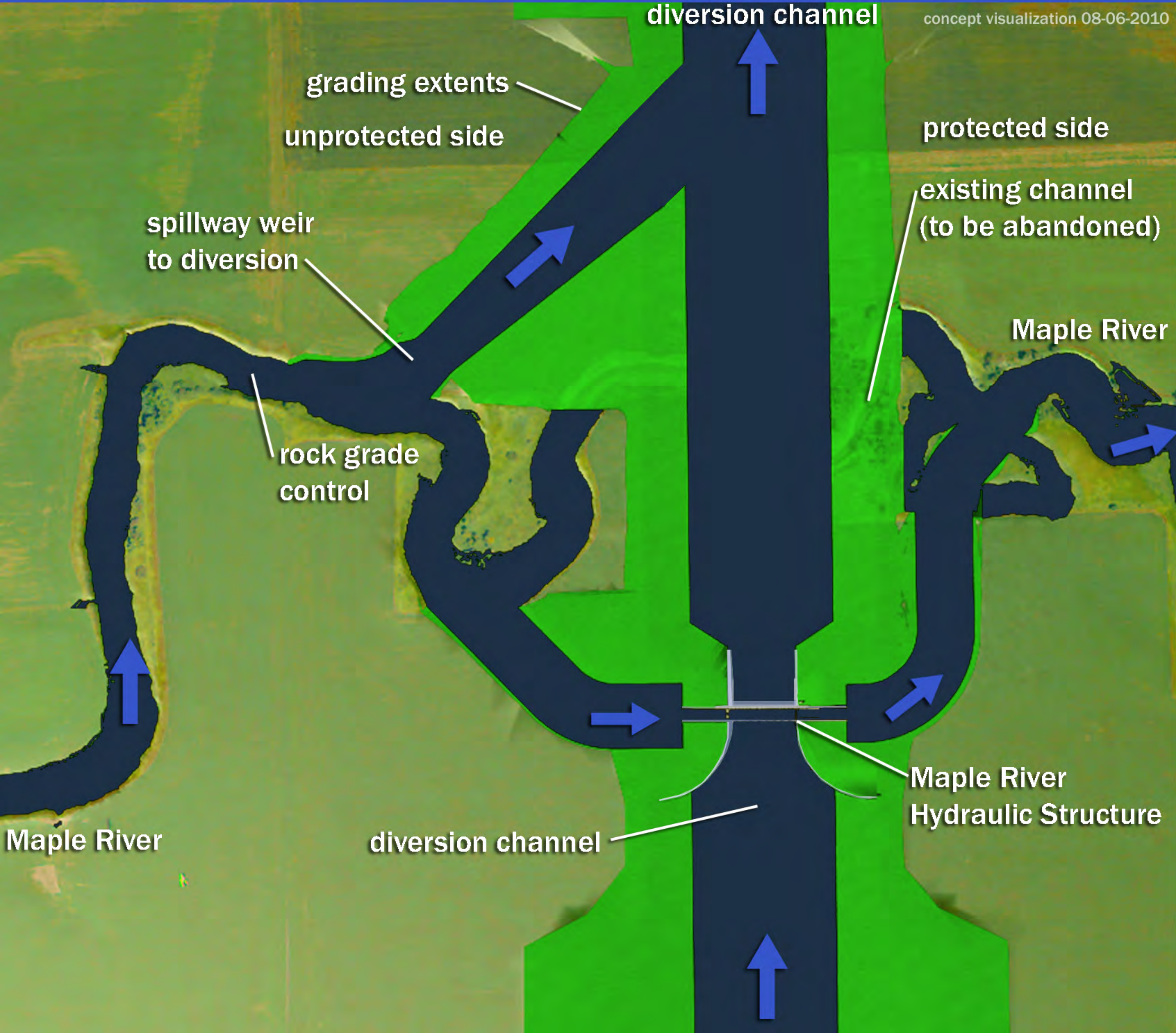
BARR



LPP ND East 35K
Flow Scenario 2
unpressurized
diversion flow,
flow over spillway

Figure F43

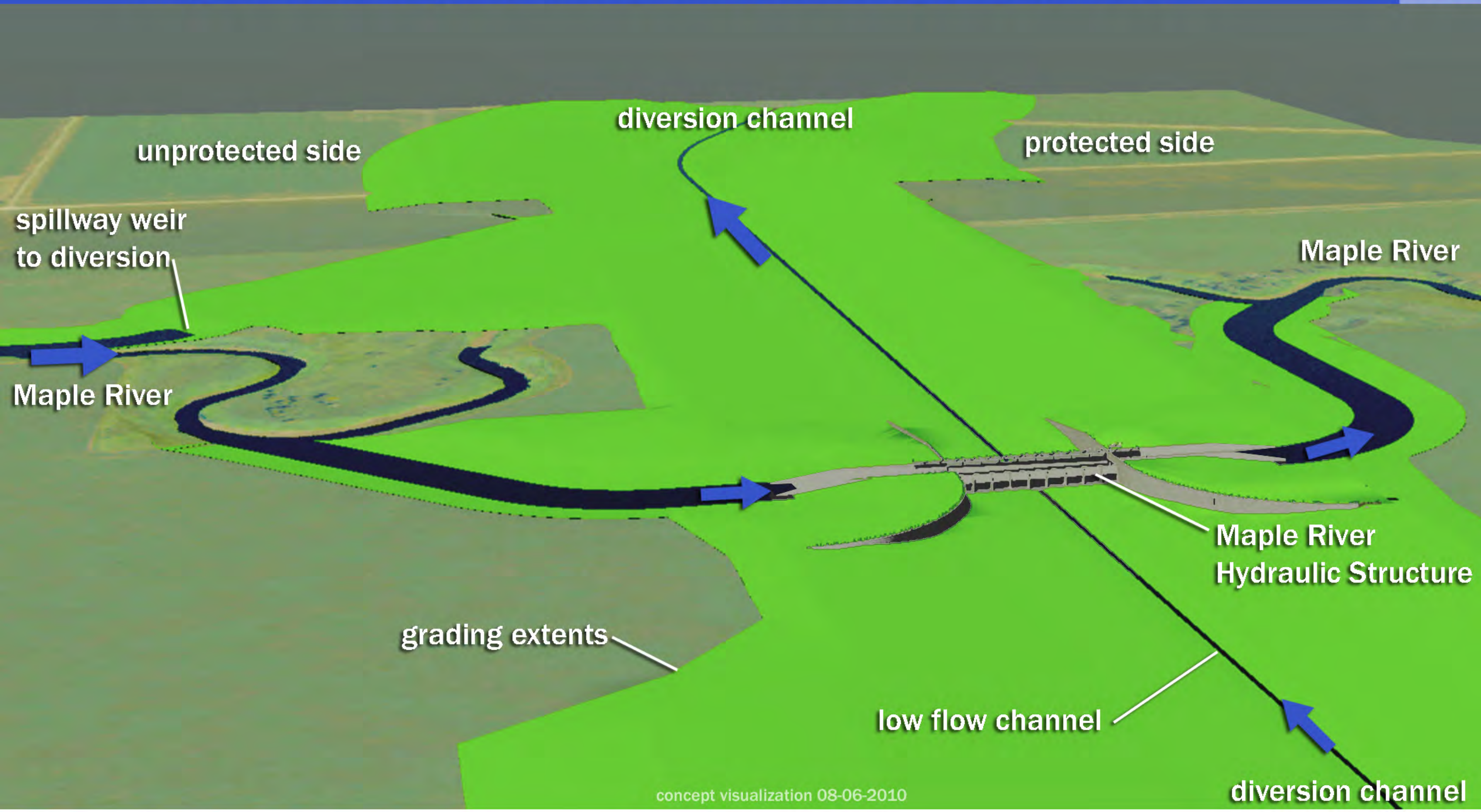
Maple River Hydraulic Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3
pressurized
diversion flow,
flow over spillway

Figure F44

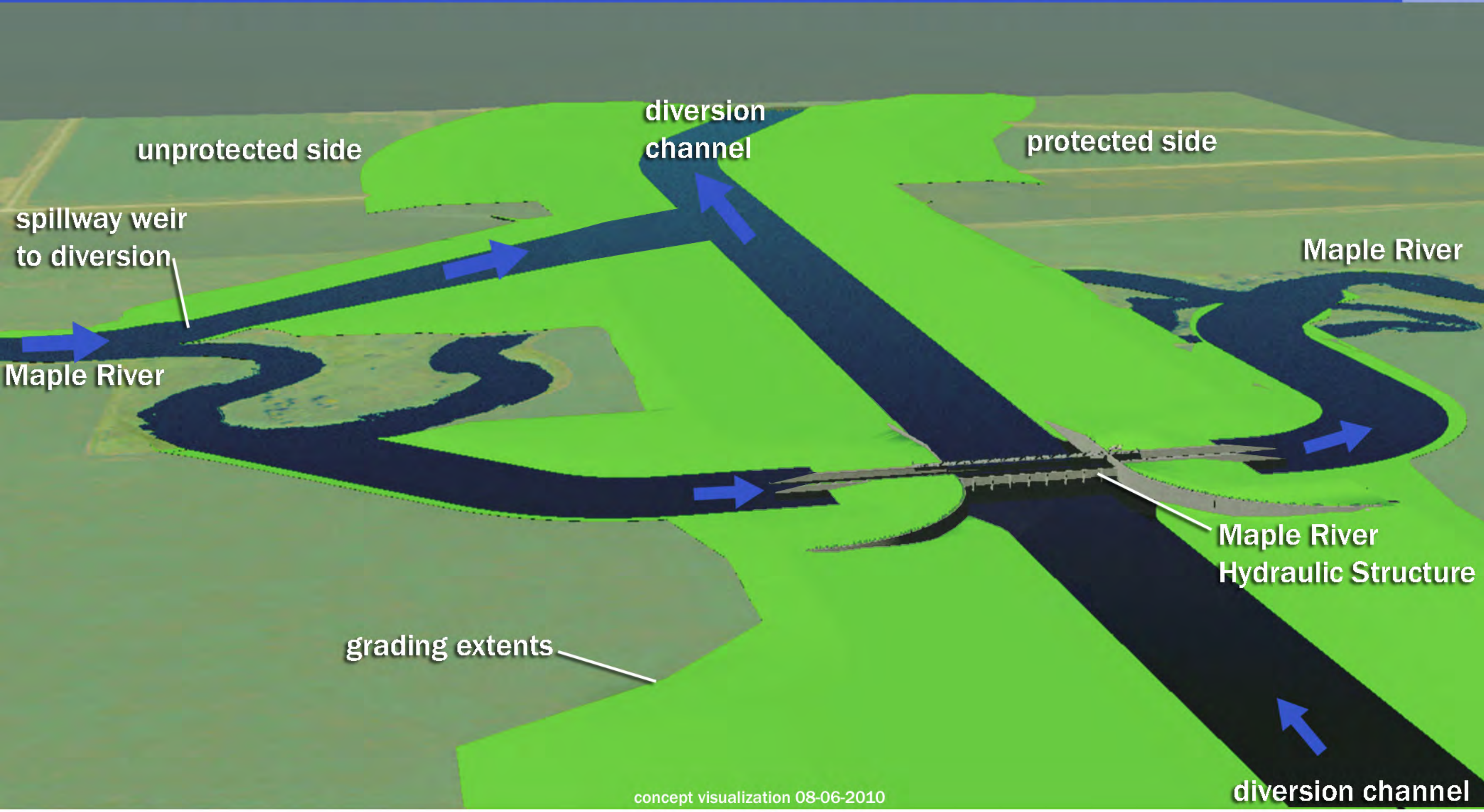
Maple River Hydraulic Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F45

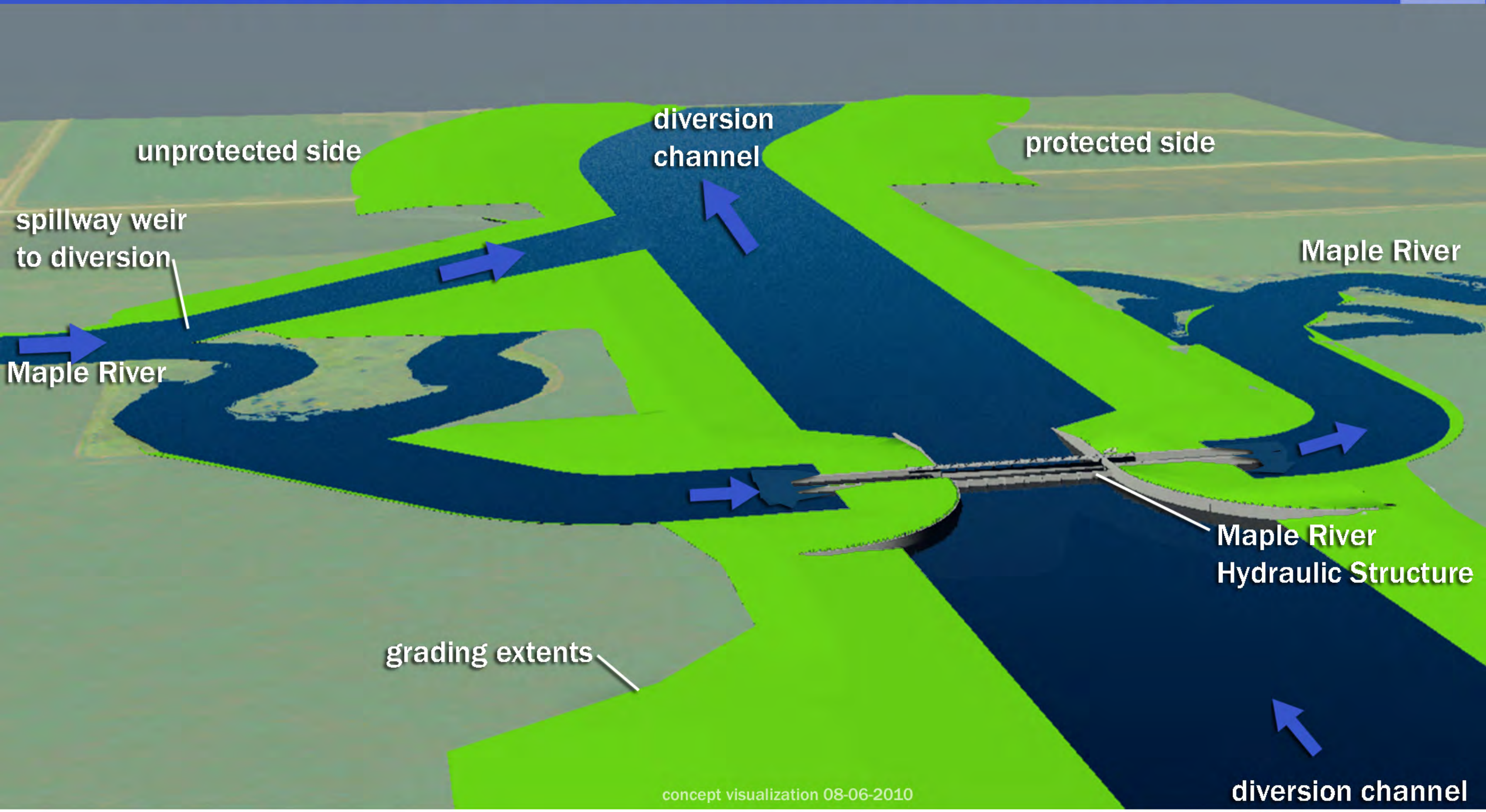
Maple River Hydraulic Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F46

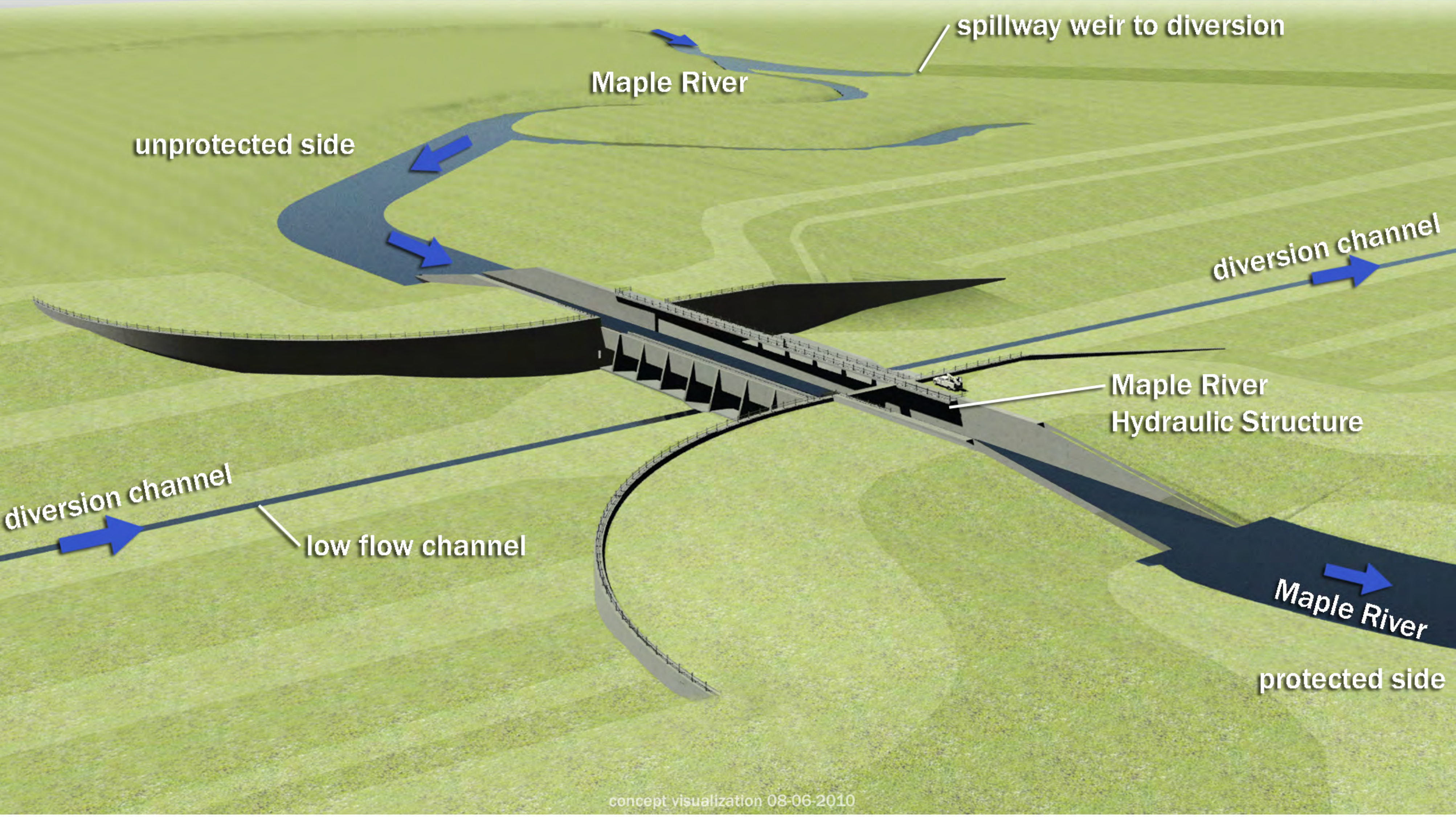
Maple River Hydraulic Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F47

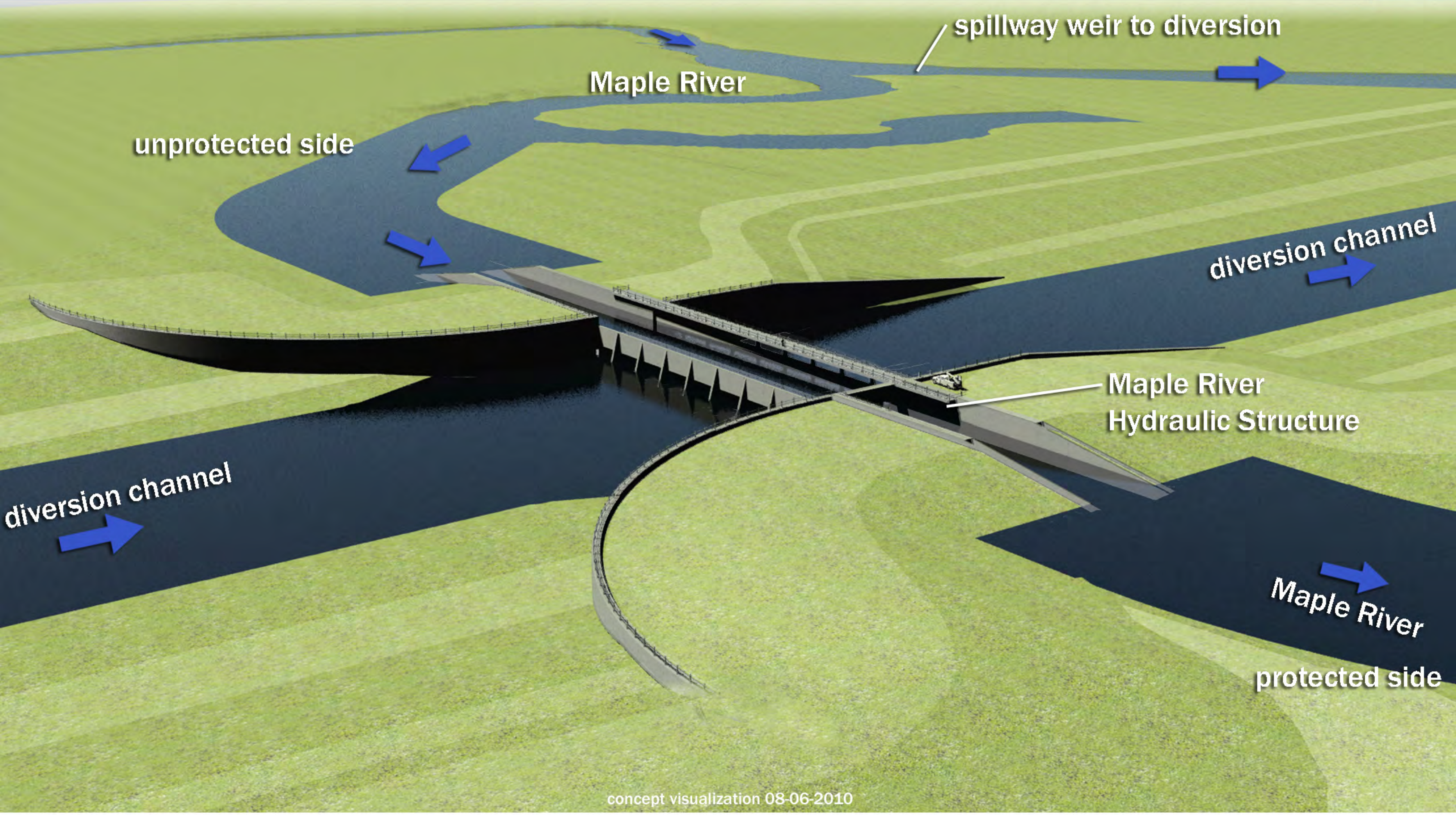
Maple River Hydraulic Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F48

Maple River Hydraulic Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

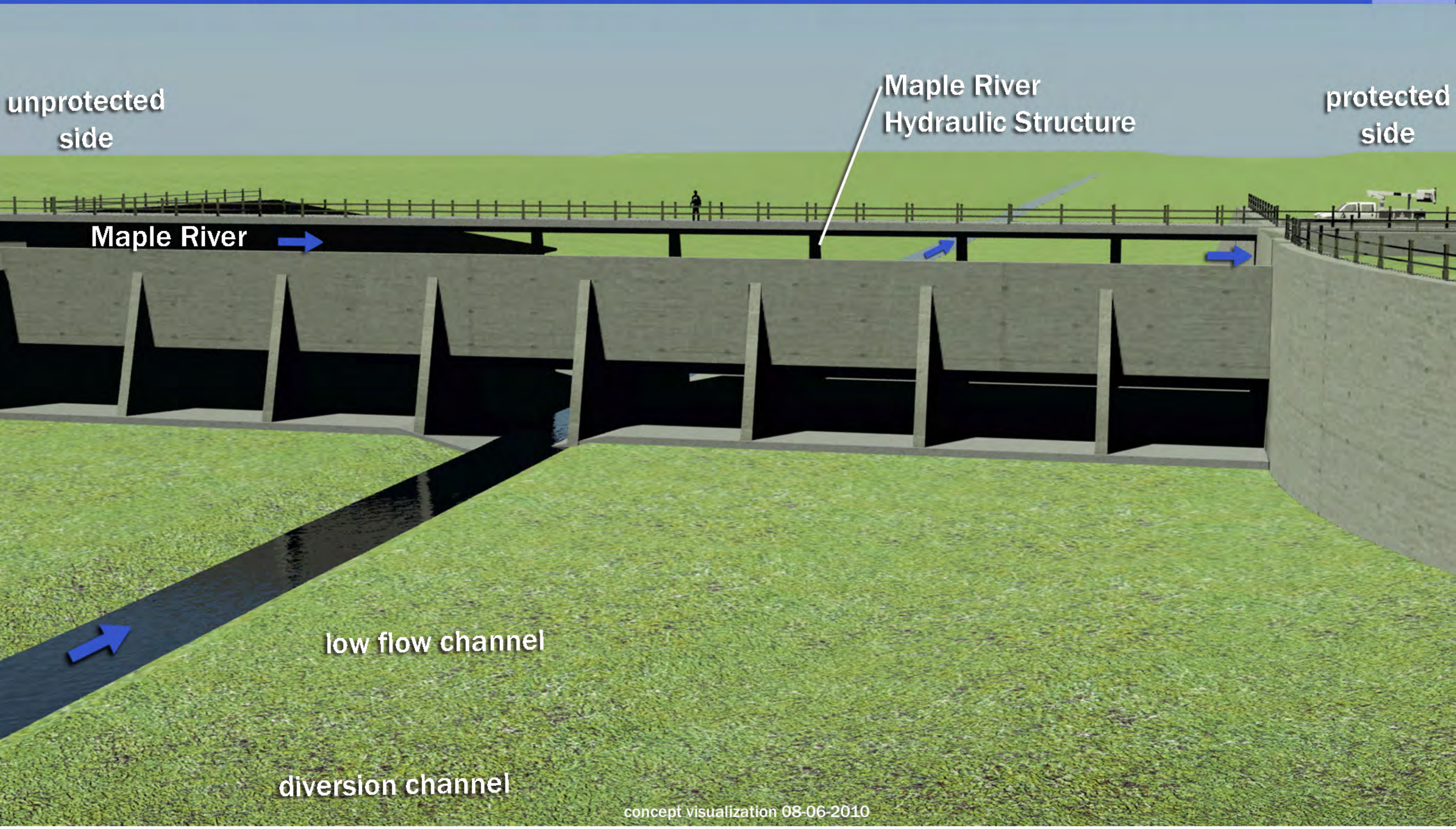
Figure F49

Maple River Hydraulic Structure - Flow Scenario 3

LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F50

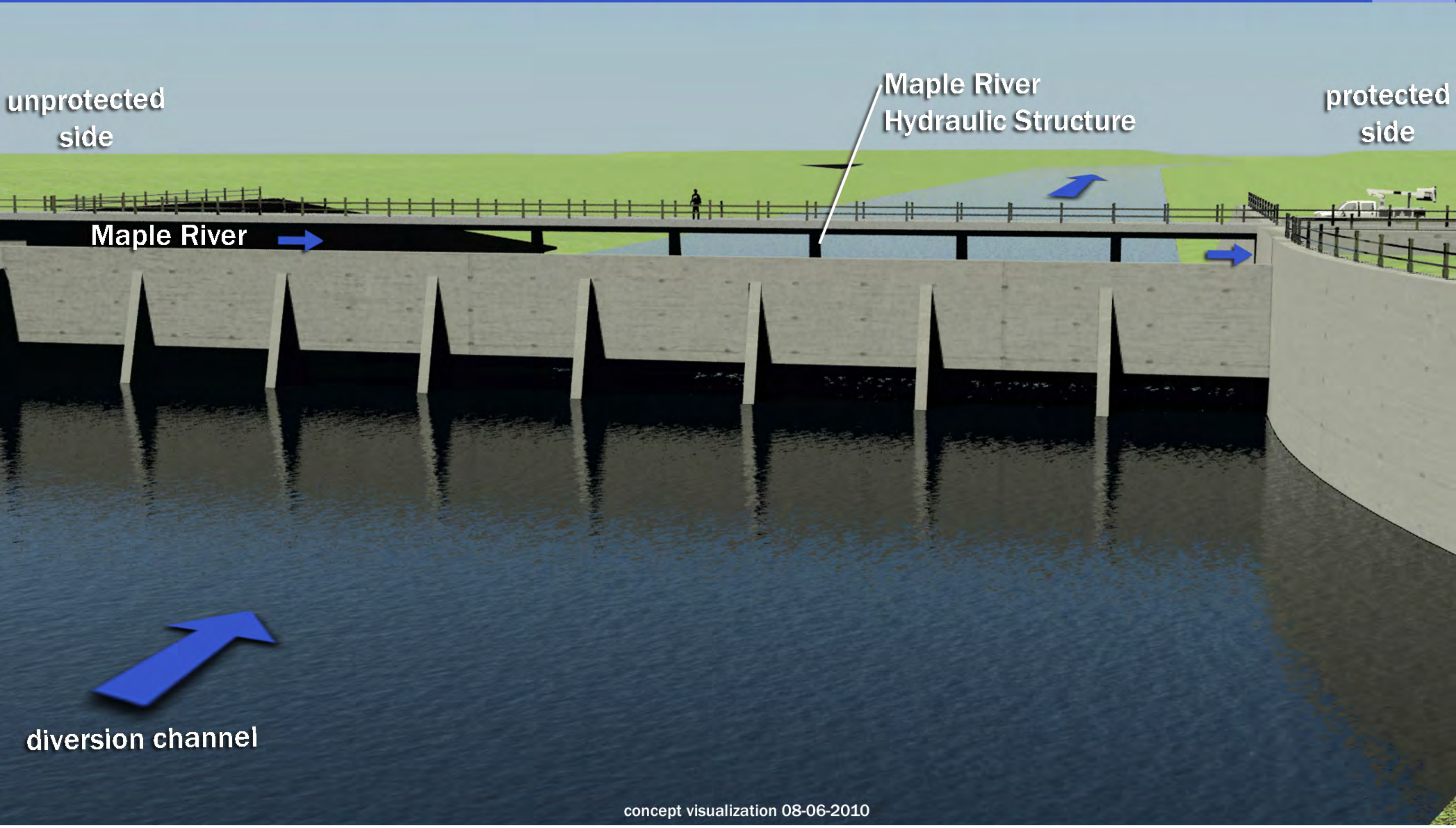
Maple River Hydraulic Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F51

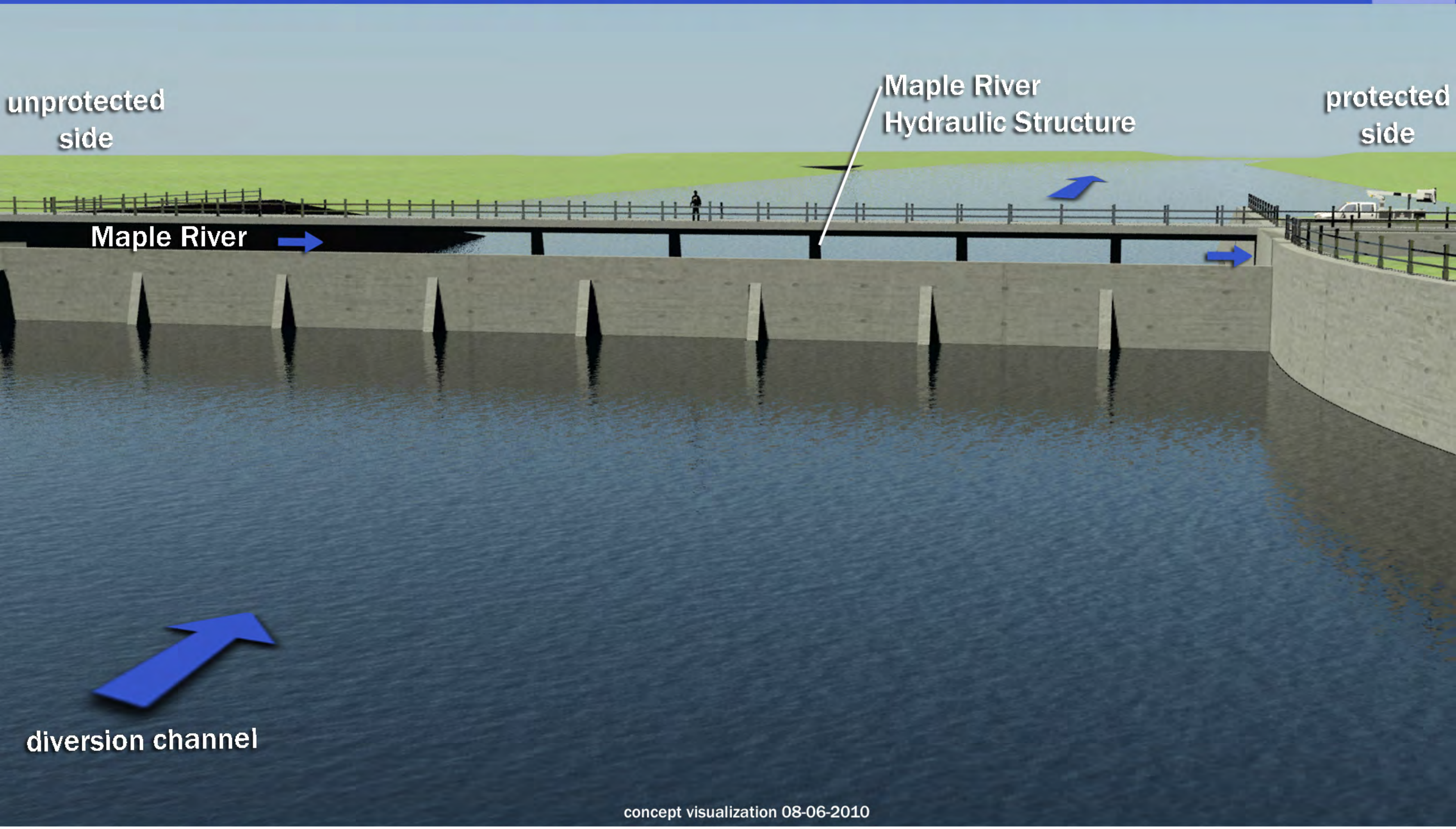
Maple River Hydraulic Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F52

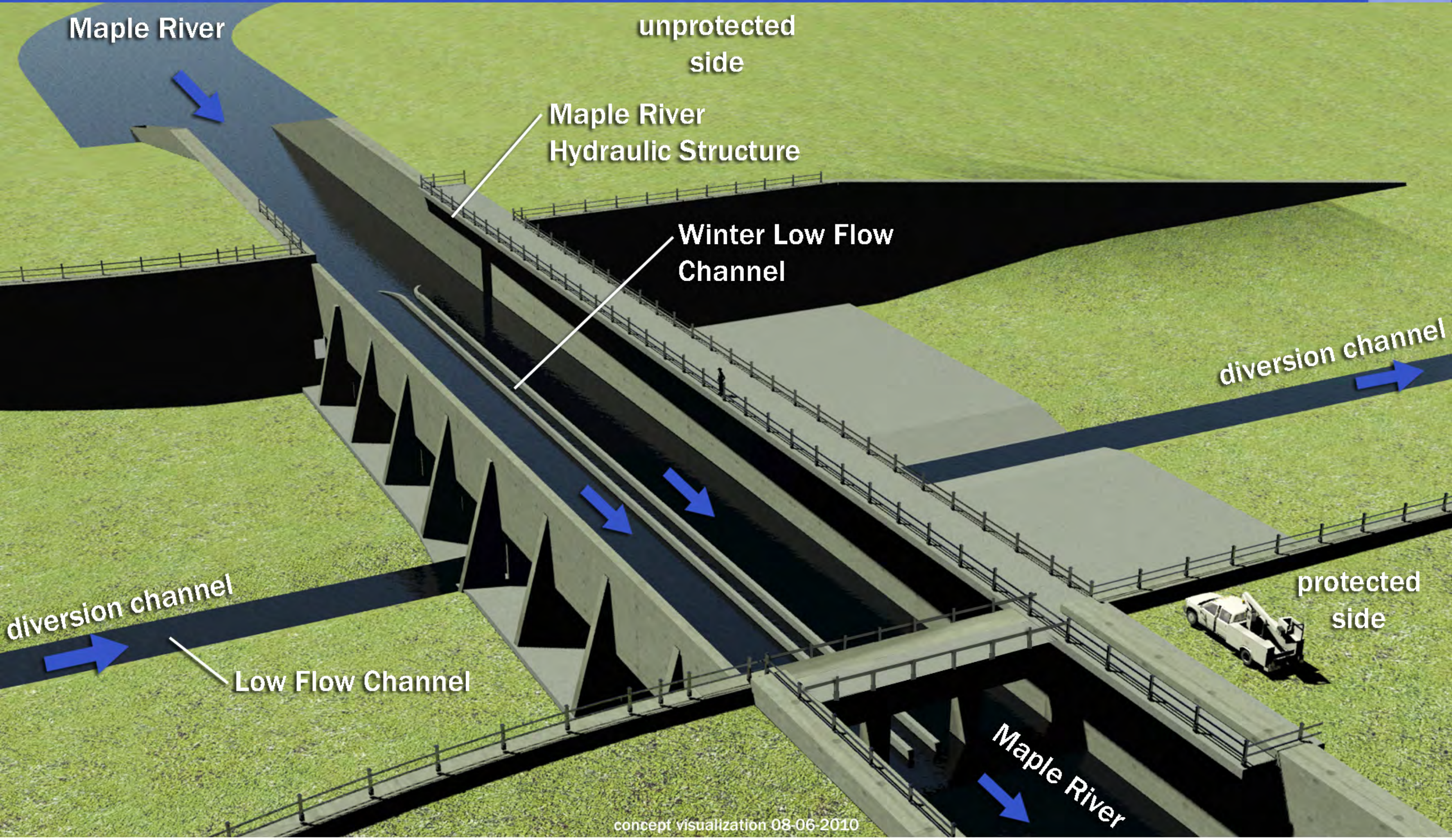
Maple River Hydraulic Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F53

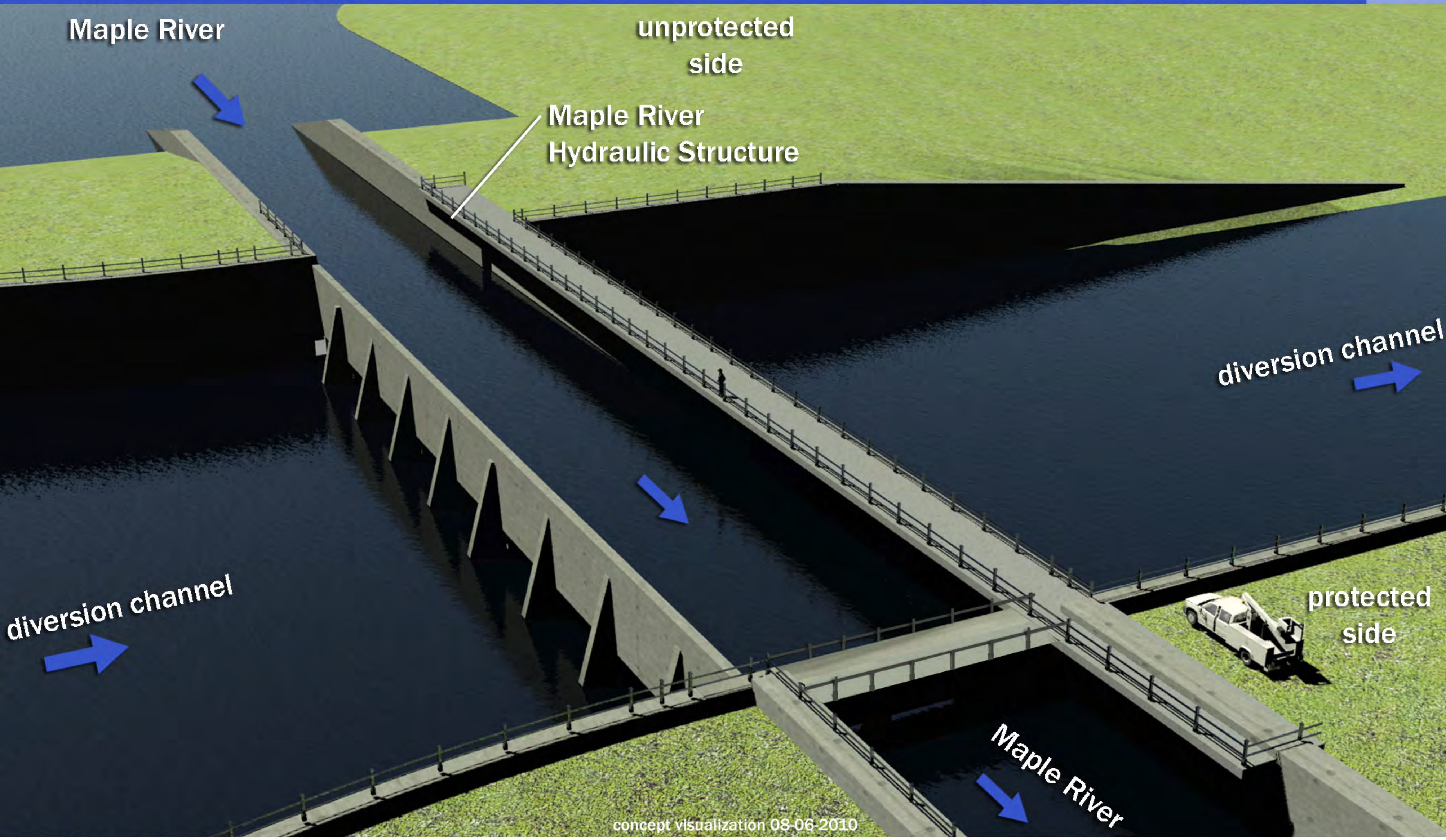
Maple River Hydraulic Structure - Flow Scenario 1



LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F54

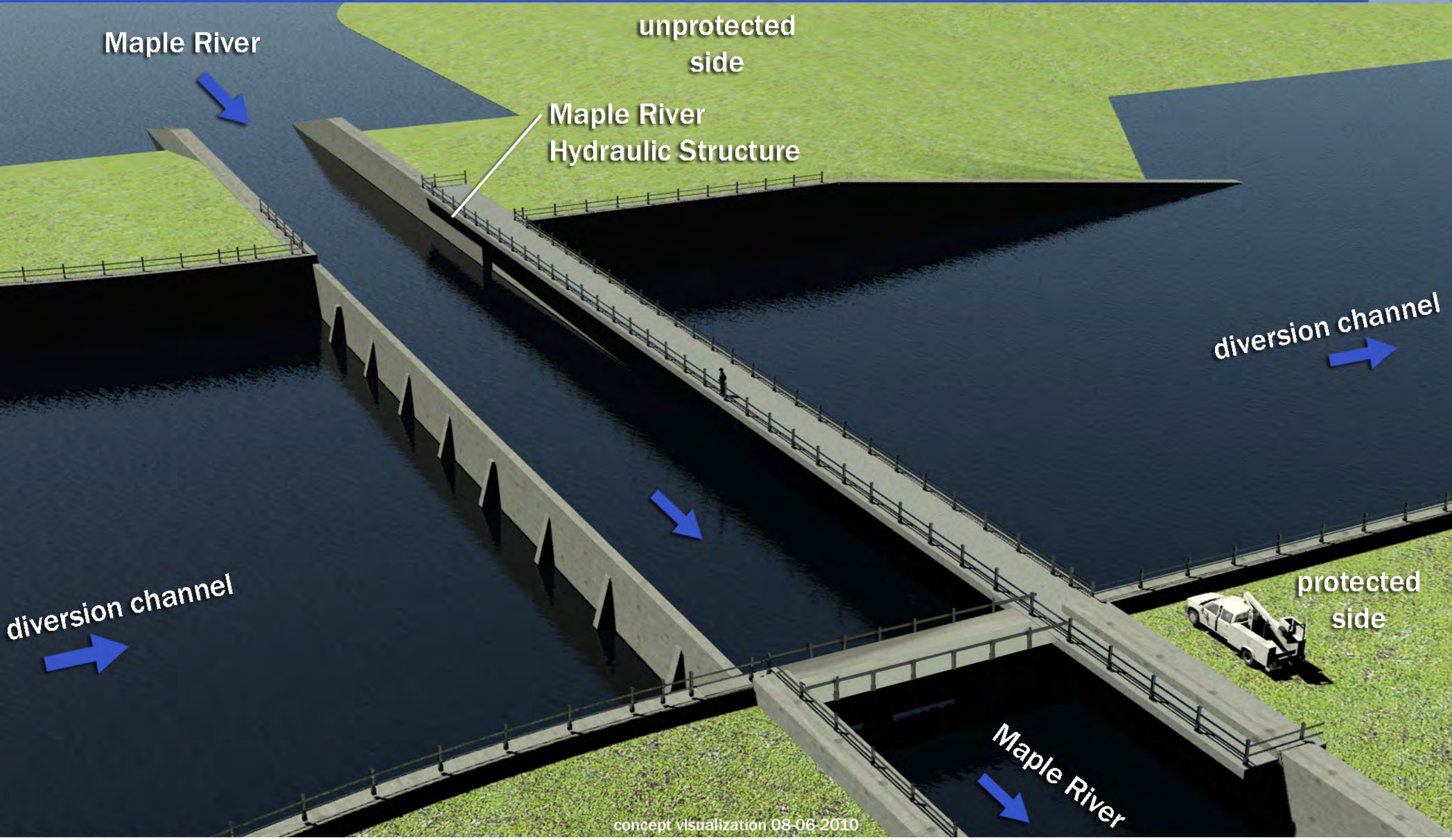
Maple River Hydraulic Structure - Flow Scenario 2



LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F55

Maple River Hydraulic Structure - Flow Scenario 3



LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F56

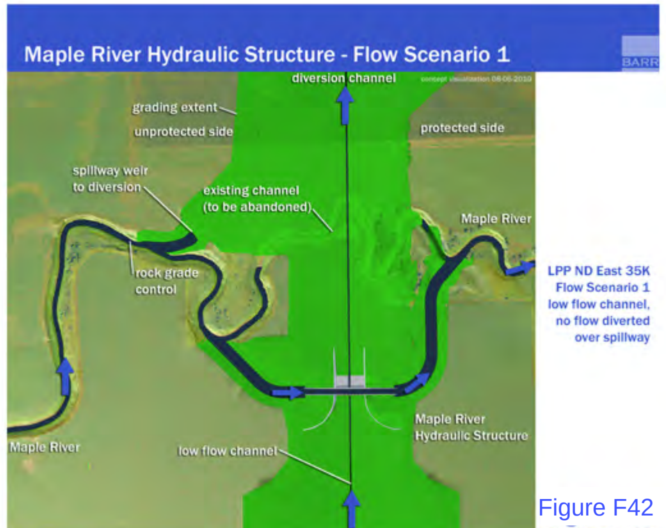


Figure F42

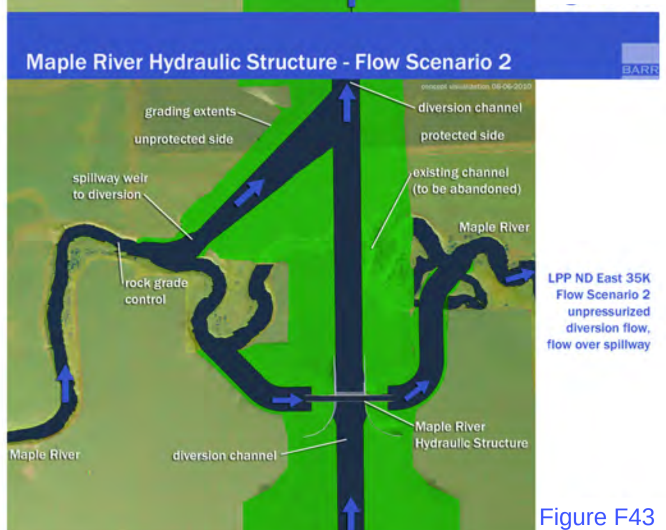


Figure F43

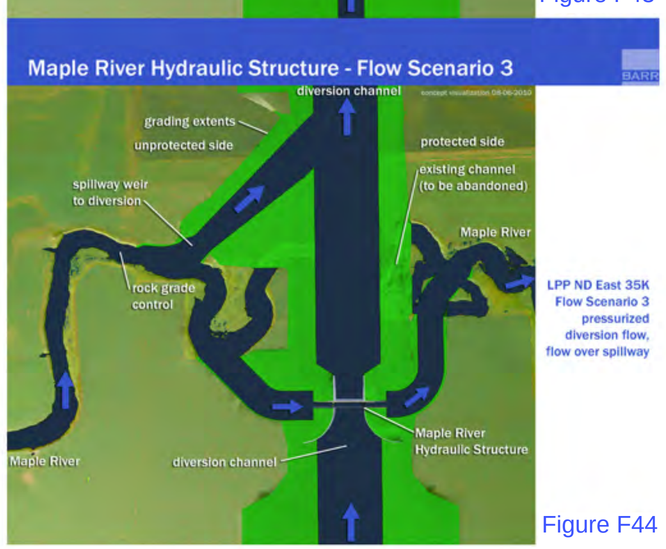
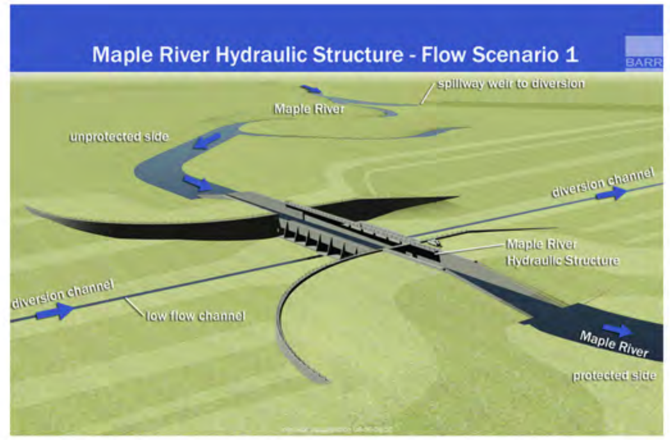
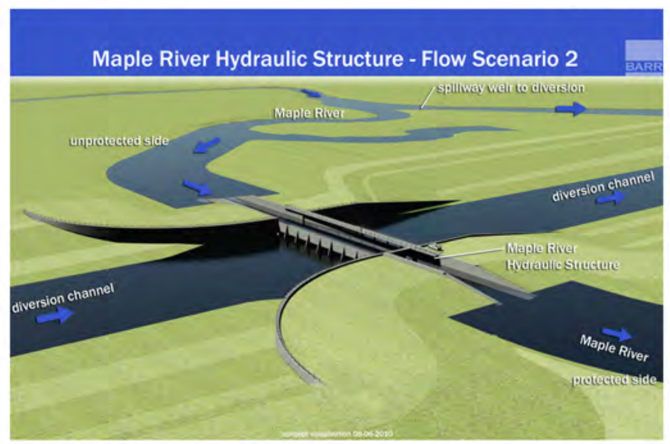


Figure F44



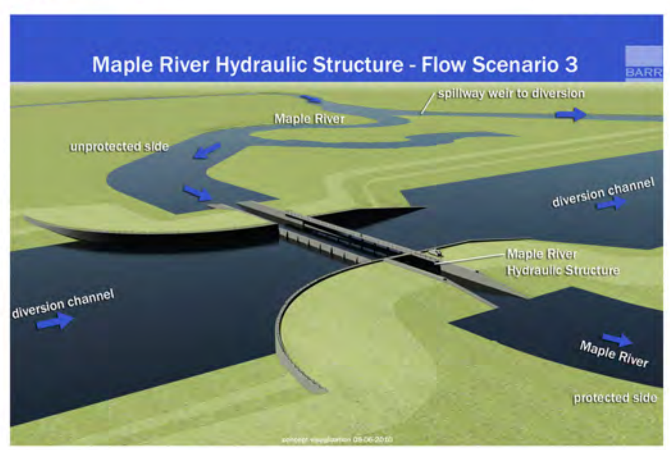
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F48



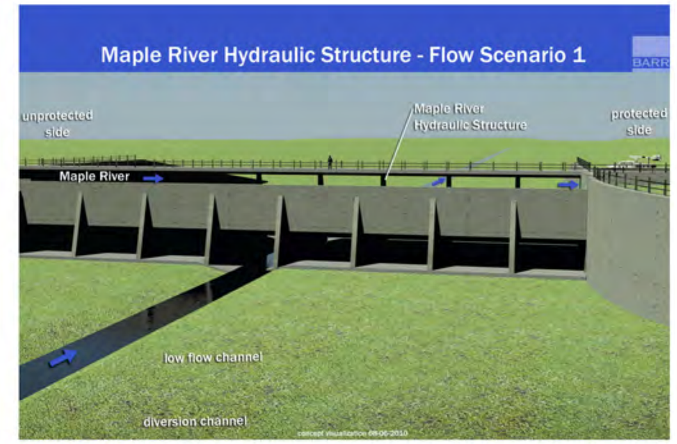
LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F49



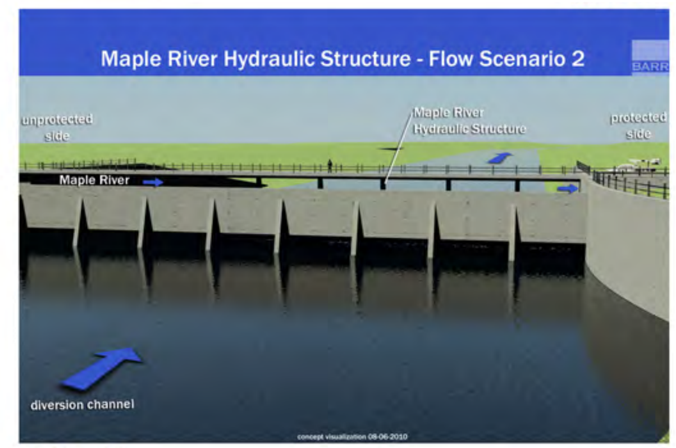
LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F50



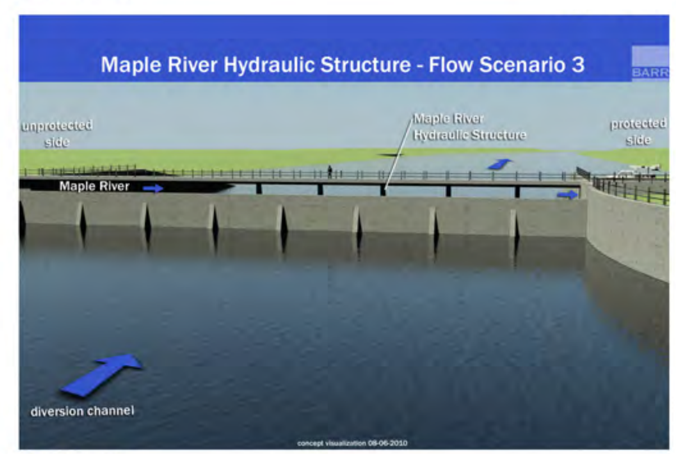
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F51



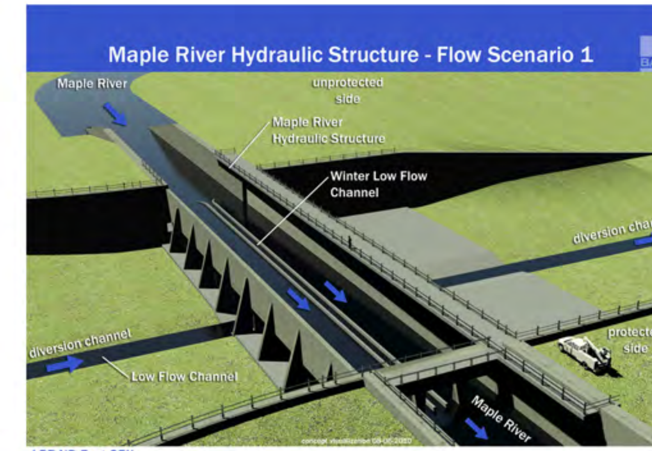
LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F52



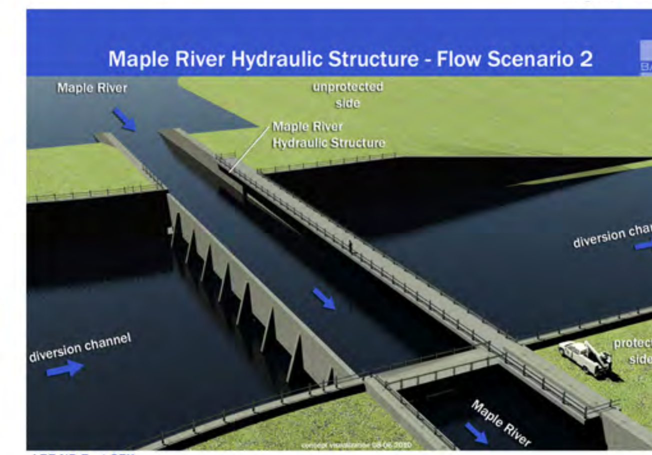
LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F53



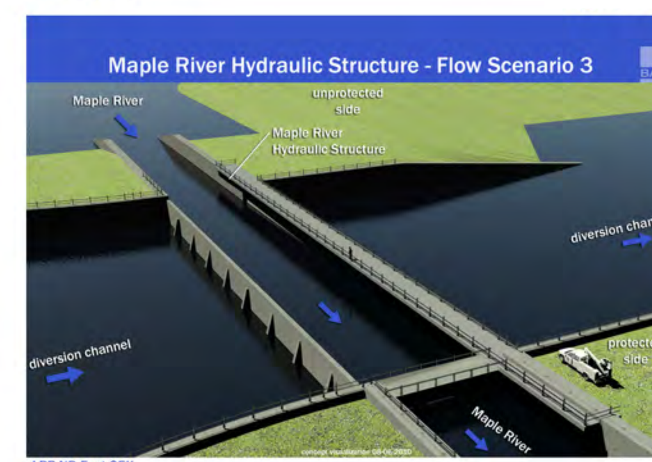
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F54



LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F55



LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F56

Figure F57
MOSAIC OF RENDERINGS OF
MAPLE RIVER HYDRAULIC STRUCTURES

Table F1 Major Hydraulic Structures per Alignment

Alignment	Hydraulic Structures
FCP (previously known as MN Short-35K)	Inlet to Extension Channel from Red River of the North
	Inlet to Diversion Channel from Red River of the North
	Control Structure on Red River of the North
	Outlet from Diversion Channel into Red River of the North
LPP	Control Structure on Red River of the North
	Control Structure on Wolverton Creek
	Control Structure in Wild Rice River
	Diversion Inlet Structure
	Inlet/Outlet to Storage Area 1
	Diversion Channel Transition and Aqueduct at the Sheyenne River
	Spillway from Sheyenne River to Diversion Channel
	Diversion Channel Transition and Aqueduct at the Maple River
	Spillway from Maple River to Diversion Channel
	Drop Structure at Lower Rush River to Diversion Channel
	Drop Structure at Rush River to Diversion Channel
	Outlet Structure from Diversion Channel into Red River of the North

Table F2 Preliminary design of Control Structures on the Red River of the North and Wild Rice River (for 500-year event flow on Red River of the North)

Alternative	Gate Invert (ft)	Gate Width (total) (ft)	Gate Height (ft)	Upstream 500-yr Water Surface Elevation (ft)	Downstream 500-yr Water Surface Elevation (ft)
FCP - Red River[(a)]	867.00	150	47	913.61	909.50
LPP - Red River[(b)]	875.51	150	50	922.33	914.84
LPP – Wild Rice River[(b)]	890.80	60	30	922.45	911.90

(a) Design of FCP Red River control structure from Phase 3 which was done using the HEC-RAS steady flow model
(b) Design of LPP control structures verified in Phase 4 using the HEC-RAS unsteady flow model

Table F3 Preliminary design of Inlet Structures of Diversion Channel at the Red River of the North (RRN) and the Wild Rice River (WRR)

Alternative	Location	Crest Elevation Weir #1 (ft)	Length Weir #1 (ft)	Crest Elevation Weir #2 (ft)	Length Weir #2 (ft)	Crest Elevation Weir #3 (ft)	Length Weir #3 (ft)	Total Length (ft)
FCP	East of RRN Inlet to Extension Channel	902.0	200	N/A	N/A	N/A	N/A	200
FCP	East of RRN Primary Inlet	898.3	180	905.0	280	911.0	330	330
LPP	East of WRR	902.25	60	N/A	N/A	N/A	N/A	60
LPP	Diversion Inlet Structure - West of WRR	903.25	90	N/A	N/A	N/A	N/A	90

N/A = Not applicable

Table F4 Red River of the North Fish Passage Operational Range Water Surface Elevations, Corresponding Interpolated Discharges, and Flood Frequency Data for 1901-2009 at the USGS Gage in Fargo, North Dakota

	Fish Passage Open Elevation (ft)	Fish Passage Closed Elevation (ft)	Pre-project Flow at Fargo for the Fish Passage Open, Q_{open}[a] (cfs)	Number of Events Q_{open} is Exceeded[b]	Number of Days Q_{open} is Exceeded[b]	Pre-project Flow at Fargo for the Fish Passage Closed, Q_{closed}[c] (cfs)	Number of Events Q_{closed} is Exceeded[b]	Number of Days Q_{closed} is Exceeded[b]
Pass 1 (~5-10 year)	913.9	917.4	12892	10	116	17883	6	44
Pass 2 (~20-50 year)	917.4	920.9	17883	6	44	24806	2	10

(a) Flow refers to the pre-project flow at the Fargo USGS Gage with the same recurrence interval as the post-project flow (and corresponding water elevation) at the Red River Control Structure at which the fish passage opens.
(b) As measured in the existing Fargo USGS gage record.
(c) Flow refers to the pre-project flow at the Fargo USGS Gage with the same recurrence interval as the post-project flow (and corresponding water elevation) at the Red River Control Structure at which the fish passage is closed.

Table F5 **Preliminary design of Aqueduct on Sheyenne River and related Transition to Concrete Lined Section in LPP Diversion Channel**

Description	Unit	LPP Alternative
Width of rectangular cross section of transition in Diversion Channel	(ft)	250
Effective width of aqueduct	(ft)	50
Length of rectangular cross section of crossing in Diversion Channel	(ft)	78
Width of trapezoidal cross section in Diversion Channel upstream of crossing	(ft)	250
Width of side slope of trapezoidal cross section in Diversion Channel upstream of crossing	(ft)	400
Width of trapezoidal cross section in Diversion Channel downstream of crossing	(ft)	250
Width of side slope of trapezoidal cross section in Diversion Channel downstream of crossing (including 40 ft benches on each side)	(ft)	500
Width of spillway weir that diverts water from tributary into Diversion Channel	(ft)	300
Invert of open aqueduct at crossing of rectangular cross section of transition in Diversion Channel	(ft)	898.70
Invert elevation in rectangular cross section of transition in Diversion Channel	(ft)	883.68
500-year water surface elevation in tributary at crossing of Diversion Channel	(ft)	914.9
500-year water surface elevation in Diversion Channel upstream of crossing	(ft)	905.2
500-year water surface elevation in Diversion Channel downstream of crossing	(ft)	904.7
Invert elevation in trapezoidal section (level above low flow channel) of Diversion Channel upstream of the crossing	(ft)	886.78
Invert elevation in trapezoidal section (level above low flow channel) of Diversion Channel downstream of the crossing	(ft)	886.37
Bottom of open aqueduct structure at crossing of rectangular cross section of transition in Diversion Channel	(ft)	896.70
Top of open aqueduct side wall at crossing of rectangular cross section of transition in Diversion Channel	(ft)	916
Number of openings in Diversion Channel at crossing		8

Table F6 Preliminary design of Aqueduct on Maple River and related Transition to Concrete Lined Section in LPP Diversion Channel

Description	Unit	LPP Alternative
Width of rectangular cross section of transition in Diversion Channel	(ft)	250
Effective width of aqueduct	(ft)	50
Length of rectangular cross section of crossing in Diversion Channel	(ft)	78
Width of trapezoidal cross section in Diversion Channel upstream of crossing	(ft)	250
Width of side slope of trapezoidal cross section in Diversion Channel upstream of crossing	(ft)	430
Width of trapezoidal cross section in Diversion Channel downstream of crossing	(ft)	250
Width of side slope of trapezoidal cross section in Diversion Channel downstream of crossing	(ft)	430
Width of spillway weir that diverts water from tributary into Diversion Channel	(ft)	300
Invert of open aqueduct at crossing of rectangular cross section of transition in Diversion Channel	(ft)	881.06
Invert elevation in rectangular cross section of transition in Diversion Channel	(ft)	872.06
500-year water surface elevation in tributary at crossing of Diversion Channel	(ft)	896.4
500-year water surface elevation in Diversion Channel upstream of crossing	(ft)	900.9
500-year water surface elevation in Diversion Channel downstream of crossing	(ft)	894.7
Invert elevation in trapezoidal section (level above low flow channel) of Diversion Channel upstream of the crossing	(ft)	875.06
Invert elevation in trapezoidal section (level above low flow channel) of Diversion Channel downstream of the crossing	(ft)	875.01
Bottom of open aqueduct structure at crossing of rectangular cross section of transition in Diversion Channel	(ft)	879.06
Top of open aqueduct side wall at crossing of rectangular cross section of transition in Diversion Channel	(ft)	902
Number of openings in Diversion Channel at crossing		8

Table F7 Preliminary design of 300 ft Weir Spillway to divert waters from Sheyenne River to LPP Diversion Channel (crest elevation 912.56 ft)

Event	Upstream Water Surface Elevation (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct Water Surface Elevation (ft)	Flow into Protected Area (cfs)
10-year Coincidental	913.5	1,783	812	913.5	971
50-year Coincidental	914.5	3,629	2,478	914.5	1,151
100-year Coincidental	914.8	4,176	3,016	914.8	1,160
500-year Coincidental	914.9	4,368	3,295	914.9	1,073
10-Year Local	914.4	3,996	2,180	914.3	1,816
50-Year Local	914.6	4,479	2,574	914.5	1,905
100-Year Local	914.7	4,526	2,609	914.5	1,918
500-Year Local	914.8	4,671	2,712	914.6	1,960

Table F8 Preliminary design of 300 ft Weir Spillway to divert waters from Maple River to LPP Diversion Channel (crest elevation 893.63 ft)

Event	Upstream Water Surface Elevation (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct Water Surface Elevation (ft)	Flow into Protected Area (cfs)
10-year Coincidental	896.0	5,478	2,747	894.9	2,732
50-year Coincidental	896.6	6,994	3,988	895.6	3,007
100-year Coincidental	896.6	7,079	4,088	895.7	2,991
500-year Coincidental	897.3	9,119	5,538	896.4	3,581
10-Year Local	895.9	5,206	2,645	894.7	2,561
50-Year Local	896.7	7,407	4,323	895.6	3,084
100-Year Local	896.8	7,595	4,473	895.7	3,122
500-Year Local	896.9	7,736	4,896	896.2	2,840

Table F9 Preliminary design of Stepped Drop to fully divert waters from Lower Rush River to LPP Diversion Channel

Description	Unit	LPP Alternative
Number of steps		16
Step height	(ft)	0.9
Step length	(ft)	1.5
Total length of steps	(ft)	24.0
Invert elevation of bed of Diversion Channel	(ft)	872.9
Elevation of crest of spillway	(ft)	886.8
Invert elevation of tributary	(ft)	885.4
Length of concrete stilling basin	(ft)	50
Width of concrete steps	(ft)	60

Table F10 Preliminary design of Stepped Drop to fully divert waters from Rush River to LPP Diversion Channel

Description	Unit	LPP Alternative
Number of steps		9
Step height	(ft)	1.1
Step length	(ft)	1.7
Total length of steps	(ft)	15.3
Invert elevation of bed of Diversion Channel	(ft)	870.6
Elevation of crest of spillway	(ft)	880.6
Invert elevation of tributary	(ft)	879.5
Length of concrete stilling basin	(ft)	50
Width of concrete steps	(ft)	100

Table F11 Phase 3: Gated Structures Load Cases

Load Case	Event Category	Allowable Pile Deflection (inches)	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	0.67 [(a)]	2.00	Undrained
1.1 – 100 yr + ice [(b)]	Unusual	0.875 [(a)]	1.50	Undrained
2 – 500 yr flood + 3ft	Unusual	0.875 [(a)]	1.50	Undrained
2.1 – 500 yr + 3ft + ice [(b)]	Extreme	0.875	1.15	Undrained
3 – construction	Unusual	0.67	1.50	Undrained
4 – Normal low flow	Usual	0.50	2.00	Drained

(a) It was agreed that an allowable deflection for the next higher event was acceptable.
(b) Static Ice loading based on 2 feet thick ice and 5,000 psf pressure

Table F12 Phase 4: Gated Structures Load Cases

Load Case	Event Category	Allowable Pile Deflection (inches)	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	0.67 [a]	2.00	Undrained
1.1 – 100 yr + ice [d, e]	Unusual	0.875 [b]	1.50	Undrained
2 – 500 yr flood	Unusual	0.875 [b]	1.50	Undrained
2.1 – Elevation 927	Extreme	1.000 [c]	1.15	Undrained
3 – construction	Unusual	0.67	1.50	Undrained
4 – Normal low flow	Usual	0.50	2.00	Drained

(a) It was agreed that an allowable deflection of 0.67-inches (as opposed to 0.50 inches) is acceptable even though this is considered a usual load case.

(b) It was agreed that an allowable deflection of 0.875-inches (as opposed to 0.67 inches) is acceptable even though this is considered an unusual load case.

(c) It was agreed that for this extreme event an allowable deflection of 1.0-inch is acceptable.

(d) Ice loads on the gated structure during the 100 year flood will be considered as dynamic forces due to crushing or bending of ice floes as provided by Andrew Tuthill from the USACE via email to Miguel Wong dated February 1, 2011 at 9:52AM. These loads will be applied to the piers.

(e) An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.

Notes:

- A 10-foot long SSP cut-off will be provided on upstream and downstream edges of gated structures. (i.e. 2 – rows of SSP)
- A 10-foot long SSP cut-off will be provided at the center of the wing and retaining wall structures. (i.e. 1 – row of SSP)
- The structure freeboard will extend 2 additional feet for a total height of 2-feet above elevation 927 (which contains the Standard Project Flood with nearly 2 feet of freeboard).
- Barr will complete a group pile analysis for 2-sections of the wing wall structures at the Red River of the North gated structure to better estimate the number and batter of piles required for the wing walls. Based on the results for these 2-sections at the Red River structure, we will estimate the piles required for the other gated structures and other wing wall sections.

Table F13 Comparison of Pile Reactions: Rigid Cap vs. GROUP

Load Case	Pile Row	Summary Pile Reactions 1[b]	2	3	4	5	6	7	8	Allowable Pile Loads
Control Structure: ND RRN Wing Wall Section A										
	Rigid Cap Analysis									
1	Axial load (tons/pile)	57.10	48.50	39.91	29.59	19.28	8.96	-1.35	-11.67	59.775
	Group									
	Axial Load (tons/pile)	***61.59***[a]	53.28	44.97	34.93	20.13	5.52	-8.91	-23.57	59.775
	Horiz Displace (inc.)	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	-0.5488	
	Mz max (k-ft)	193.33	173.33	173.33	181.67	183.33	183.33	183.33	183.33	
	Horiz Loads/pile Fy (k)	-33.57	-28.87	-28.91	-30.80	-31.32	-31.40	-31.47	-31.54	
	Rigid Cap Analysis									
1.1	Axial load (tons/pile)	57.90	49.09	40.27	29.70	19.13	8.55	-2.02	-12.59	79.70
	Group									
	Axial Load (tons/pile)	53.50	47.16	40.82	32.27	20.97	9.80	-1.23	-12.29	79.70
	Horiz Displace (inc.)	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	-0.5513	
	Mz max (k-ft)	196.67	176.67	176.67	185.00	186.67	186.67	186.67	186.67	
	Horiz Loads/pile Fy (k)	-33.922	-29.177	-29.206	-31.107	-31.619	-31.674	-31.73	-31.783	
Control Structure: ND RRN Wing Wall Section F										
	Rigid Cap Analysis									
1	Axial load (tons/pile)	47.04	34.70	22.36	10.02	-2.31				59.775
	Group									
	Axial Load (tons/pile)	55.62	40.70	25.78	5.42	-1.51				59.775
	Horiz Displace (inc.)	-0.4354	-0.4354	-0.4354	-0.4354	-0.4354				
	Mz max (k-ft)	152.50	140.00	139.17	140.83	139.17				
	Horiz Loads/pile Fy (k)	-28.61	-25.32	-25.23	-25.85	-25.55				

(a) Exceeds allowable capacity by 3 percent: Okay
(b) Row 1 includes 3"H:12"V batter

Notes:

- Axial load based on local axis of battered pile
- Based on global axis of piles

Table F14 Phase 4: Aqueduct Structures Load Cases

Load Case	Event Category	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	2.00	Undrained
2 – 100 yr + ice [a, b, d]	Unusual	1.50	Undrained
3 – 500 yr flood	Unusual	1.50	Undrained
4 – 500 yr (+match levee elevation) [e]	Extreme	1.15	Undrained
5 – construction	Unusual	1.50	Undrained
6 – Normal low flow [c]	Usual	2.00	Drained

(a) Ice loads on the structure in the diversion channel during the 100 year flood will be considered as dynamic forces due to crushing or bending of ice floes as provided by Andrew Tuthill from the USACE via email to Miguel Wong dated February 1, 2011 at 9:52AM. These loads will be applied to the piers.

(b) Ice loads within the crossing channel above the diversion will be considered for the 100-year flood elevation and be considered static 5,000 psf for 2-foot thick ice.

(c) Ice loads within the minimum flow channel will be considered static 5,000 psf for 2-foot thick ice at the top of the low flow walls.

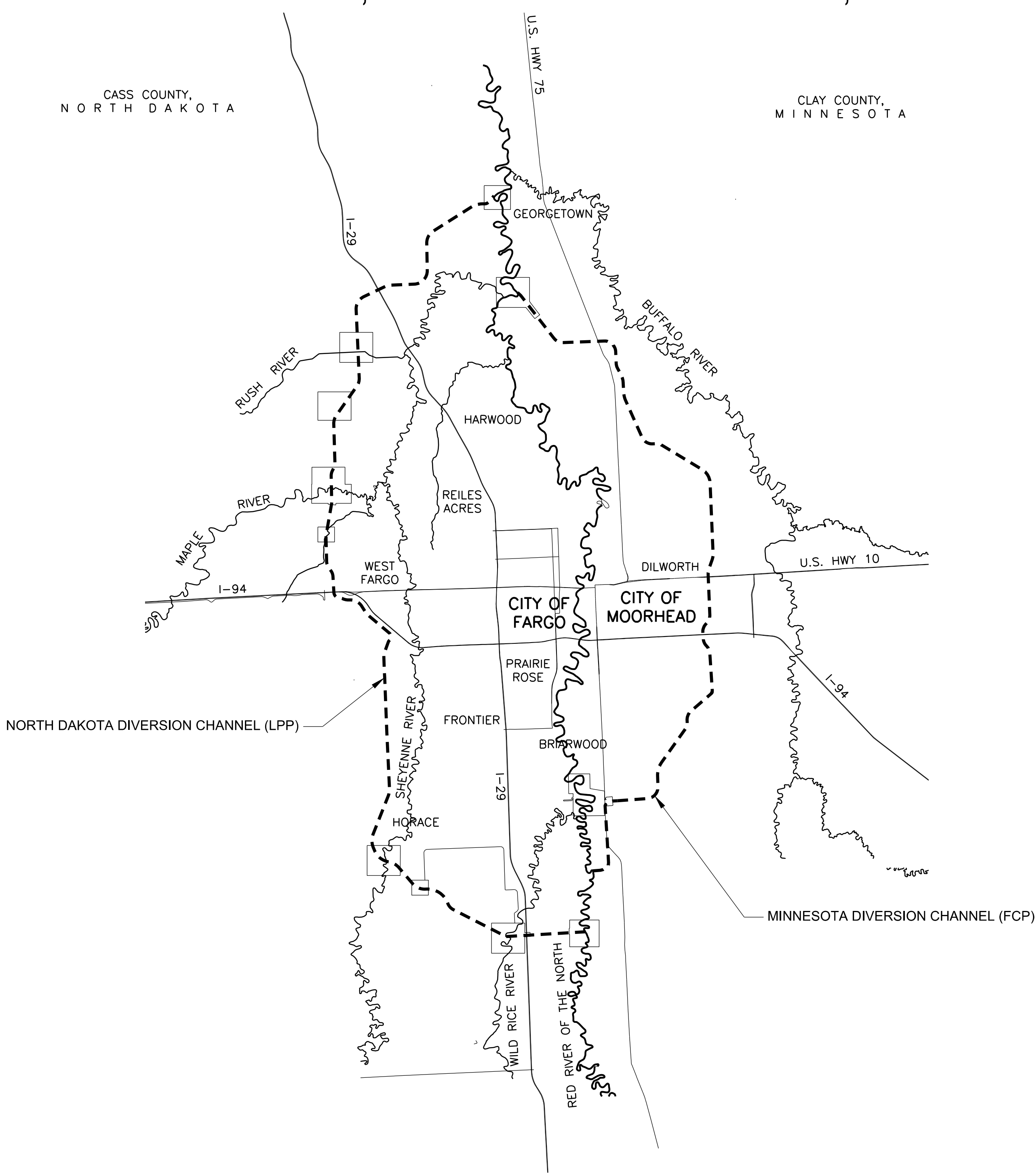
(d) An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.

(e) The top of structure design elevation will match the elevation of the highest levee adjacent to the structure.

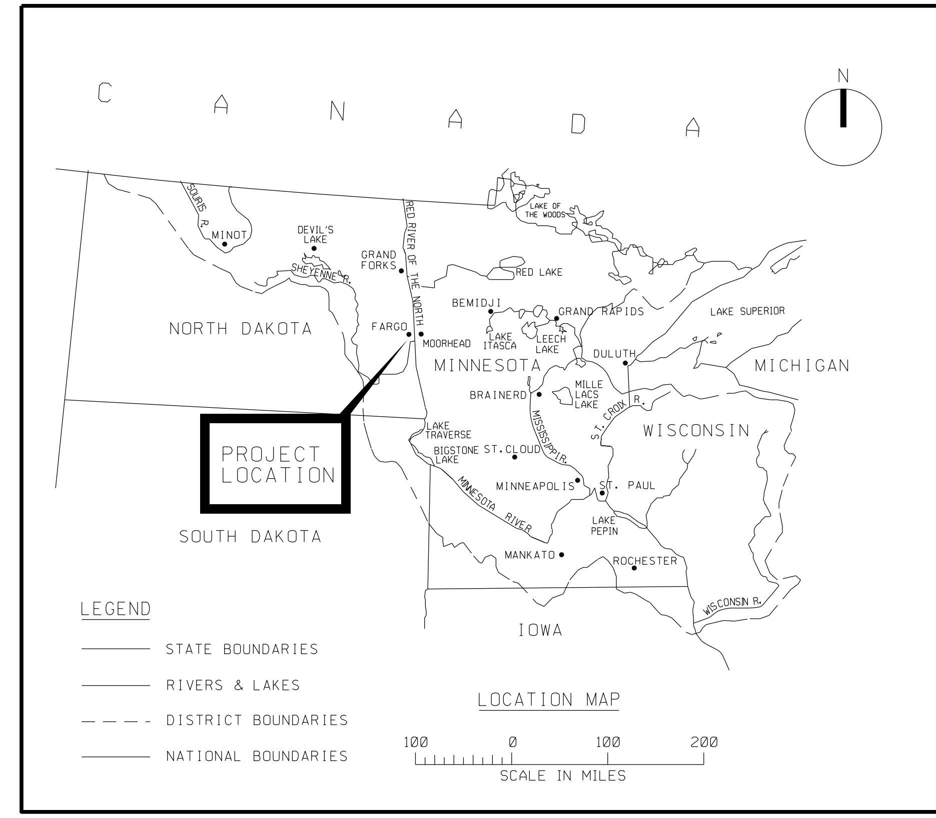
Notes:

- A 10-foot long SSP cut-off will be provided on upstream and downstream edges of the crossing structures. (i.e. 2 – rows of SSP)
- A 10-foot long SSP cut-off will be provided at the center of the wing and retaining wall structures. (i.e. 1 – row of SSP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER OF THE NORTH DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY – PHASE 4
FARGO, ND AND MOORHEAD, MN



VICINITY MAP



HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
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MARK	DESCRIPTION	DATE	APPR

NORTH DAKOTA DIVERSION (LPP) AND
MINNESOTA DIVERSION (FCP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: BARR
DWN BY: DMD
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_G-001.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT STUDY
FARGO, ND AND MOORHEAD, MN

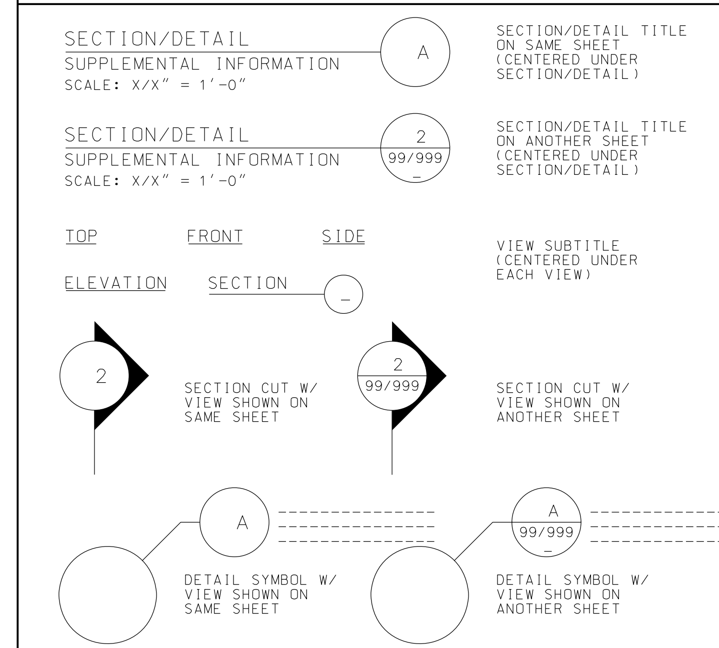
COVER SHEET
VICINITY MAP AND
LOCATION MAP

SHEET IDENTIFICATION
G-001

DRAWING INDEX

DRAWING NO.	SHEET	DESCRIPTION	CAD FILE
GENERAL DRAWINGS			
.	G-001	COVER SHEET, VICINITY MAP, LOCATION MAP	FMM_G-001HYD.DGN
.	G-002	DRAWING INDEX AND GENERAL INFORMATION	FMM_G-002HYD.DGN
.	G-003	OVERALL PROJECT LAYOUT	FMM_G-003HYD.DGN
MINNESOTA DIVERSION (FCP) DRAWINGS			
.	S-401	FCP CONTROL STRUCTURE ON RED RIVER OF THE NORTH, PLAN VIEW	FMM_S-401HYD.DGN
.	S-402	FCP CONTROL STRUCTURE ON RED RIVER OF THE NORTH, GATES PLAN, SECTION AND ELEVATION	FMM_S-402HYD.DGN
.	S-403	FCP CONTROL STRUCTURE ON RED RIVER OF THE NORTH, WING WALL PLAN, ELEVATION AND SECTION	FMM_S-403HYD.DGN
.	S-404	FCP FISH PASSAGE STRUCTURE ON RED RIVER OF THE NORTH, PLAN VIEW AND SECTIONS	FMM_S-404HYD.DGN
.	S-405	FCP WEIR STRUCTURE ON RED RIVER OF THE NORTH, PLAN VIEW AND SECTIONS	FMM_S-405HYD.DGN
.	S-406	FCP OUTLET STRUCTURE ON RED RIVER OF THE NORTH, PLAN VIEW	FMM_S-406HYD.DGN
NORTH DAKOTA DIVERSION (LPP) DRAWINGS			
.	S-407	LPP CONTROL STRUCTURE ON RED RIVER OF THE NORTH, PLAN VIEW	FMM_S-407HYD.DGN
.	S-408	LPP CONTROL STRUCTURE ON RED RIVER OF THE NORTH, GATES PLAN, SECTION AND ELEVATION	FMM_S-408HYD.DGN
.	S-409	LPP CONTROL STRUCTURE ON RED RIVER OF THE NORTH, WINGWALL PLAN, ELEVATION AND SECTION	FMM_S-409HYD.DGN
.	S-410	LPP FISH PASSAGE STRUCTURE ON RED RIVER OF THE NORTH, PLAN VIEW AND SECTIONS	FMM_S-410HYD.DGN
.	S-411	WOLVERTON CREEK CLOSURE/DRAINAGE STRUCTURE, PLAN VIEW	FMM_S-411HYD.DGN
.	S-412	WOLVERTON CREEK CLOSURE/DRAINAGE STRUCTURE, PLAN VIEW AND SECTIONS	FMM_S-412HYD.DGN
.	S-413	STORAGE AREA 1 INLET AND CONTROL STRUCTURE ON THE WILD RICE RIVER, PLAN VIEW	FMM_S-413HYD.DGN
.	S-414	CONTROL STRUCTURE ON THE WILD RICE RIVER, PLAN VIEW AND ELEVATION	FMM_S-414HYD.DGN
.	S-415	CONTROL STRUCTURE ON THE WILD RICE RIVER, SECTIONS	FMM_S-415HYD.DGN
.	S-416	FISH PASSAGE SYSTEM ON WILD RICE RIVER, PLAN AND SECTIONS	FMM_S-416HYD.DGN
.	S-417	WEIR STRUCTURE FROM CONNECTING CHANNEL TO WILD RICE RIVER, PLAN VIEW AND ELEVATION	FMM_S-417HYD.DGN
.	S-418	STORAGE AREA 1, PLAN VIEW	FMM_S-418HYD.DGN
.	S-419	STORAGE AREA 1, PLAN VIEW AND EMBANKMENT SECTION	FMM_S-419HYD.DGN
.	S-420	STORAGE AREA 1, TYPICAL CLOSURE/DRAINAGE STRUCTURE, PLAN VIEW AND SECTIONS	FMM_S-420HYD.DGN
.	S-421	INLET STRUCTURE TO DIVERSION, PLAN VIEW	FMM_S-421HYD.DGN
.	S-422	INLET STRUCTURE TO DIVERSION, PLAN VIEW AND SECTION	FMM_S-422HYD.DGN
.	S-423	INLET STRUCTURE TO DIVERSION, ELEVATIONS AND SECTIONS	FMM_S-423HYD.DGN
.	S-424	SHEYENNE RIVER AQUEDUCT, PLAN VIEW	FMM_S-424HYD.DGN
.	S-425	SHEYENNE RIVER AQUEDUCT, FOUNDATION PLAN, PLAN VIEW	FMM_S-425HYD.DGN
.	S-426	SHEYENNE RIVER AQUEDUCT, GENERAL ARRANGEMENT PLAN, PLAN VIEW	FMM_S-426HYD.DGN
.	S-427	SHEYENNE RIVER AQUEDUCT, SECTIONS	FMM_S-427HYD.DGN
.	S-428	SHEYENNE RIVER AQUEDUCT, SECTIONS CONT'D	FMM_S-428HYD.DGN
.	S-429	SHEYENNE RIVER AQUEDUCT, SPILLWAY WEIRS	FMM_S-429HYD.DGN
.	S-430	DRAIN 14 DROP STRUCTURE, PLAN VIEW AND SECTIONS	FMM_S-430HYD.DGN
.	S-431	MAPLE RIVER AQUEDUCT, PLAN VIEW	FMM_S-431HYD.DGN
.	S-432	MAPLE RIVER AQUEDUCT, FOUNDATION PLAN, PLAN VIEW	FMM_S-432HYD.DGN
.	S-433	MAPLE RIVER AQUEDUCT, GENERAL ARRANGEMENTPLAN, PLAN VIEW	FMM_S-433HYD.DGN
.	S-434	MAPLE RIVER AQUEDUCT, SECTIONS	FMM_S-434HYD.DGN
.	S-435	MAPLE RIVER AQUEDUCT, SECTIONS CONT'D	FMM_S-435HYD.DGN
.	S-436	MAPLE RIVER AQUEDUCT, SPILLWAY WEIRS	FMM_S-436HYD.DGN
.	S-437	LOWER RUSH RIVER DROP STRUCTURE, PLAN VIEW	FMM_S-437HYD.DGN
.	S-438	LOWER RUSH RIVER DROP STRUCTURE, PLAN AND ELEVATIONS	FMM_S-438HYD.DGN
.	S-439	LOWER RUSH RIVER DROP STRUCTURE, SECTIONS	FMM_S-439HYD.DGN
.	S-440	LOWER RUSH RIVER FISH PASSAGE SYSTEM, PLAN, PROFILE AND SECTION	FMM_S-440HYD.DGN
.	S-441	RUSH RIVER DROP STRUCTURE, PLAN VIEW	FMM_S-441HYD.DGN
.	S-442	RUSH RIVER DROP STRUCTURE, PLAN AND ELEVATIONS	FMM_S-442HYD.DGN
.	S-443	RUSH RIVER DROP STRUCTURE, SECTIONS	FMM_S-443HYD.DGN
.	S-444	RUSH RIVER FISH PASSAGE SYSTEM, PLAN VIEW AND PROFILE	FMM_S-444HYD.DGN
.	S-445	LPP OUTLET STRUCTURE ON THE RED RIVER OF THE NORTH, PLAN VIEW	FMM_S-445HYD.DGN
.	S-446	LPP OUTLET STRUCTURE ON THE RED RIVER OF THE NORTH, PLAN VIEW AND SECTION	FMM_S-446HYD.DGN
.	S-447	LPP OUTLET STRUCTURE ON THE RED RIVER OF THE NORTH, ELEVATIONS AND SECTION	FMM_S-447HYD.DGN
.	S-448	LOW FLOW CHANNEL DOWNSTREAM OF LOWER RUSH RIVER, PLAN VIEW AND SECTIONS	FMM_S-448HYD.DGN
.	S-449	SIDE CHANNEL CONTROL STRUCTURE (TYPICAL) PLAN VIEW AND SECTIONS	FMM_S-449HYD.DGN
.	S-450	SIDE INLET 72" RCP (TYPICAL) PLAN VIEW AND SECTIONS	FMM_S-450HYD.DGN
.	S-451	SIDE INLET 2-72" RCP (TYPICAL) PLAN VIEW AND SECTIONS	FMM_S-451HYD.DGN

GENERAL CROSS-REFERENCING SYMBOLS



ABBREVIATIONS

ALUM.	ALUMINUM	PC	POINT OF CURVATURE
C.I.P.	CAST IN PLACE	PI	POINT OF INTERSECTION
CL.	CLEAR COVER	PT	POINT OF TANGENCY
CL 2	CLASS 2	POB	POINT OF BEGINNING
CONC.	CONCRETE	PVI	POINT OF VERTICAL INTERSECTION
DIA.	DIAMETER	RCPP	REINFORCED CONCRETE PRESSURE PIPE
DIP	DUCTILE IRON PIPE	R.O.W.	RIGHT-OF-WAY
EL.	ELEVATION	RR	RAILROAD
FES	FLARED END SECTION	S	SLOPE
GALV.	GALVANIZED	SPPA	STRUCTURAL PLATE PIPE ARCH
HGL	HYDRAULIC GRADE LINE	STA	STATION
L	LENGTH	TYP.	TYPICAL
MIN.	MINIMUM	USFWS	U.S. FISH AND WILDLIFE SERVICE
MSL	MEAN SEA LEVEL		
NAD83	NORTH AMERICAN DATUM - 1983		
NAVD88	NORTH AMERICAN VERTICAL DATUM - 1988		

USFWS COWARDIN WETLAND CLASSIFICATIONS:

PABF	PALUSTRINE/AQUATIC BED/SEMIPERMANENTLY FLOODED
PEMA	PALUSTRINE/EMERGENT/TEMPORARILY FLOODED
PEMC	PALUSTRINE/EMERGENT/SEASONALLY FLOODED
PFOA	PALUSTRINE/FORESTED/TEMPORARILY FLOODED

GENERAL LEGEND

- BORING LOCATION
- PROJECT ALIGNMENT POINT
- BENCH MARK
- MANHOLE
- EXISTING STRUCTURE (UNSPECIFIED)
- WATER SURFACE (PROFILE)
- FILL
- CUT
- RIPRAP/ROCK
- DRAINAGE CULVERT
- LIMITS OF WORK
- E — EXISTING OVERHEAD POWER LINE
- UNDERGROUND POWER
- EXISTING GROUND SURFACE (PROFILE)
- PROPOSED GROUND SURFACE (PROFILE)
- EDGE OF WATER
- WATERMAIN
- TELEPHONE
- ACCESS ROAD
- 900 — EXISTING GRADE ELEVATION CONTOUR
- 900 — PROPOSED FINISH GRADE ELEVATION CONTOUR

NORTH DAKOTA DIVERSION (LPP) AND MINNESOTA DIVERSION (FCP)

US Army Corps of Engineers®
St. Paul District

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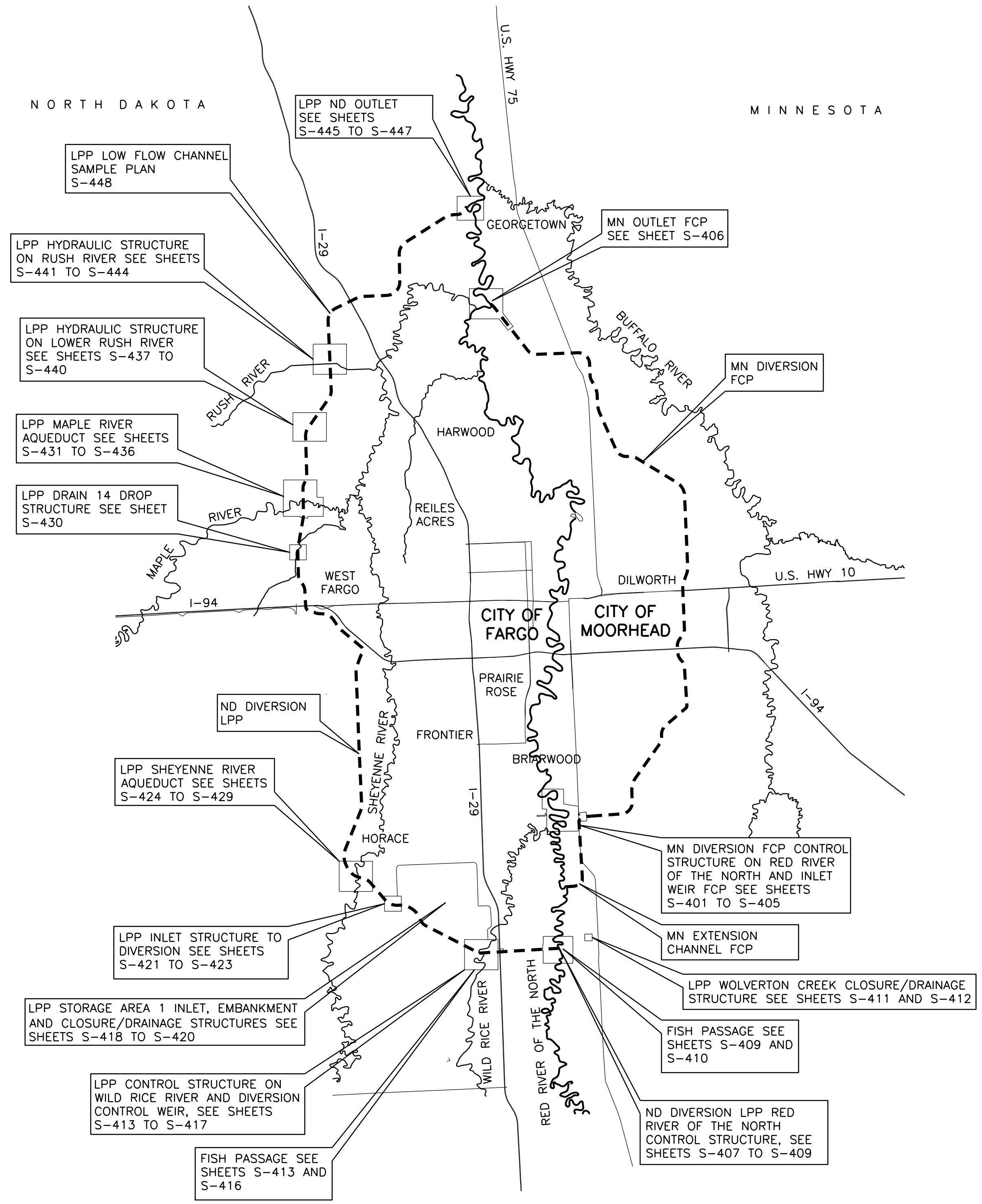
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: BARR	DWN BY: DMD	CKD BY: BKL	DATE: 28FEB2011
SUBMITTED BY: N/A			SOLICITATION NO.: N/A
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011		CONTRACT NO.: N/A
SIZE: ANSI D	FILE NAME: FMM_G-002.dgn		FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

DRAWING INDEX
AND GENERAL INFORMATION

SHEET IDENTIFICATION
G-002



OVERALL PROJECT LAYOUT

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

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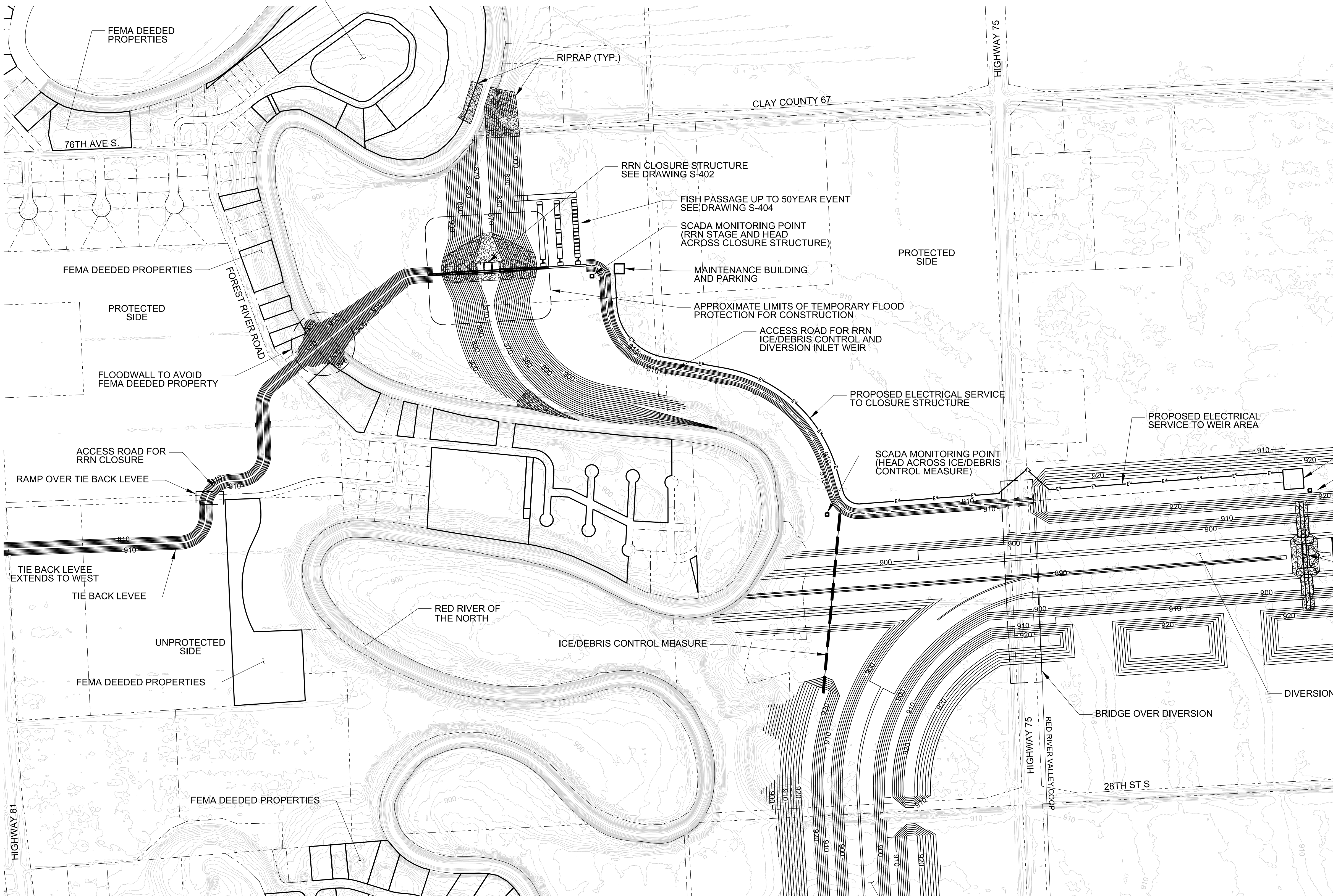
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011	FILE NUMBER: N/A
SIZE: ANSI D	FILE NAME: FMM_G-003.dgn	

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILTY STUDY
FARGO, ND AND MOORHEAD, MN

OVERALL PROJECT LAYOUT

SHEET IDENTIFICATION
G-003



NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PLANNING STAGE (PHASE 3 FEASIBILITY DESIGN). IT IS FOR A DIVERSION ON THE MINNESOTA SIDE AND NO FLOOD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.

PLAN

RRN - MINNESOTA INLET STRUCTURE

400 0 400 800
SCALE IN FEET

N

ICE/DEBRIS CONTROL MEASURE AT ENTRANCE TO EXTENSION CHANNEL (NOT SHOWN) SITE INCLUDES ACCESS ROAD, POWER AND SCADA

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

MINNESOTA DIVERSION (FCP)

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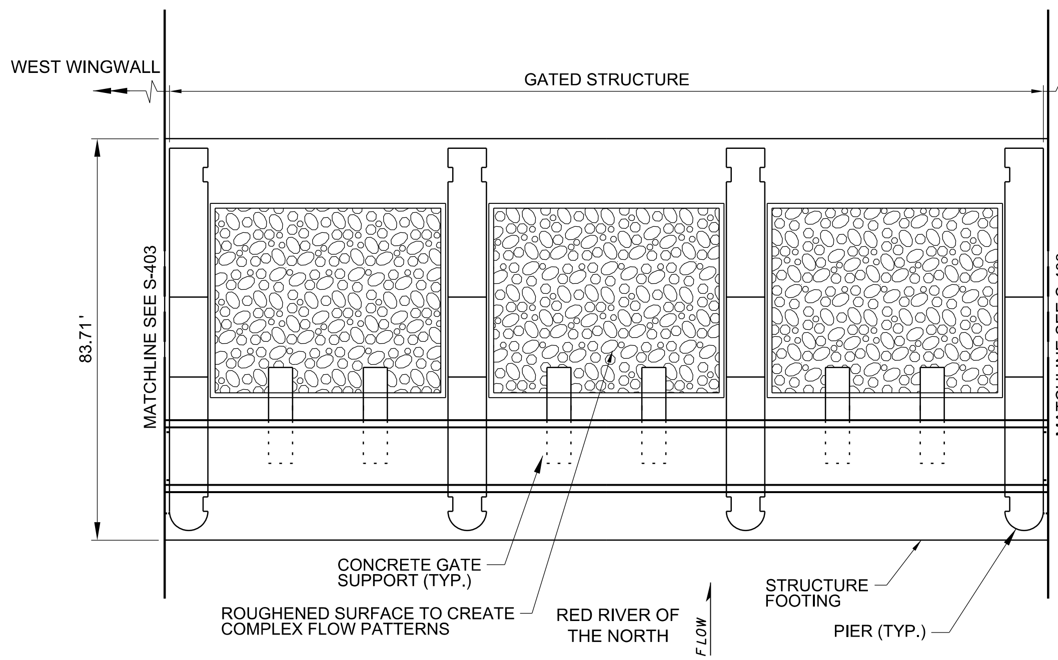
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MW1/PKN
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-401HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - FCP CONTROL STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW

SHEET IDENTIFICATION
S-401

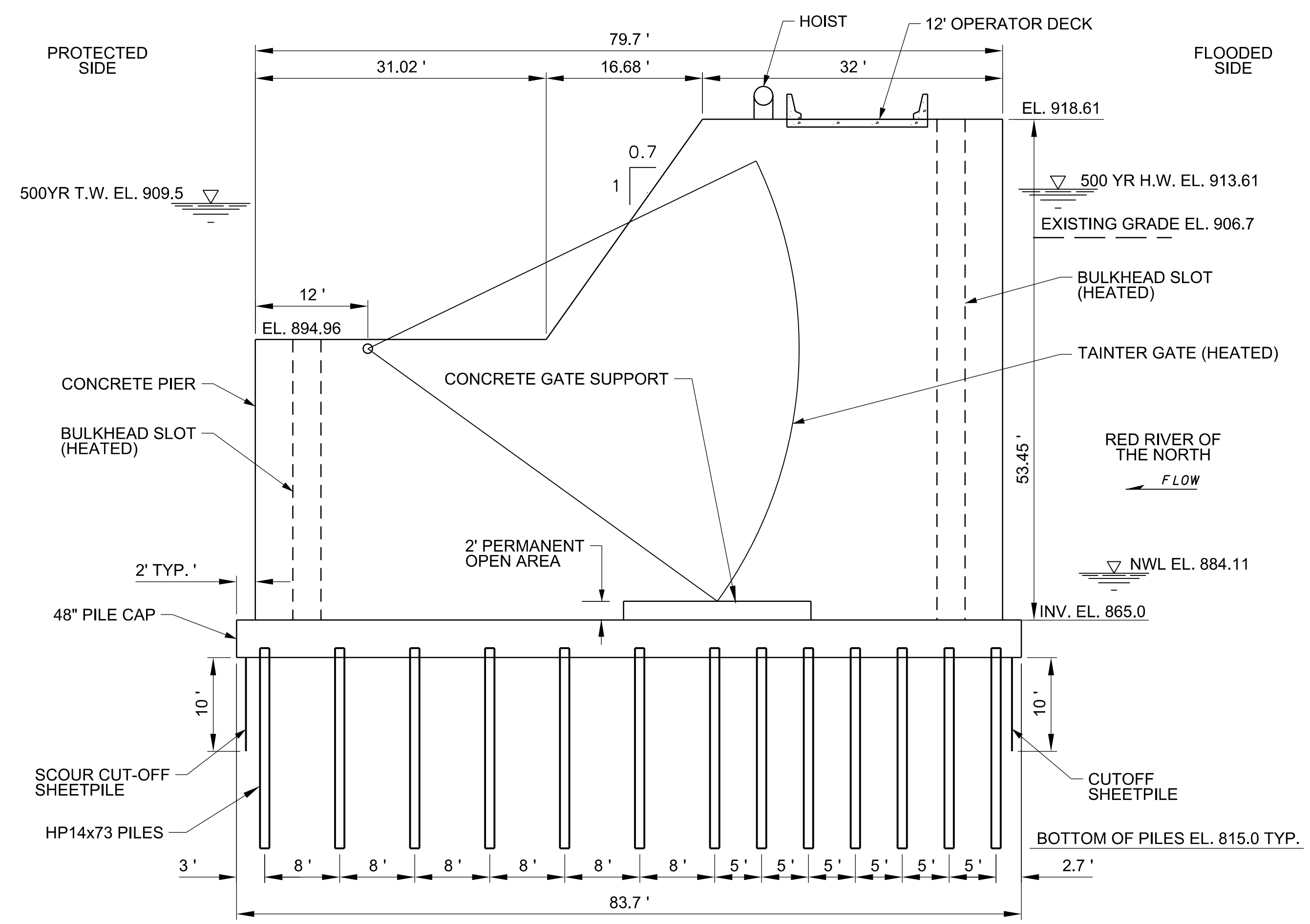


PLAN

RRN - MINNESOTA CONTROL STRUCTURE
GATED SECTION

20 0 20 40
SCALE IN FEET

N

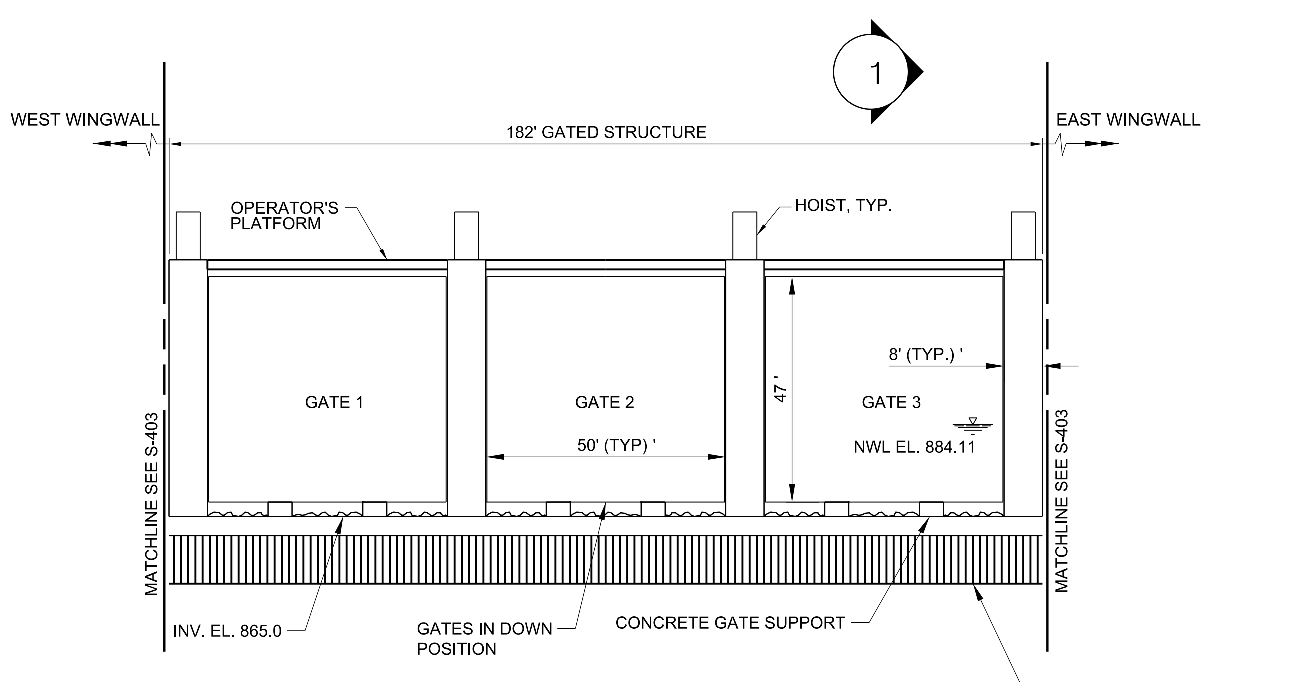


1

SECTION

RRN CONTROL GATED STRUCTURE
SCALE: N.T.S.

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	67.4
UNDRAINED UNUSUAL	89.8
UNDRAINED EXTREME	117.1
DRAINED USUAL	35.4



ELEVATION

RRN - MINNESOTA CONTROL STRUCTURE
GATED SECTION

20 0 20 40
SCALE IN FEET

NOTE: PILES NOT SHOWN FOR CLARITY.

NOTES:

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QUANTITIES (GATED STRUCTURE)

CONCRETE VOLUMES			STEEL	PILES			
FTG.	STEM	DECK	REINF.		NO.	LENGTH	TOTAL
CY	CY	CY	LBS	NO.	ROWS	FT	FT
2,257	3,953	128	475,140	286	13	47	13,442

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

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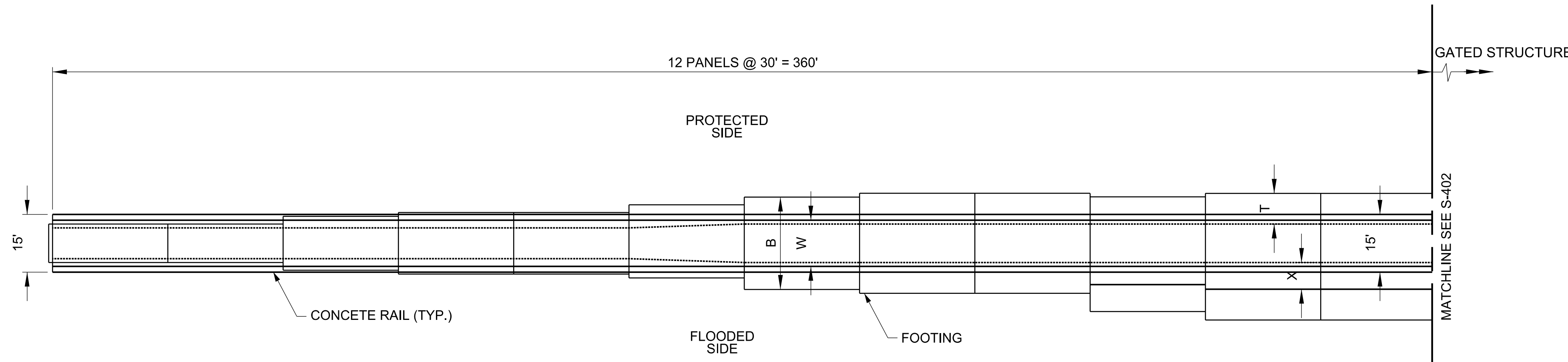
DESIGNED BY: MW1/PKN		DATE: 28FEB2011
DWN BY: MJJ	CKD BY: PKN/BKL	SOLICITATION NO.: N/A
SUBMITTED BY: N/A		CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011	FILE NUMBER: N/A
SIZE: ANSI D	FILE NAME: FMM_S-402HYD.dgn	

MINNESOTA DIVERSION (FCP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - FCP CONTROL STRUCTURE ON RED RIVER OF THE NORTH GATES PLAN, SECTION AND ELEVATION

SHEET IDENTIFICATION
S-402

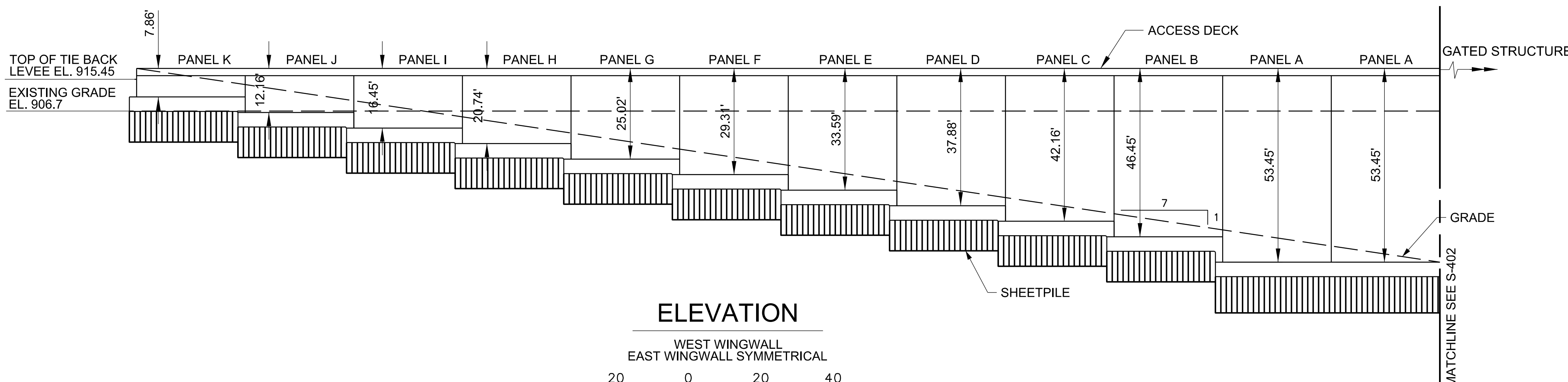


PLAN

WEST WINGWALL
(EAST WINGWALL SYMMETRICAL)

20 0 20 40
SCALE IN FEET

N

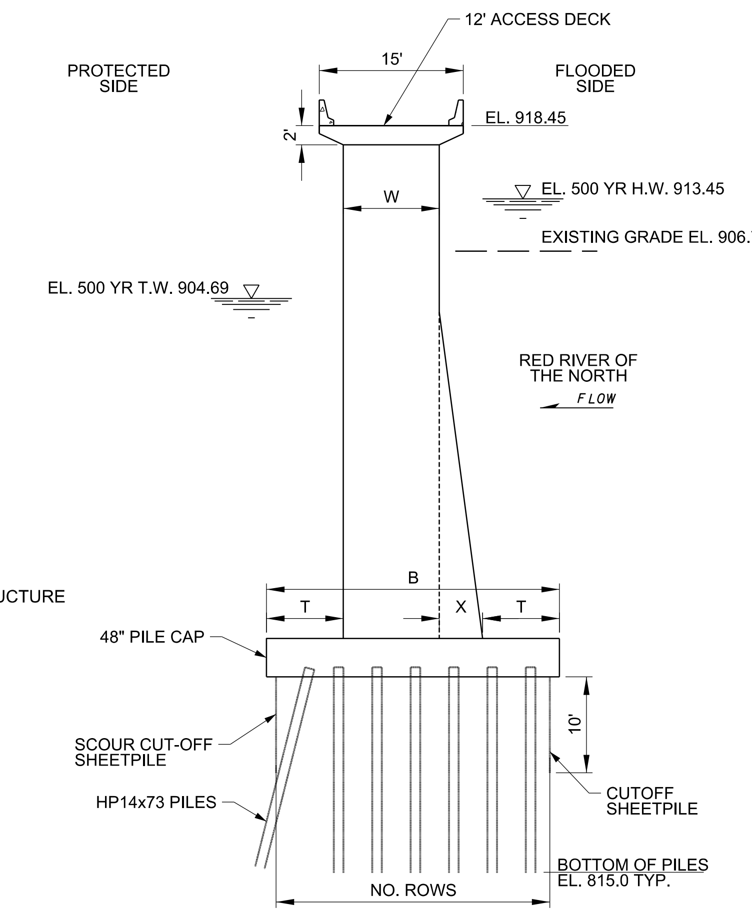


ELEVATION

WEST WINGWALL
EAST WINGWALL SYMMETRICAL

20 0 20 40
SCALE IN FEET

NOTE: PILES AND CONCRETE RAILS NOT SHOWN FOR CLARITY.



1 SECTION

RRN CONTROL STRUCTURE WINGWALL
SCALE: N.T.S.

QUANTITIES (WINGWALLS)

PANEL	STEM W (FT)	STEM BATTER X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. CY	STEM CY	DECK CY	STEEL REINF. LBS	PILES NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	10	6.92	32.9	8	302	1462	60	135,927	84	7	47	3,948
B	10	5.75	29.8	7	141	604	30	64,660	37	6	54	1,998
C	10	0	26.0	8	123	446	30	58,583	33	6	58	1,914
D	10	0	26.0	8	123	399	30	56,019	30	5	63	1,890
E	10	0	24.0	7	114	351	30	51,511	27	5	67	1,809
F	9	0	19.0	5	90	273	29	43,471	24	4	71	1,704
G	8	0	16.0	4	76	205	29	36,302	20	3	75	1,500
H	8	0	16.0	4	76	167	29	33,971	16	3	80	1,280
I	8	0	14.0	3	66	128	29	27,812	13	3	84	1,092
J	8	0	10.0	1	47	90	29	22,029	11	2	88	968
K	8	0	10.0	1	47	52	29	20,134	8	2	93	744
			WEST WINGWALL		1,205	4,177	354	550,419	303			18,847
			EAST WINGWALL		1,205	4,177	354	550,419	303			18,847
			TOTAL		2,410	8,354	708	1,100,838	606			37,694

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	67.4
UNDRAINED UNUSUAL	89.8
UNDRAINED EXTREME	117.1
DRAINED USUAL	35.4

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PLANNING STAGE (PHASE 3 FEASIBILITY DESIGN). IT IS FOR A DIVERSION ON THE MINNESOTA SIDE AND NO FLOOD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

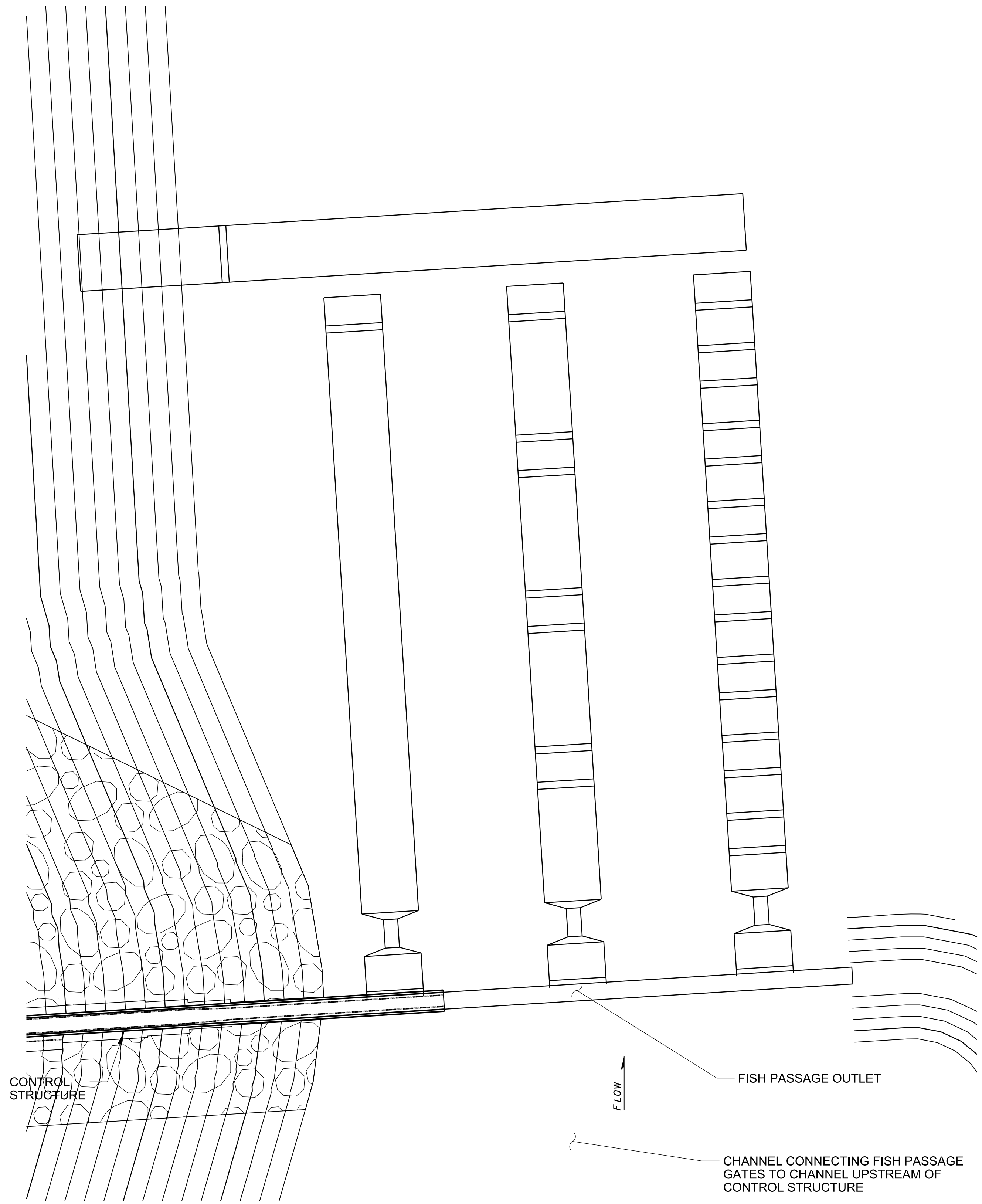
DESIGNED BY: MWT/PKN
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-403HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

MINNESOTA DIVERSION (FCP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - FCP CONTROL STRUCTURE ON RED RIVER OF THE NORTH WINGWALL PLAN,ELEVATION AND SECTION

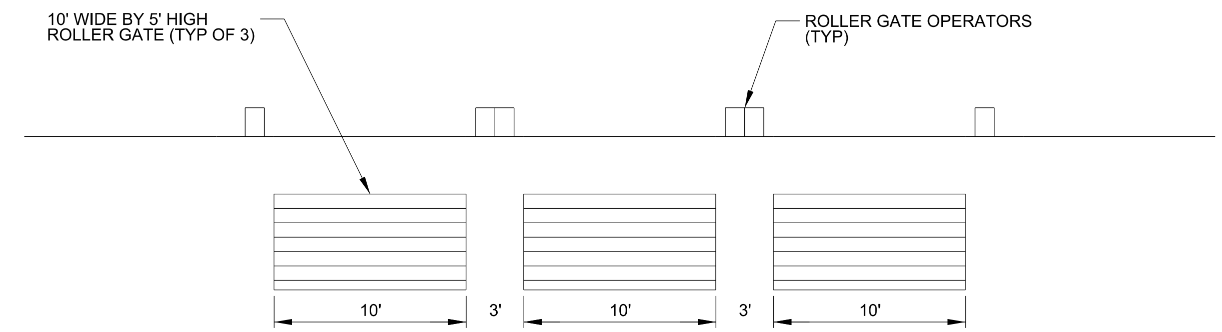
SHEET IDENTIFICATION
S-403



PLAN VIEW: FISH PASSAGE FCP

50 0 50 100
SCALE IN FEET

N



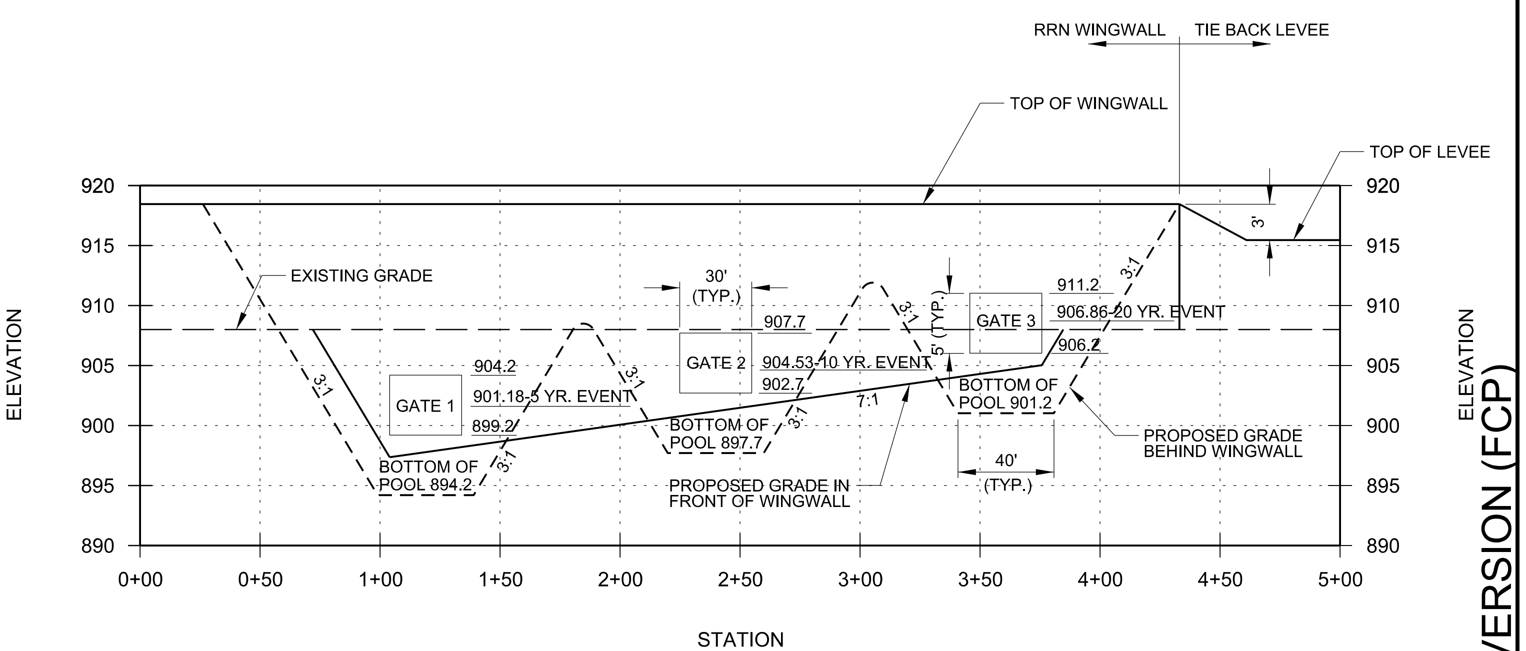
ELEVATION

TYPICAL FISH PASSAGE GATE ARRANGEMENT FOR ONE OF THE THREE FISH PASSAGES

5 0 5 10
SCALE IN FEET

NOTE:

EACH FISH PASSAGE WILL CONSIST OF THREE 10' WIDE BY 5' HIGH ROLLER GATES. ALL THREE WILL BE OPENED OR CLOSED TOGETHER TO FACILITATE FISH PASSAGE FOR THE ASSOCIATED FISH RAMP.



1 SECTION

MINNESOTA FCP CONTROL FISH PASSAGE
SCALE: NTS

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PLANNING STAGE (PHASE 3 FEASIBILITY DESIGN). IT IS FOR A DIVERSION ON THE MINNESOTA SIDE AND NO FLOOD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

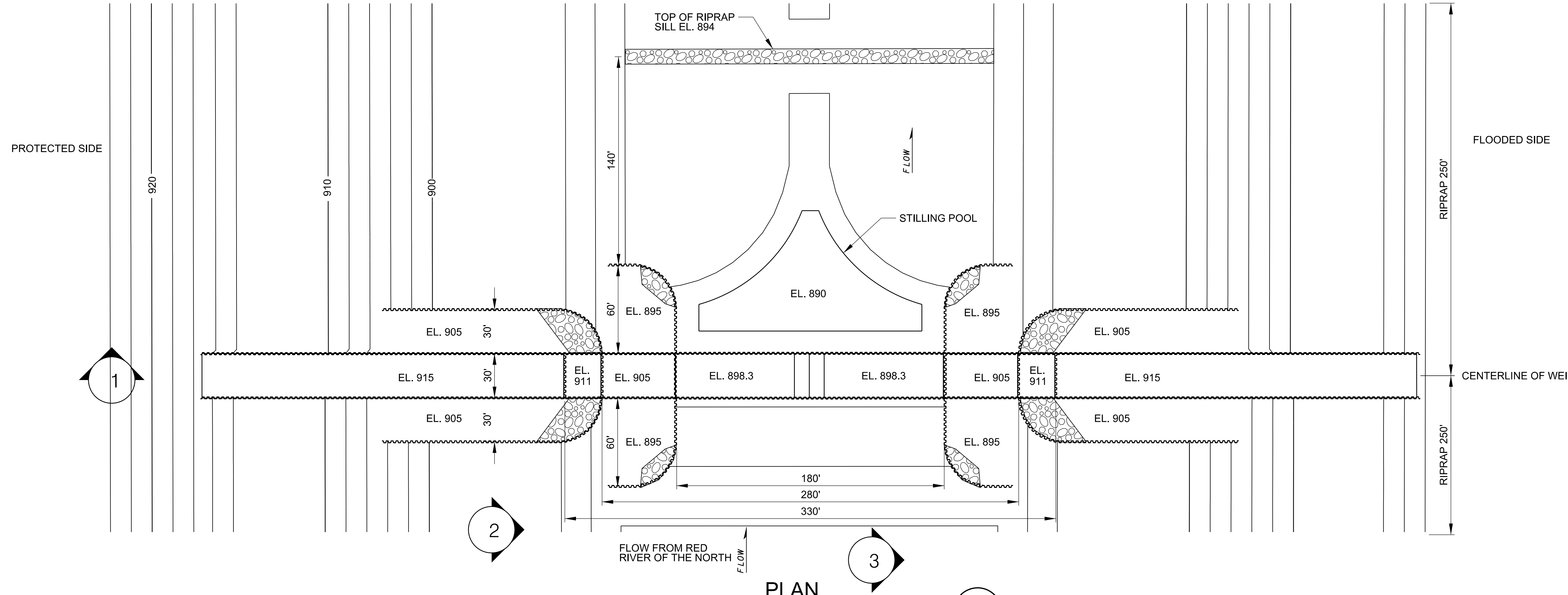
DESIGNED BY: MRM/MW1/PKN
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-404HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

MINNESOTA DIVERSION (FCP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - FCP FISH PASSAGE
STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW AND SECTIONS

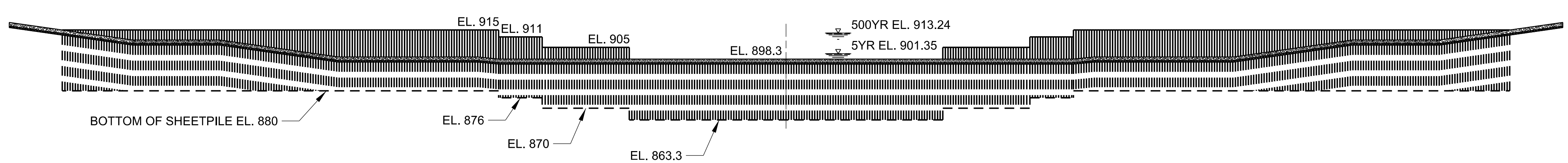
SHEET IDENTIFICATION
S-404



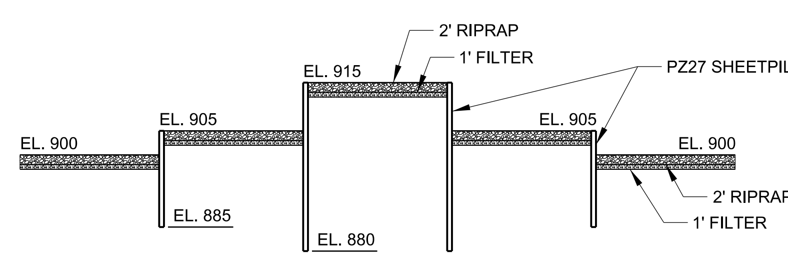
PLAN

RRN - MINNESOTA DIVERSION INLET WEIR STRUCTURE

50 0 50 100
SCALE IN FEET



1 SECTION



2 SECTION

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. CENTER OF WEIR TO HAVE V-NOTCH TO PREVENT SUDDEN TERMINATION OF FLOW.

4. RIPRAP EXTEND 250' UPSTREAM AND DOWNSTREAM OF WEIR.

5. WEIR IS STEPPED TO CONTROL UPSTREAM FLOOD STAGE.

6. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PLANNING STAGE (PHASE 3 FEASIBILITY DESIGN). IT IS FOR A DIVERSION ON THE MINNESOTA SIDE AND NO FLOOD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

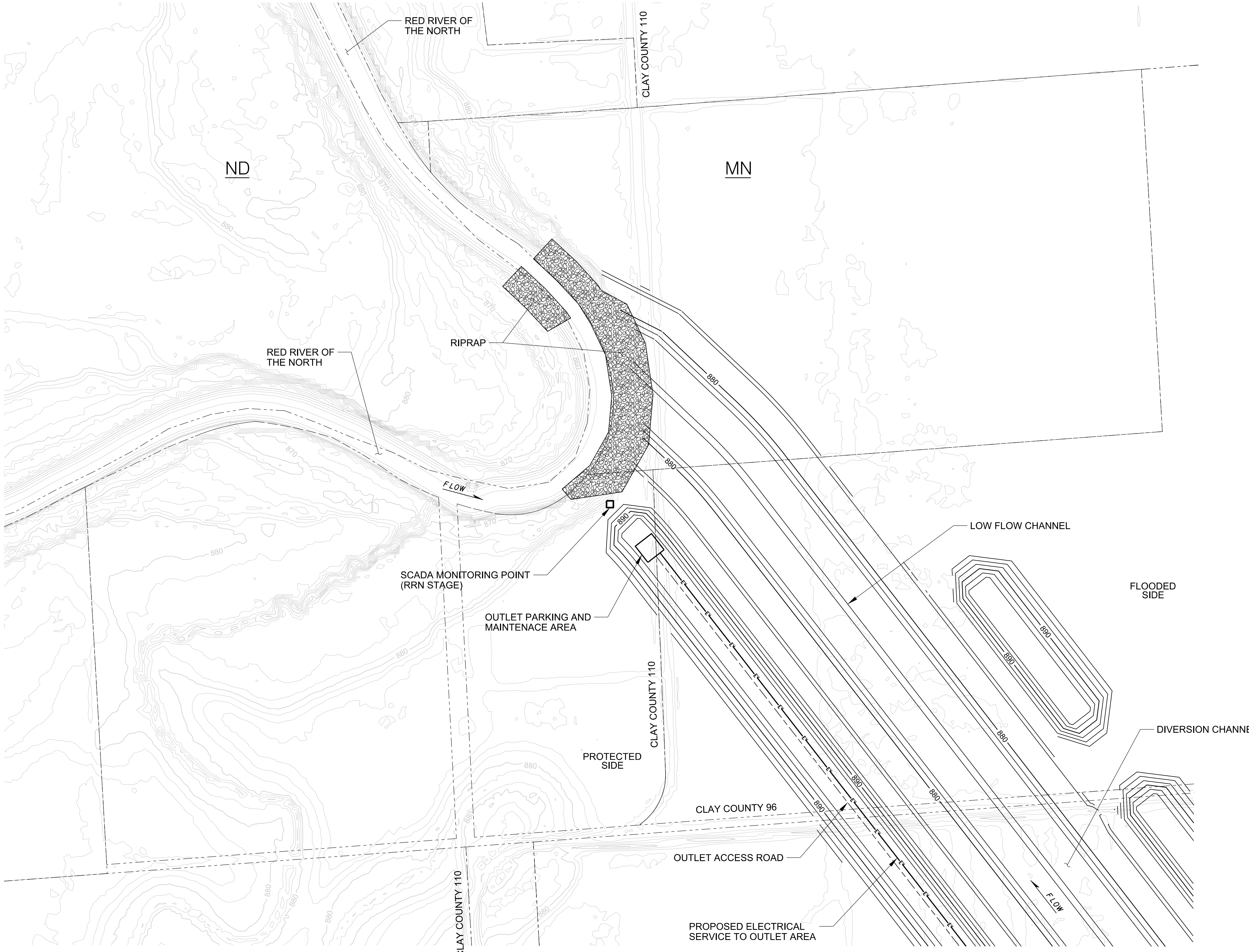
DESIGNED BY: MWT/PKN/BKL
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY:
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-405H10.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

MINNESOTA DIVERSION (FCP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - FCP WEIR
STRUCTURE ON RED RIVER OF THE NORTH
DIVERSION - PLAN VIEW & SECTIONS

SHEET IDENTIFICATION
S-405



PLAN VIEW

RRN - MINNESOTA OUTLET STRUCTURE

200 0 200 400
SCALE IN FEET

N

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PLANNING STAGE (PHASE 3 FEASIBILITY DESIGN). IT IS FOR A DIVERSION ON THE MINNESOTA SIDE AND NO FLOOD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

MINNESOTA DIVERSION (FCP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

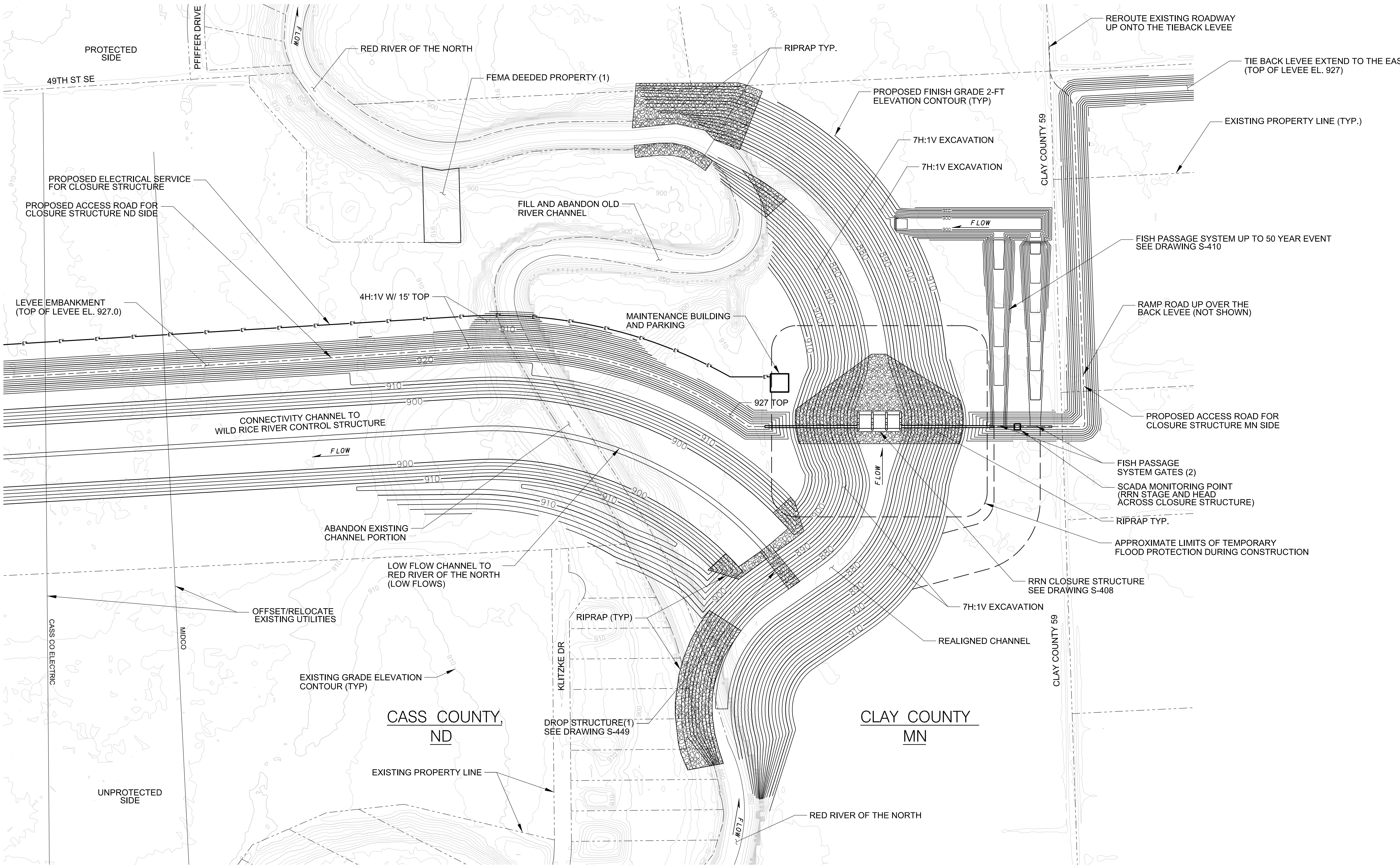
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MW1/PKN
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-406HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - FCP OUTLET STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW

SHEET IDENTIFICATION
S-406



PLAN VIEW

RRN -NORTH DAKOTA CONTROL STRUCTURE

200 0 200 400

SCALE IN FEET

N

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MRM/MWT/PKN
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-407HY0.dgn

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

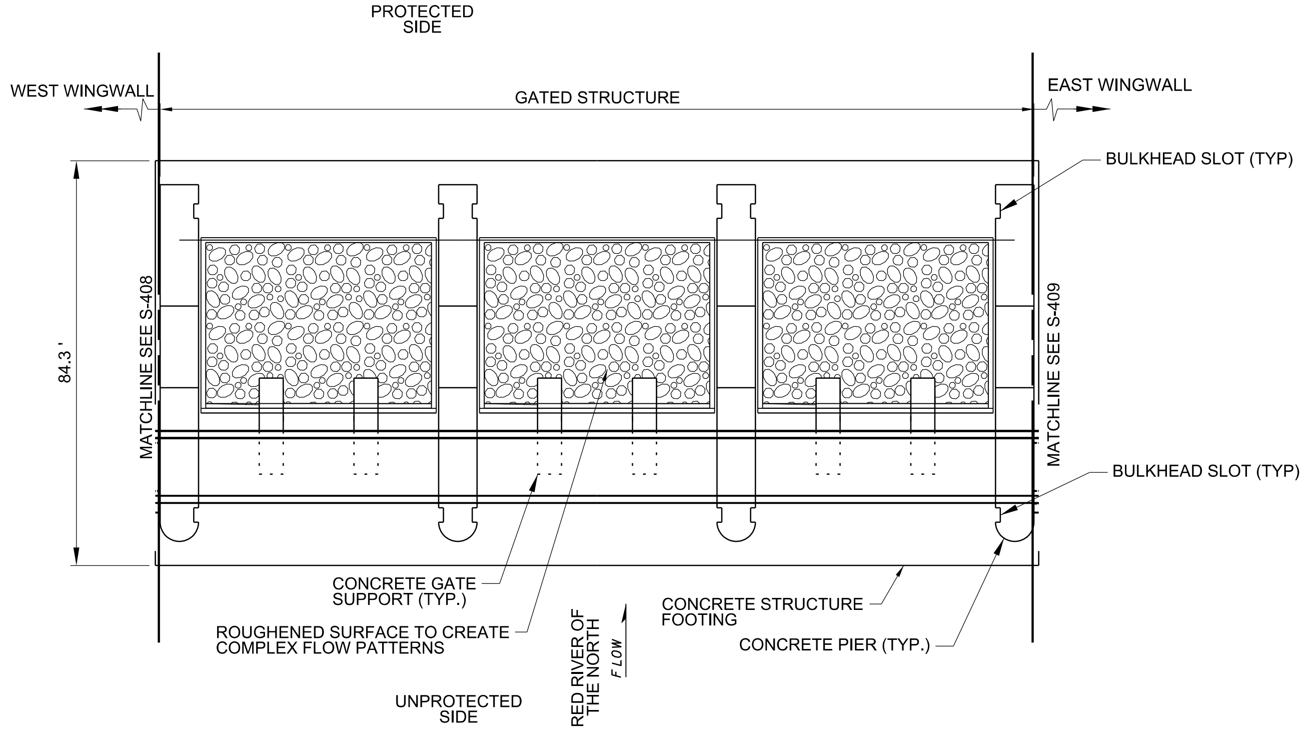
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LPP CONTROL STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW

SHEET IDENTIFICATION
S-407

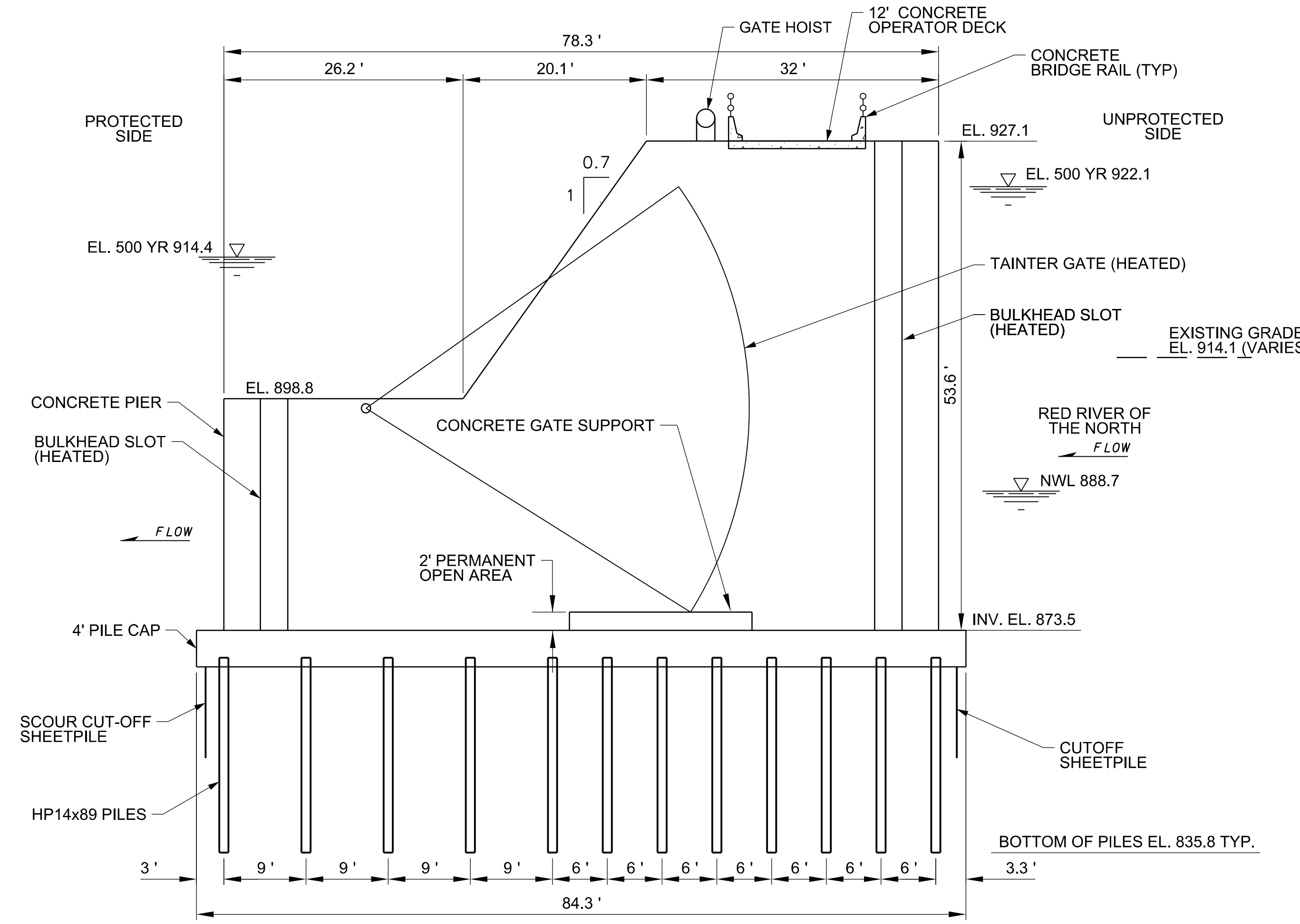


PLAN

NORTH DAKOTA LPP RRN CONTROL STRUCTURE
GATED SECTION

20 0 20 40
SCALE IN FEET

N



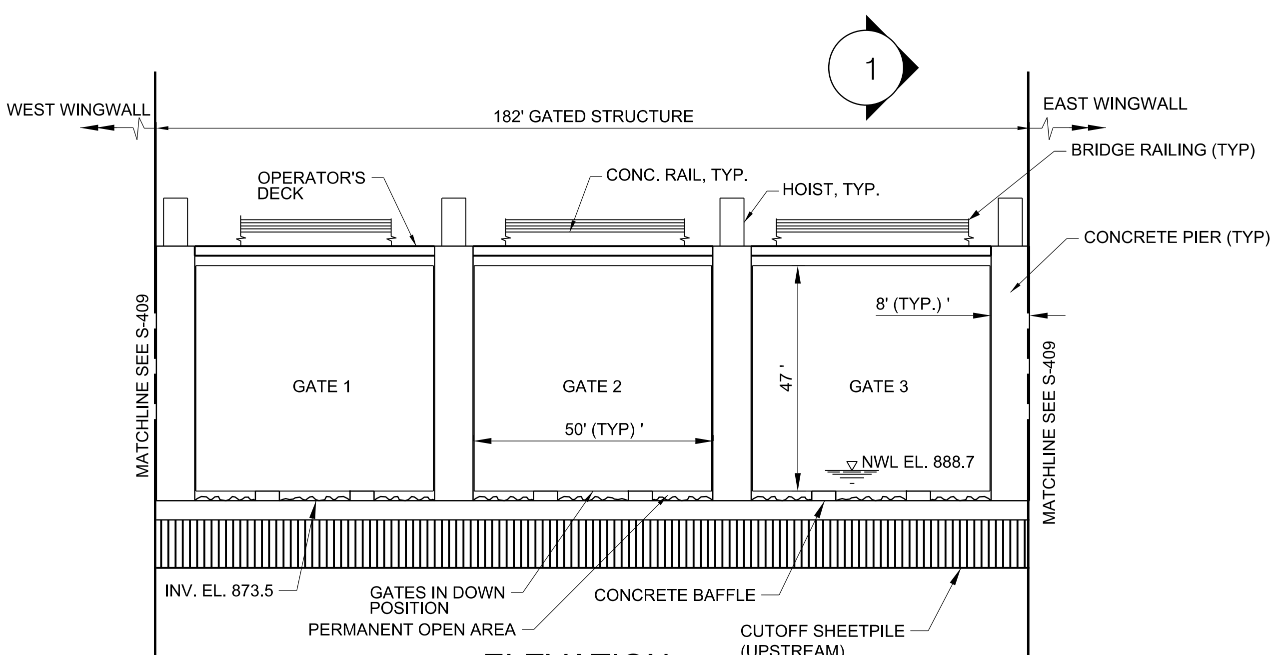
1 SECTION

NORTH DAKOTA LPP RRN CONTROL GATED STRUCTURE
SCALE: N.T.S.

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	59.8
UNDRAINED UNUSUAL	79.7
UNDRAINED EXTREME	104.0
DRAINED USUAL	31.8

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. FULL GATE DETAIL NOT SHOWN.



ELEVATION

NORTH DAKOTA LPP RRN CONTROL STRUCTURE
GATED SECTION

20 0 20 40
SCALE IN FEET

NOTE: PILES NOT SHOWN FOR CLARITY.

QUANTITIES (GATED STRUCTURE)

CONCRETE VOLUMES			STEEL	PILES			
FTG.	PIER	DECK	REINF.		NO.	LENGTH	TOTAL
CY	CY	CY	LBS	NO.	ROWS	FT	FT
2,273	3,759	128	467,450	318	12	35	11,037

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

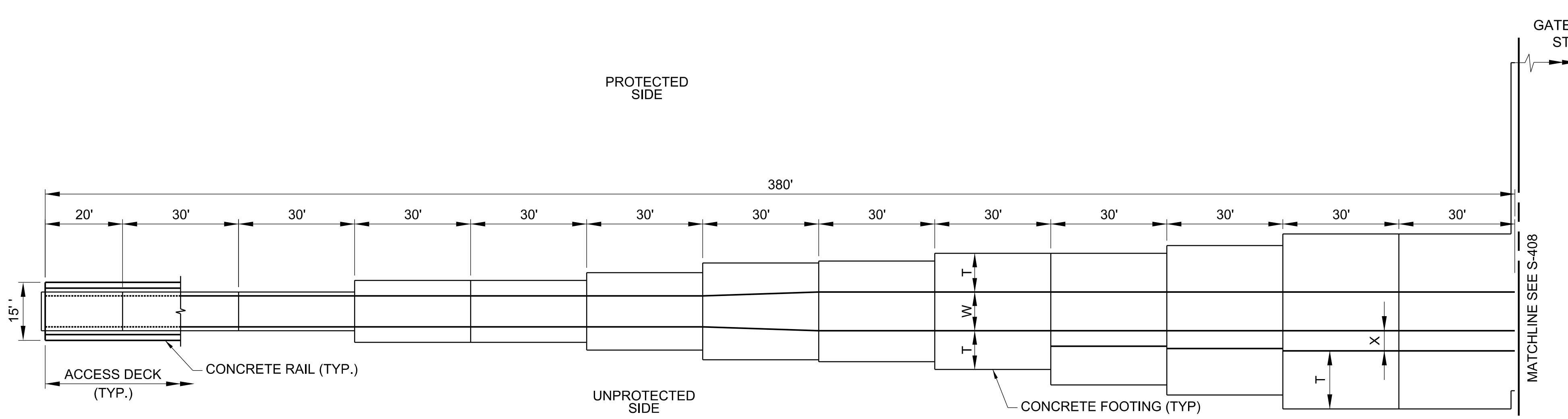
DESIGNED BY: MRM/MW1/PKN
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-408HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL- LPP CONTROL
STRUCTURE ON RED RIVER OF THE NORTH
GATES PLAN, SECTION AND ELEVATION

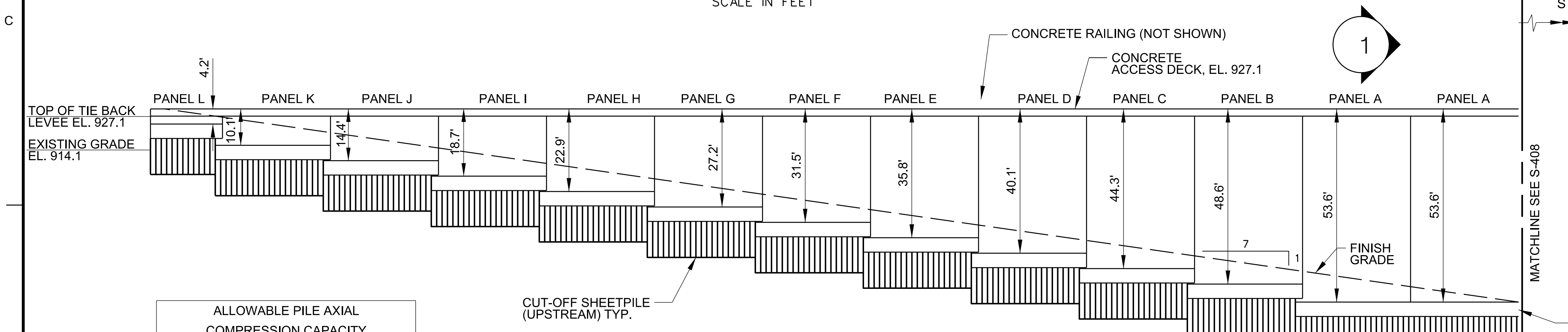
SHEET IDENTIFICATION
S-408



PLAN

WEST WINGWALL
(EAST WINGWALL SYMMETRICAL)

20 0 20 40
SCALE IN FEET



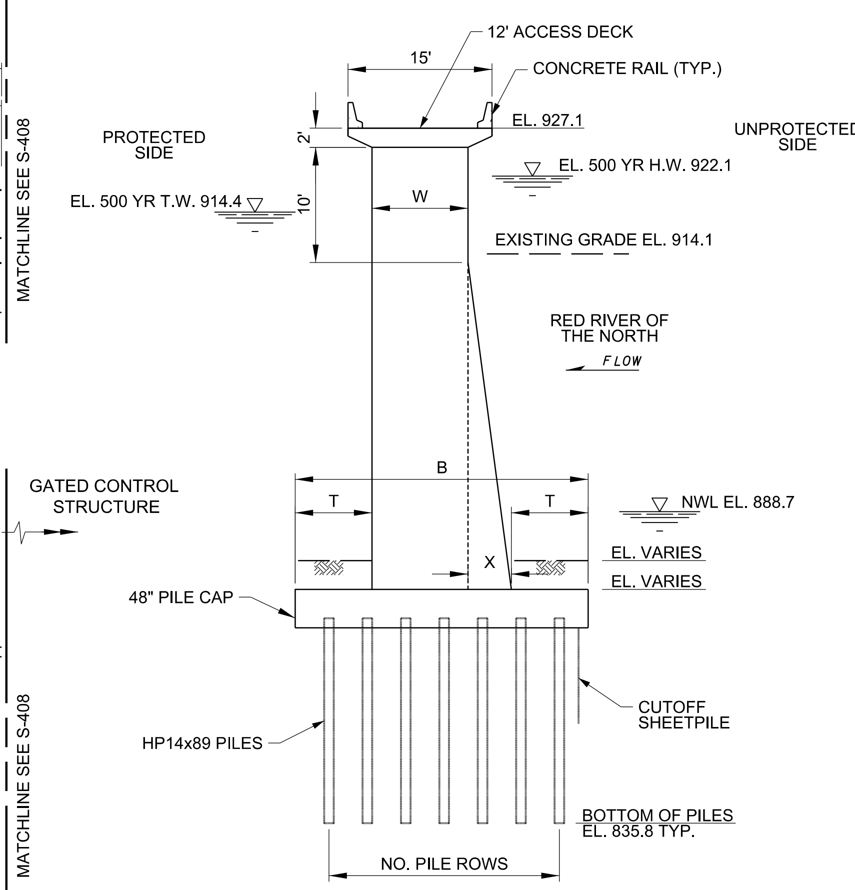
ELEVATION

WEST WINGWALL
(EAST WINGWALL SYMMETRICAL)

20 0 20 40
SCALE IN FEET

NOTE: PILES AND CONCRETE RAILS NOT SHOWN FOR CLARITY.

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	59.8
UNDRAINED UNUSUAL	79.7
UNDRAINED EXTREME	104.0
DRAINED USUAL	31.8



1 SECTION
RRN CONTROL STRUCTURE WINGWALL
SCALE: N.T.S.

NOTES:

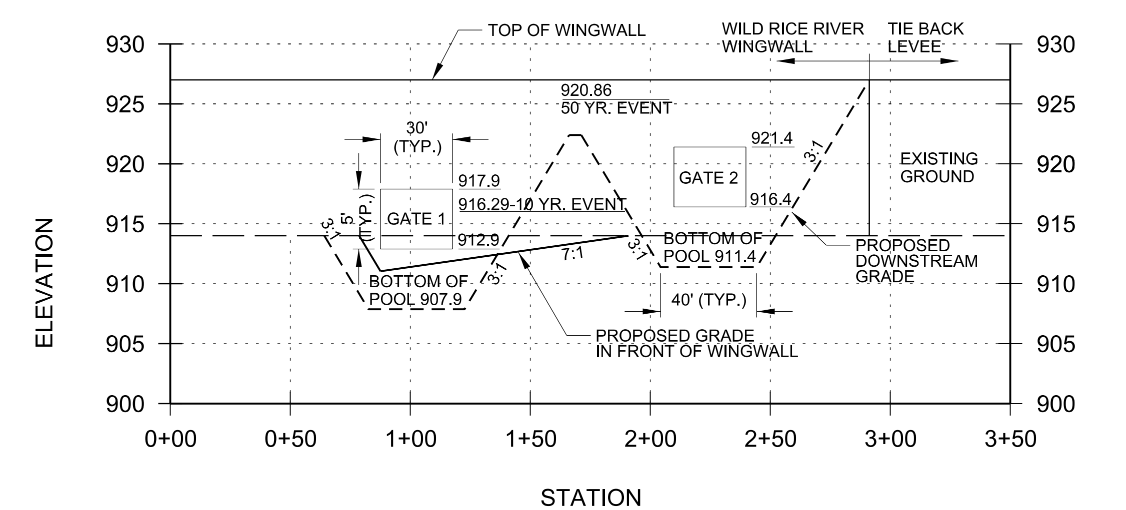
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

QUANTITIES (WINGWALLS)

PANEL	STEM W (FT)	STEM BATTER X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. CY	STEM CY	DECK CY	STEEL REINF. LBS	PILES NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	10	5.2	45.2	15.0	214	694	30	86,665	51	8	35	1,770
A	10	5.2	45.2	15.0	214	694	30	86,665	51	8	35	1,770
B	10	4.6	38.6	12.0	183	611	30	79,975	45	7	40	1,786
C	10	4.0	34.0	10.0	161	543	30	69,562	42	7	44	1,847
D	10	0	30.0	10.0	142	423	30	62,888	38	6	48	1,834
E	10	0	26.0	8.0	123	375	30	56,233	34	5	53	1,786
F	9	0	25.0	8.0	119	295	29	51,879	29	5	57	1,648
G	8	0	20.0	6.0	95	224	29	43,892	24	4	61	1,466
H	8	0	16.0	4.0	76	186	29	37,236	19	3	65	1,242
I	8	0	16.0	4.0	76	148	29	34,470	17	3	70	1,184
J	8	0	10.0	1.0	47	110	29	25,614	14	2	74	1,035
K	8	0	10.0	1.0	47	72	29	22,848	10	2	78	782
L	8	0	10.0	1.0	33	13	19	13,243	4	2	84	336
				WEST WINGWALL	1,531	4,387	373	668,168	378			18,486
				EAST WINGWALL	1,531	4,387	373	668,168	378			18,486
				TOTAL	3,062	8,774	746	1,336,337	756			36,972

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G



2 SECTION
S-410
NORTH DAKOTA LPP CONTROL FISH PASSAGE
SCALE: N.T.S.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

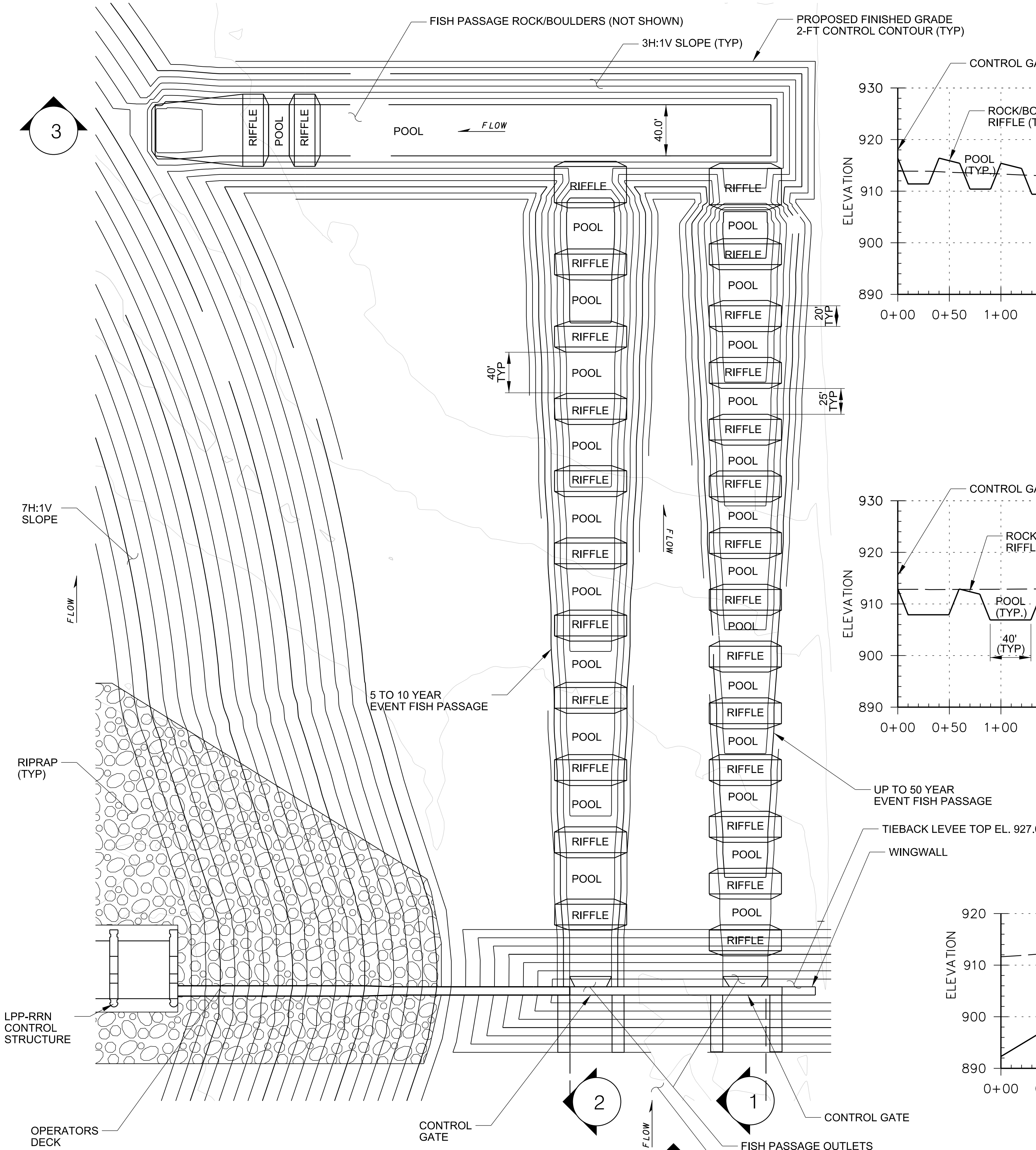
DESIGNED BY: MRM/MW1/PKN
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-409HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

NORTH DAKOTA DIVERSION (LPP)

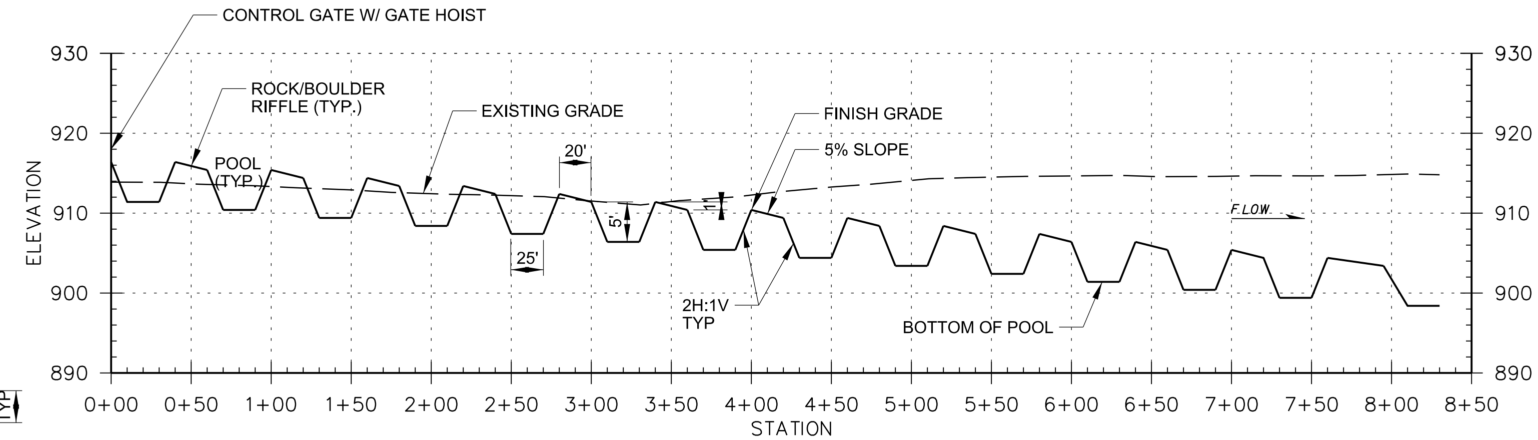
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL- LLP CONTROL
STRUCTURE ON RED RIVER OF THE NORTH
WINGWALL PLAN, ELEVATION AND SECTION

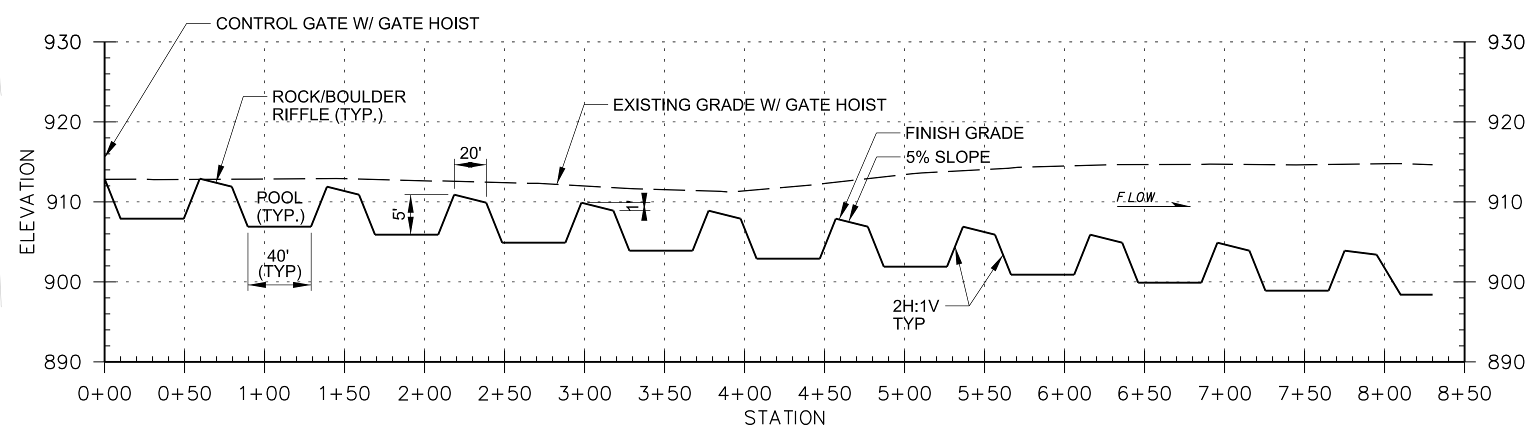
SHEET IDENTIFICATION
S-409



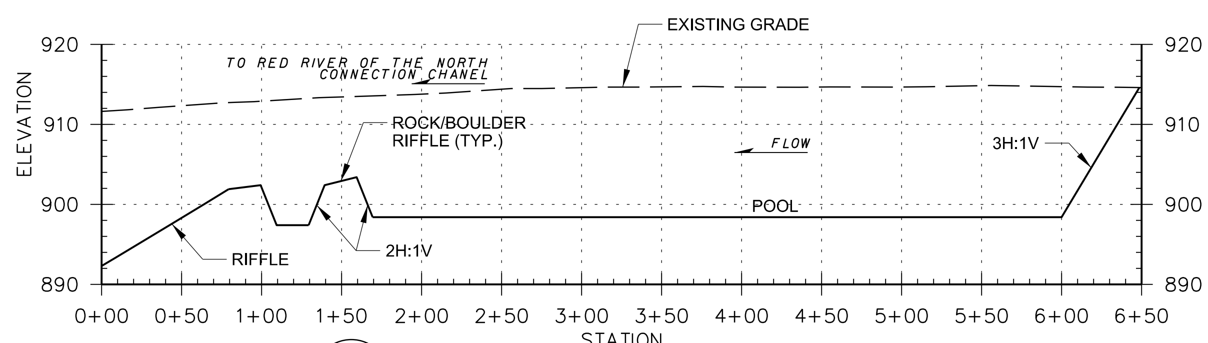
PLAN VIEW: FISH PASSAGE LPP
LPP RED RIVER CONTROL STRUCTURE FISH PASSAGE SYSTEM
SCALE IN FEET



1 PROFILE
PROPOSED FISH PASSAGE
UP TO 50 YR. FLOOD EVENTS
N.T.S.



2 PROFILE
PROPOSED FISH PASSAGE SYSTEM–CENTERLINE
5 TO 10 YR. FLOOD EVENTS
N.T.S.



3 PROFILE
FISH PASSAGE CONNECTION TO MAIN CHANNEL CENTERLINE
N.T.S.

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

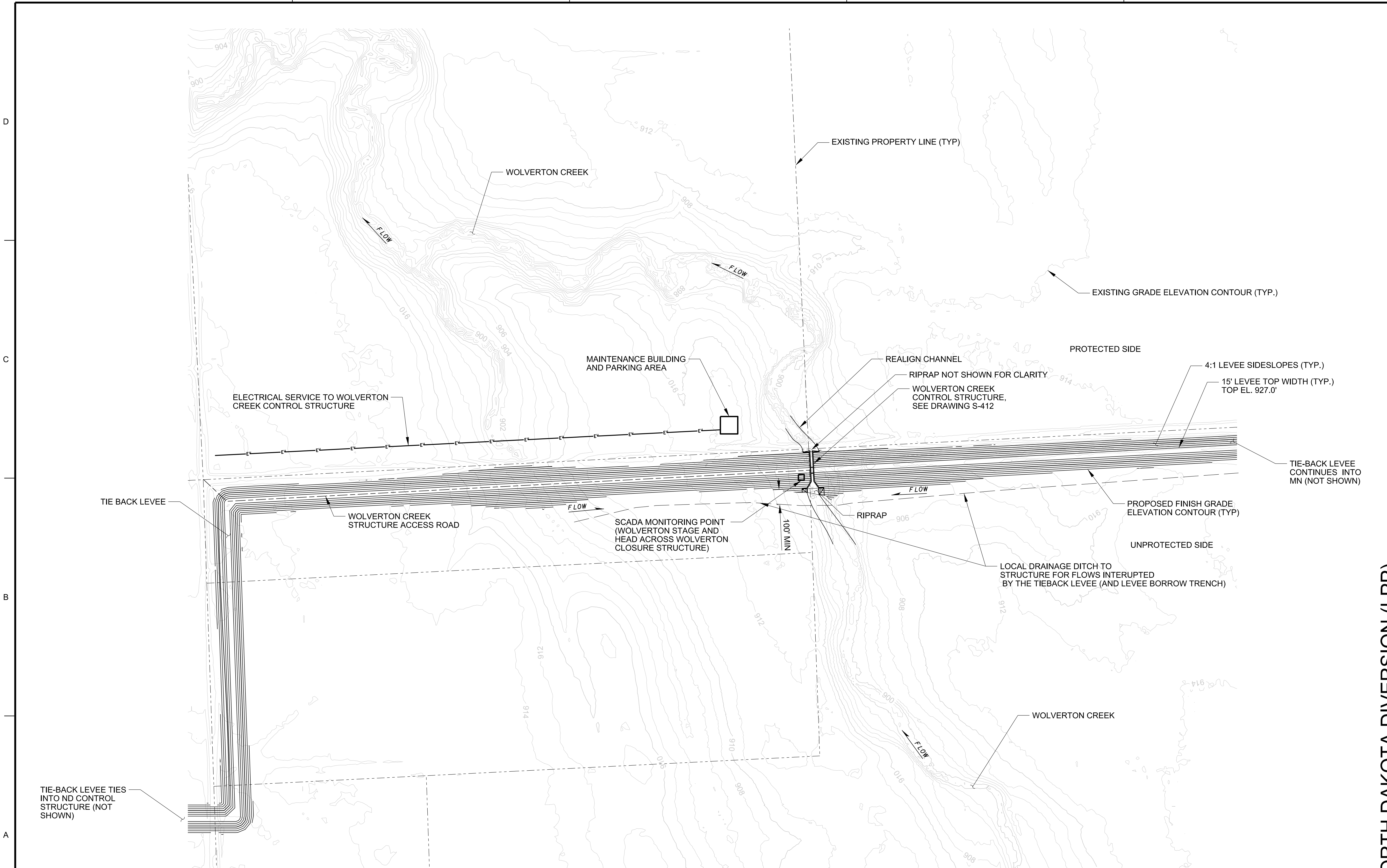
DESIGNED BY: MW/SGW/JMA
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-410HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LPP FISH PASSAGE
STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW AND SECTIONS

SHEET IDENTIFICATION
S-410



NOTES:

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PLAN VIEW

WOLVERTON CREEK CLOSER/DRAINAGE STRUCTURE

200 0 200 400
SCALE IN FEET

HORIZONTAL COORDINATE SYSTEM: STATE PLANE - ND SOUTH NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET

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MARK	DESCRIPTION	DATE	APPR
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NORTH DAKOTA DIVERSION (LPP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

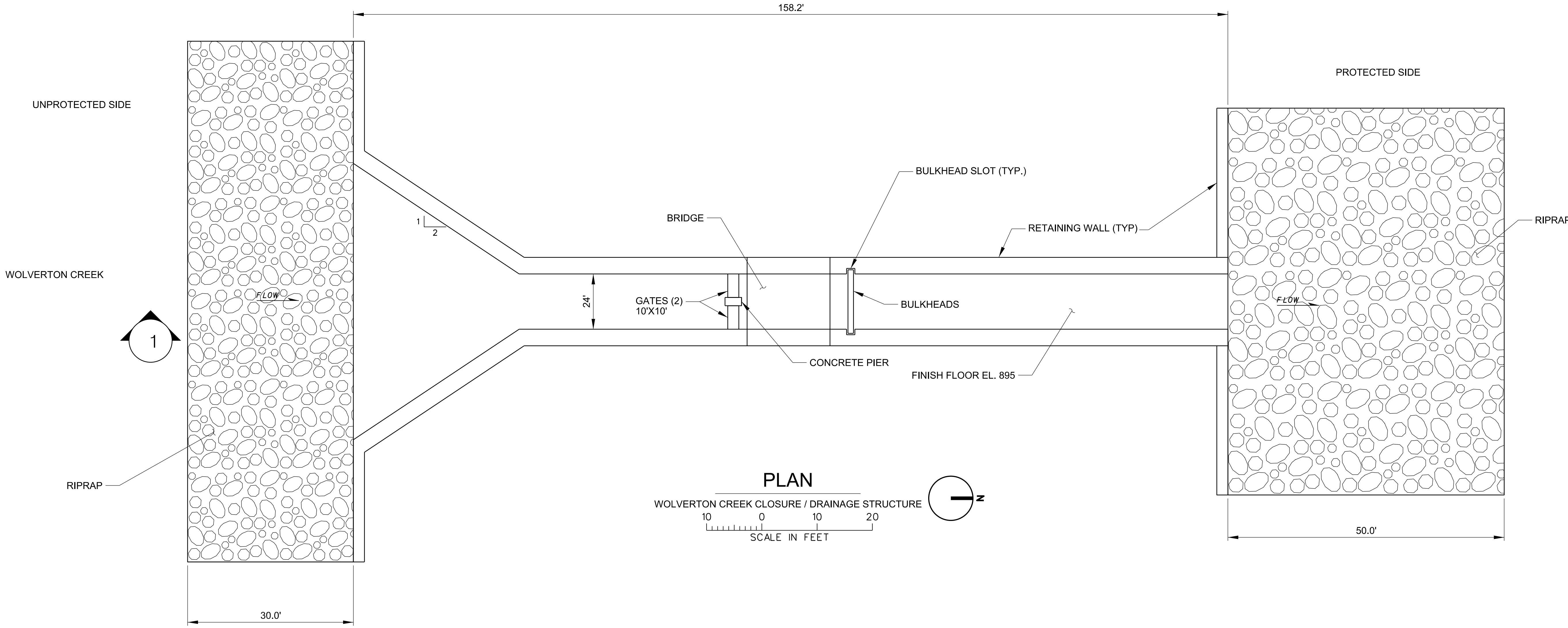
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MW1/JDA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-411HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL-WOLVERTON CREEK
CLOSURE / DRAINAGE STRUCTURE
PLAN VIEW

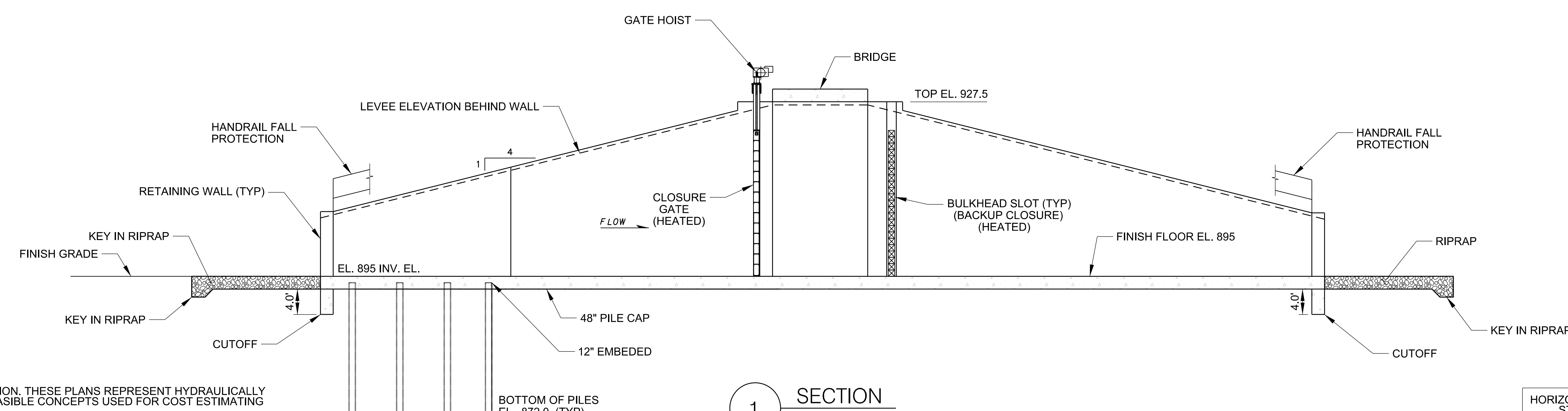
SHEET IDENTIFICATION
S-411



PLAN

WOLVERTON CREEK CLOSURE / DRAINAGE STRUCTURE

10 0 10 20
SCALE IN FEET



1 SECTION

SIDE RETAINING WALL
SCALE: N.T.S.

NOTES:

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HORIZONTAL COORDINATE SYSTEM:
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VERTICAL COORDINATE SYSTEM:
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MARK	DESCRIPTION	DATE	APPR
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ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

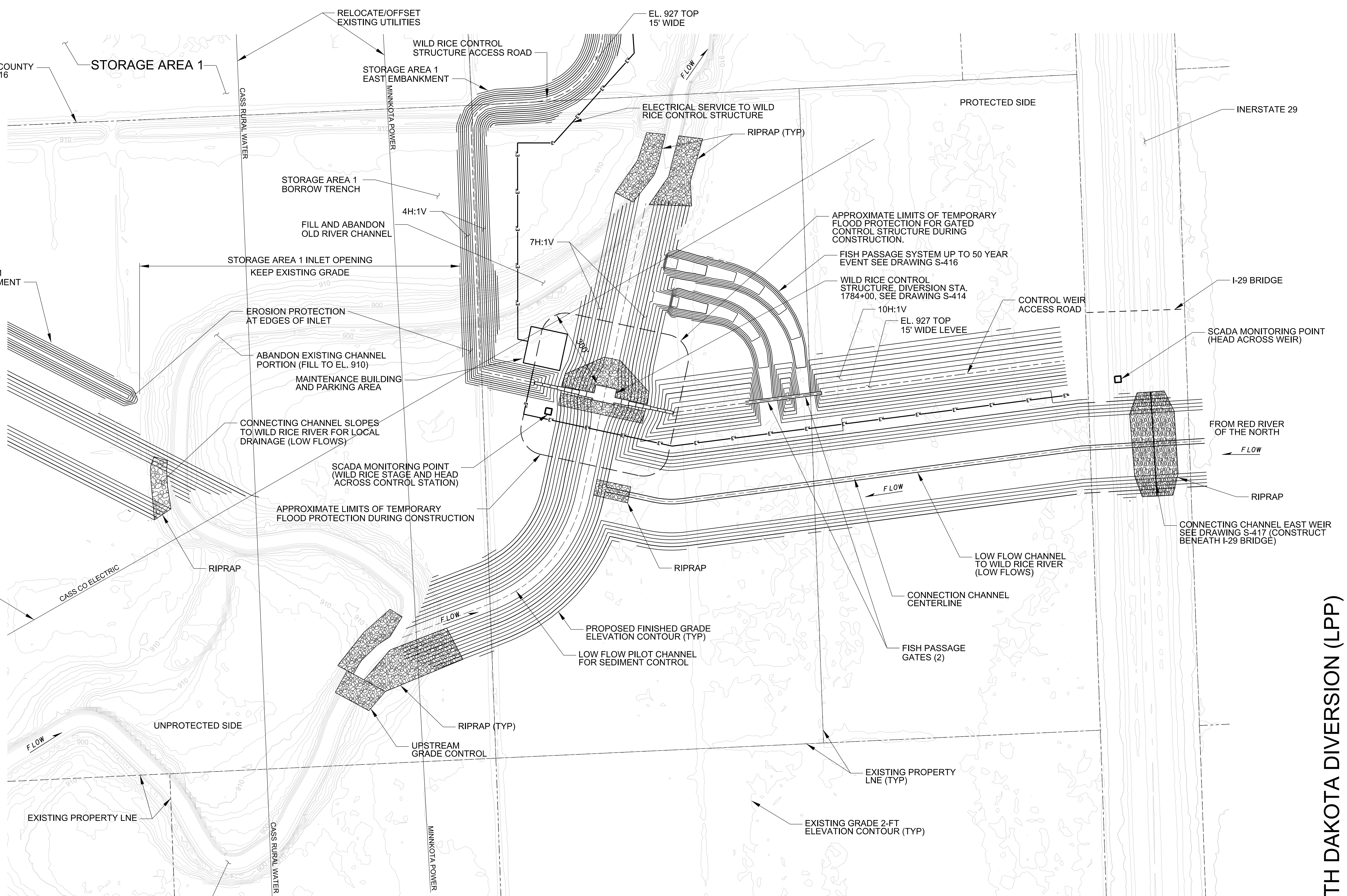
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: JDA/JJW2
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-412HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - WOLVERTON CREEK
CLOSURE / DRAINAGE STRUCTURE
PLAN AND SECTION

SHEET IDENTIFICATION
S-412



PLAN VIEW

WILD RICE CONTROL STRUCTURE AND DIVERSION CONTROL WEIR

200 0 200 400

SCALE IN FEET

N

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

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HORIZONTAL COORDINATE SYSTEM:
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VERTICAL COORDINATE SYSTEM:
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NORTH DAKOTA DIVERSION (LPP)

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ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

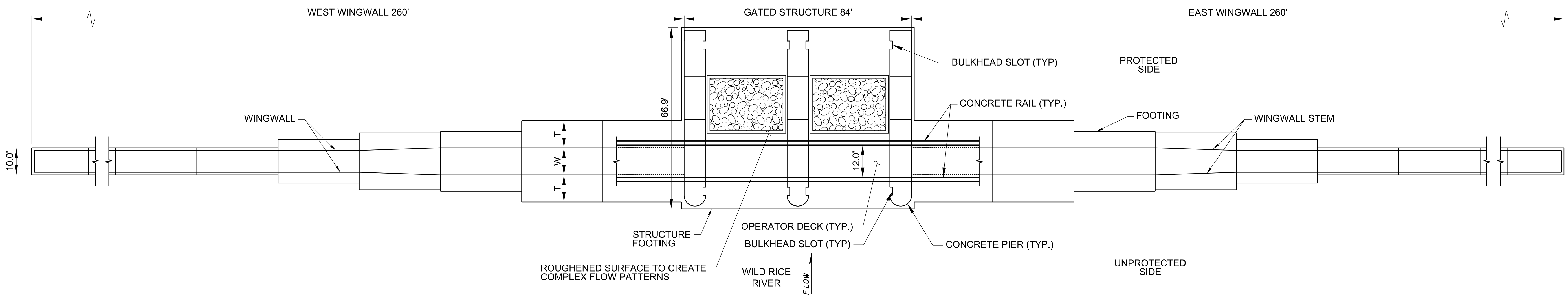
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MW1/PKN	DATE: 28FEB2011		
DWN BY: MJJ	CKD BY: PKN/BKL	SOLICITATION NO.: N/A	
SUBMITTED BY: N/A		CONTRACT NO.: N/A	
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011	FILE NUMBER: N/A	
SIZE: ANSI D	FILE NAME: FMM_S-413HYD.dgn		

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILTY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - STORAGE AREA 1 INLET/ CONTROL STRUCTURE ON WILD RICE RIVER
PLAN VIEW

SHEET IDENTIFICATION
S-413



PLAN

WILD RICE RIVER CONTROL STRUCTURE AND WINGWALL

20 0 20 40

SCALE IN FEET

NOTES:

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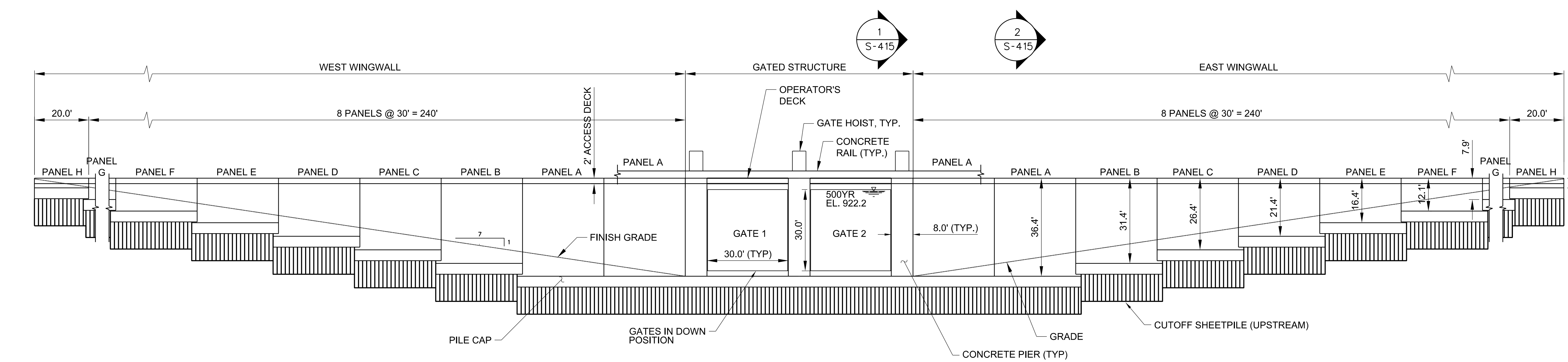

ELEVATION

WILD RICE RIVER CONTROL STRUCTURE AND WINGWALL

20 0 20 40

SCALE IN FEET

NOTE: PILES AND CONCRETE RAILS NOT SHOWN FOR CLARITY.

HORIZONTAL COORDINATE SYSTEM: STATE PLANE - ND SOUTH NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET

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ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MWY/PKN | DATE: 28FEB2011
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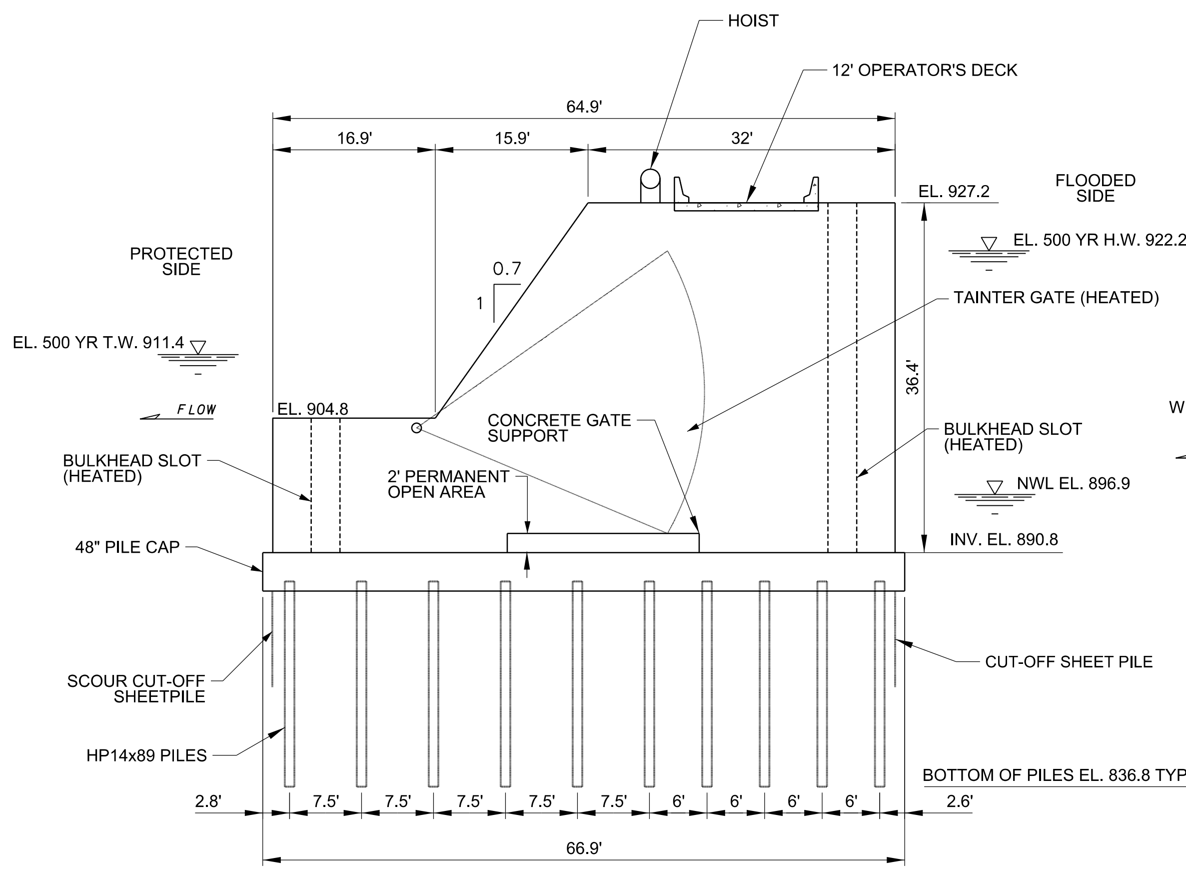
SUBMITTED BY: N/A | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-414HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

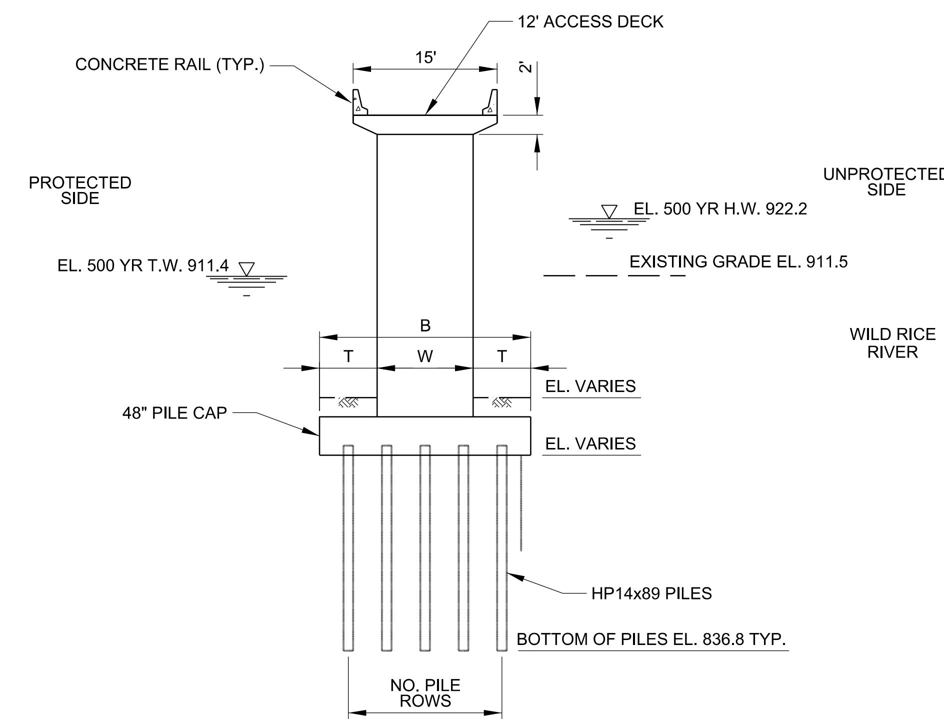
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - CONTROL STRUCTURE ON WILD RICE RIVER
PLAN VIEW AND ELEVATION

SHEET IDENTIFICATION
S-414



1 SECTION
S-414 WILD RICE RIVER CONTROL STRUCTURE
SCALE: N.T.S.



2 SECTION
S-414 WILD RICE RIVER CONTROL STRUCTURE
SCALE: N.T.S.

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	74.1
UNDRAINED UNUSUAL	98.7
UNDRAINED EXTREME	128.8
DRAINED USUAL	43.7

NOTES:

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3. FULL GATE DETAIL NOT SHOWN.

QUANTITIES (GATED STRUCTURE)

CONCRETE VOLUMES			STEEL	PILES			
FTG.	PIER	DECK	REINF.		NO.	LENGTH	TOTAL
CY	CY	CY	LBS	NO.	ROWS	FT	FT
832	1,604	59	192,447	100	10	51	5,100

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

QUANTITIES (WINGWALLS)

				CONCRETE VOLUMES			STEEL	PILES			
	STEM	FTG. WIDTH	FTG. TOE	FTG.	STEM	DECK	REINF.		NO.	LENGTH	TOTAL
PANEL	W (FT)	B (FT)	T (FT)	CY	CY	CY	LBS	NO.	ROWS	FT	FT
A	10	30.0	10.0	142	382	30	56,928	30	5	51	1,530
A	10	30.0	10.0	142	382	30	56,928	30	5	51	1,530
B	10	22.0	6.0	104	327	30	46,343	24	4	56	1,344
C	9	21.0	6.0	100	244	29	42,206	20	4	61	1,220
D	8	16.0	4.0	76	173	29	34,180	15	3	66	990
E	8	10.0	1.0	47	128	29	25,541	10	2	71	710
F	8	10.0	1.0	47	90	29	23,208	8	2	75	602
G	8	10.0	1.0	47	52	29	20,878	6	2	80	477
H	8	10.0	1.0	33	9	19	12,999	4	2	84	335
		WEST WINGWALL		739	1,788	254	319,211	147			8,738
		EAST WINGWALL		739	1,788	254	319,211	147			8,738
		TOTAL		1,478	3,576	508	638,422	294			17,476

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
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MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

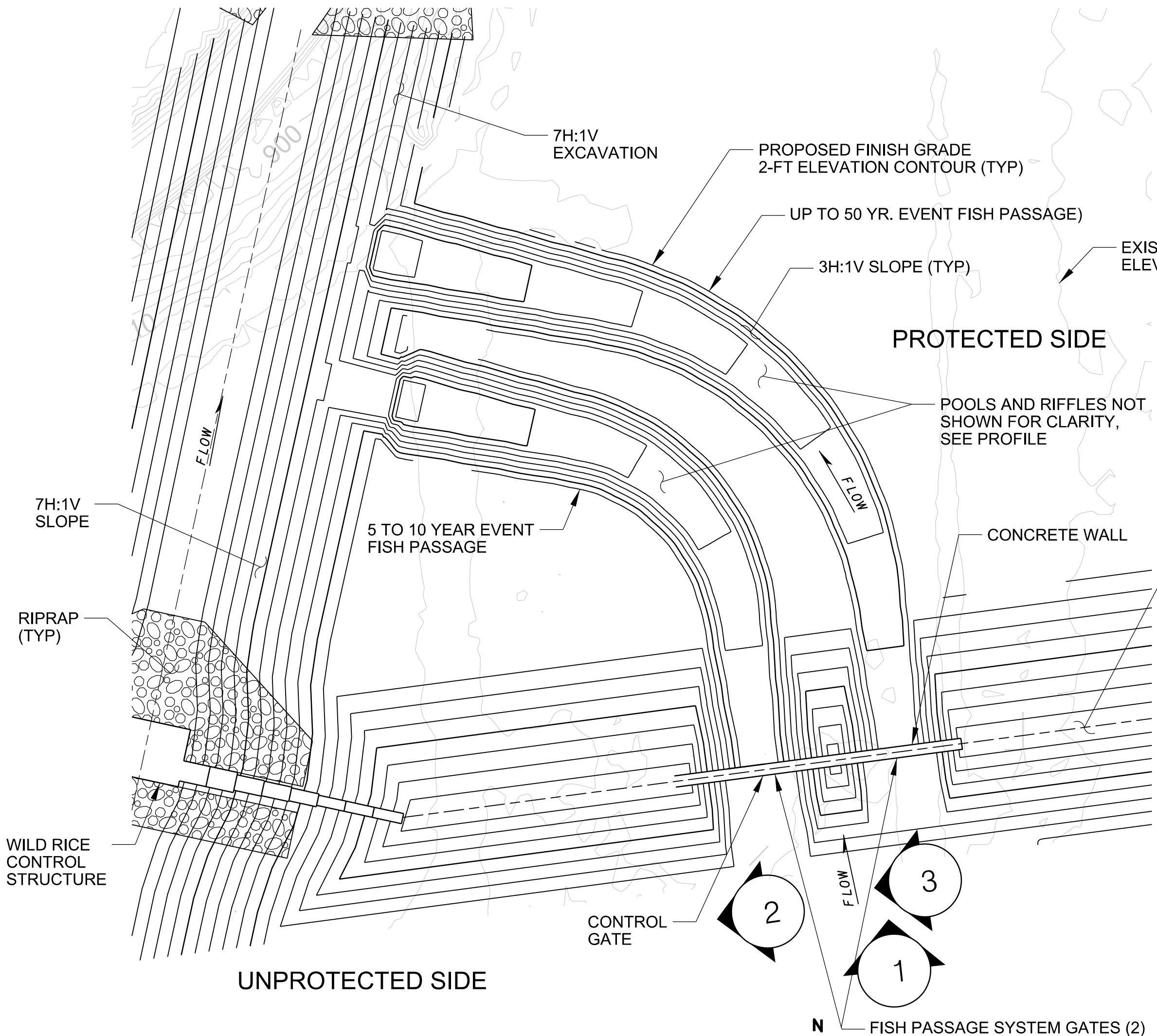
DESIGNED BY: MRM/MMY/PKN | DATE: 28FEB2011 | SOLICITATION NO.: N/A
DWN BY: MJJ | CKD BY: PKN/BKL | CONTRACT NO.: N/A
SUBMITTED BY: N/A | FILE NUMBER: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011
SIZE: ANSI D | FILE NAME: FMM_S-415HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - CONTROL STRUCTURE ON WILD RICE RIVER SECTIONS

SHEET IDENTIFICATION
S-415



PLAN VIEW

WILD RICE RIVER LPP FISH PASSAGE STRUCTURE

SCALE IN FEET: 100 0 100 200

NOTES:

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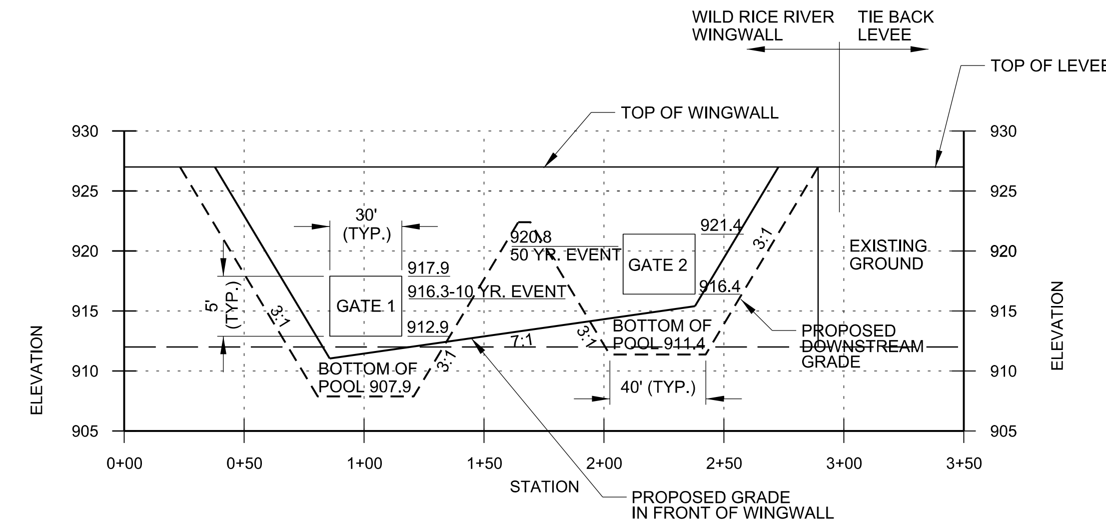
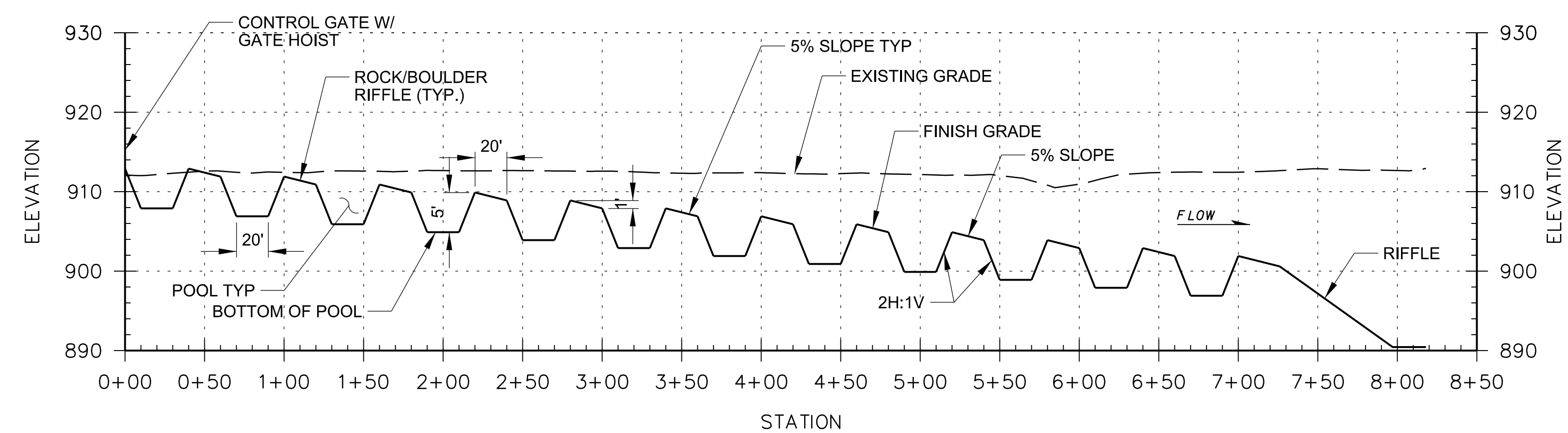


1 SECTION

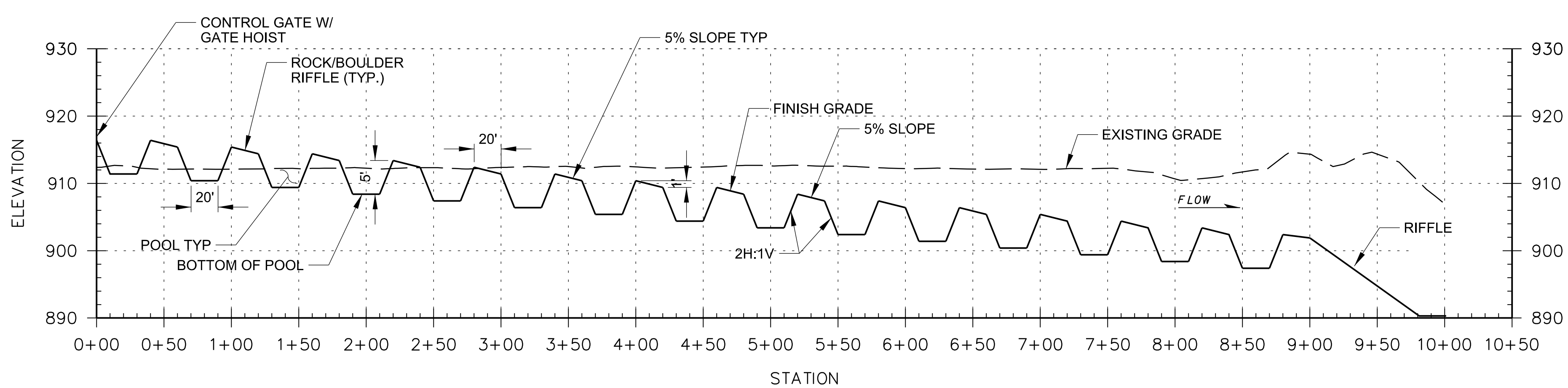
WILD RICE RIVER LLP FISH PASSAGE SYSTEM

SCALE: NTS



2 PROFILE

WILD RICE RIVER LPP FISH PASSAGE SYSTEM
5 TO 10 YEAR FLOOD EVENTS – CENTERLINE



3 PROFILE

WILD RICE RIVER LPP FISH PASSAGE SYSTEM
UP TO 50 YEAR EVENTS – CENTERLINE

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
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DWN BY: MJJ | CKD BY: BKL | SOLICITATION NO.: N/A
SUBMITTED BY: N/A | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-416HYD.dgn

U.S. ARMY CORPS OF ENGINEERS
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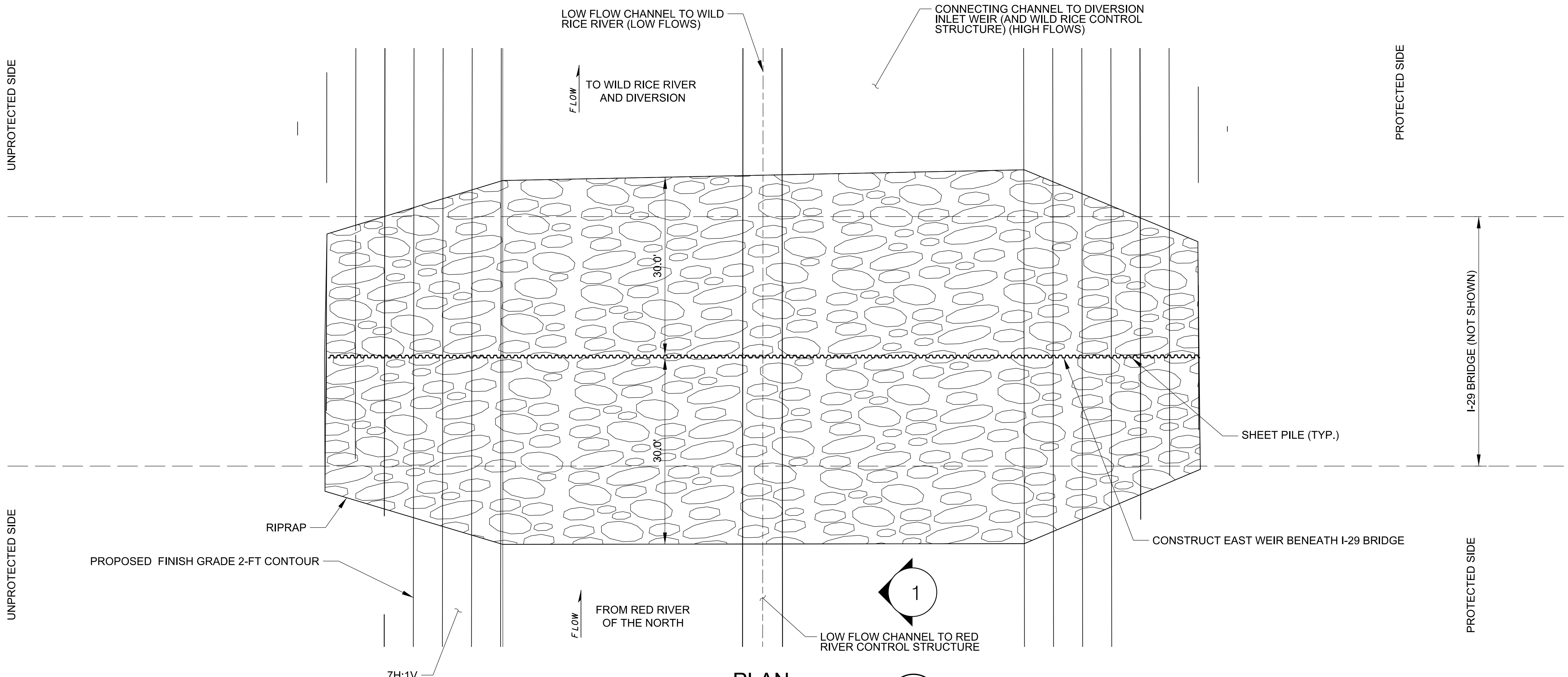
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

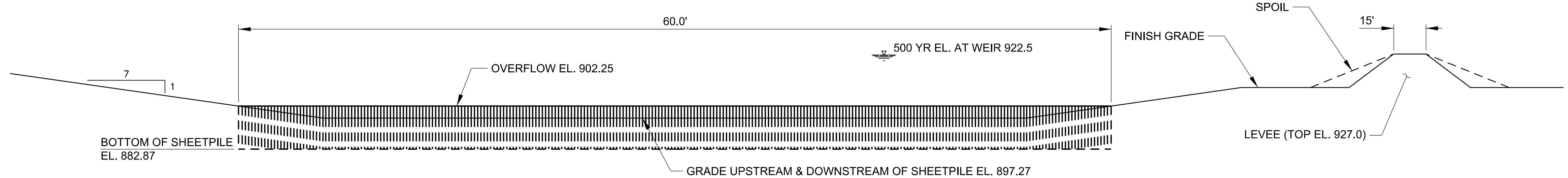
FLOOD CONTROL - LPP FISH PASSAGE
STRUCTURE ON WILD RICE RIVER
PLAN VIEW AND SECTIONS

SHEET IDENTIFICATION
S-416



PLAN

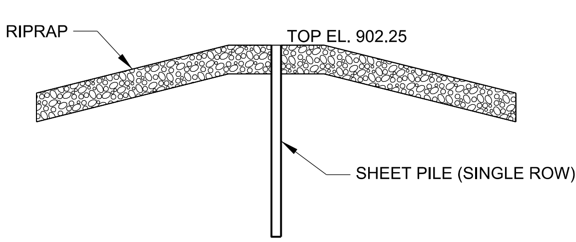
CONTROL WEIR SEPARATING
RED RIVER FROM WILD RICE RIVER
SCALE: N.T.S.



NOTE: I-29 BRIDGE NOT SHOWN

ELEVATION

CONTROL WEIR SEPARATING
RED RIVER FROM WILD RICE RIVER
SCALE: N.T.S.



1 SECTION

CONTROL WEIR SEPERATING RED RIVER FROM WILD RICE RIVER
SCALE: N.T.S.

NOTES:

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HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWT/PKN/BKL	DWN BY: MJJ	CKD BY: PKN/BKL	DATE: 28FEB2011
SUBMITTED BY: N/A			SOLICITATION NO.: N/A
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011		CONTRACT NO.: N/A
SIZE: ANSI D	FILE NAME: FMM_S-417HYD.dgn		FILE NUMBER: N/A

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ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - WEIR STRUCTURE FROM CONNECTING CHANNEL TO WILD RICE RIVER - PLAN VIEW AND ELEVATION

SHEET IDENTIFICATION
S-417

PLAN VIEW

STORAGE AREA 1

1000 0 1000 2000

SCALE IN FEET

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

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HORIZONTAL COORDINATE SYSTEM: STATE PLANE - ND SOUTH NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET

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MARK	DESCRIPTION	DATE	APPR
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DESIGNED BY: MW1/JJW2
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CKD BY: BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-418H10.dgn

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

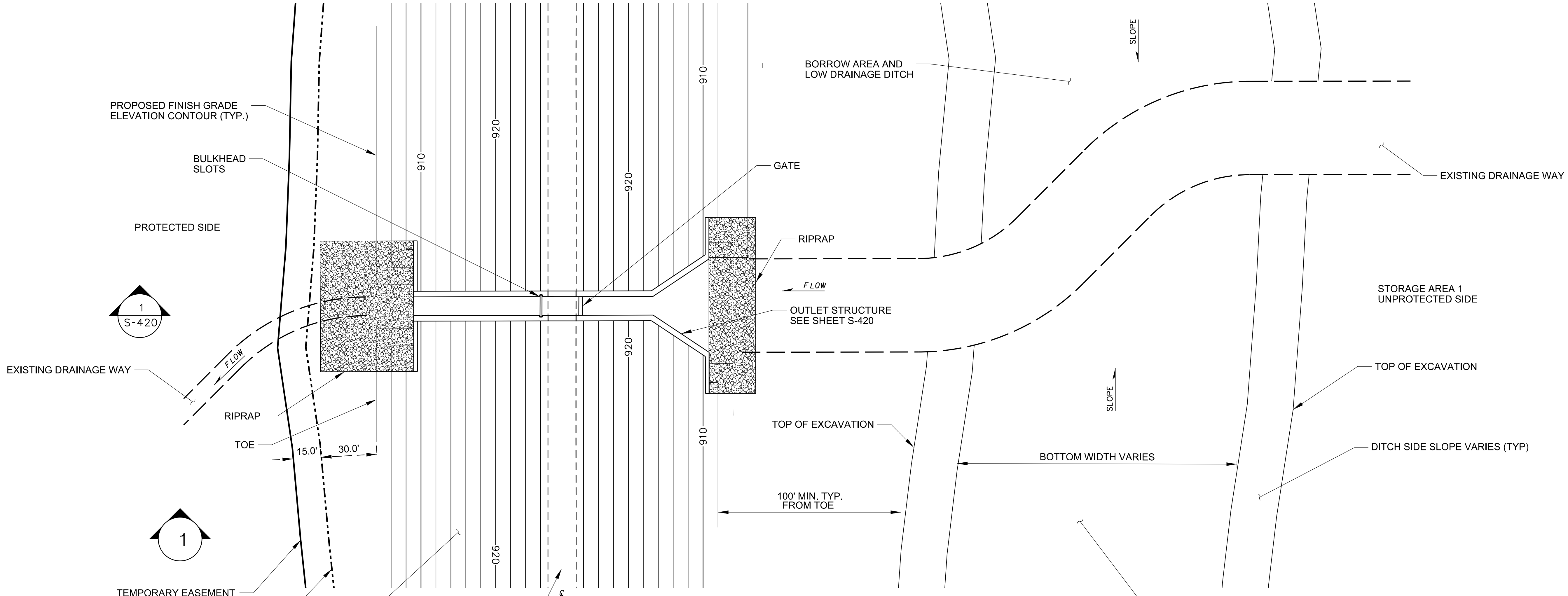
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
STORAGE AREA 1
PLAN VIEW

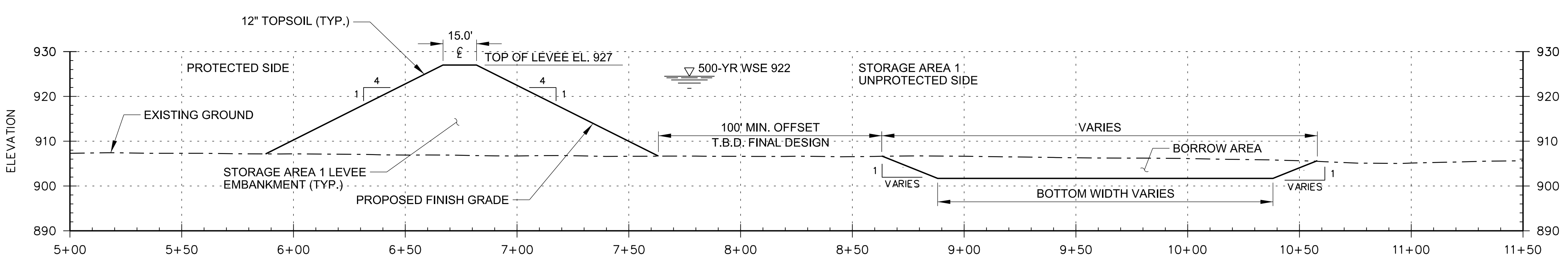
SHEET IDENTIFICATION
S-418



PLAN

STORAGE AREA 1 CLOSURE/DRAINAGE STRUCTURE

30 0 30 60
SCALE IN FEET



1 SECTION

NOTES:

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2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
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MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

NORTH DAKOTA DIVERSION (LPP)

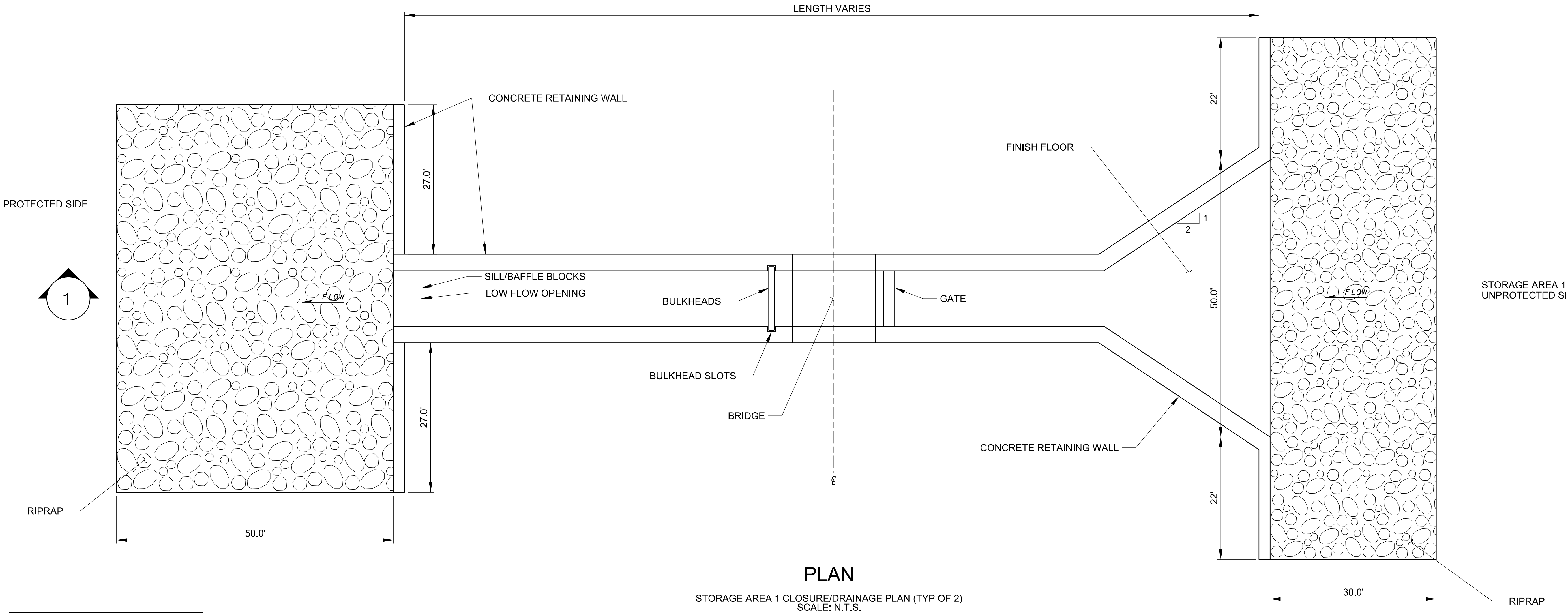
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
STORAGE AREA 1
PLAN VIEW AND EMBANKMENT SECTION

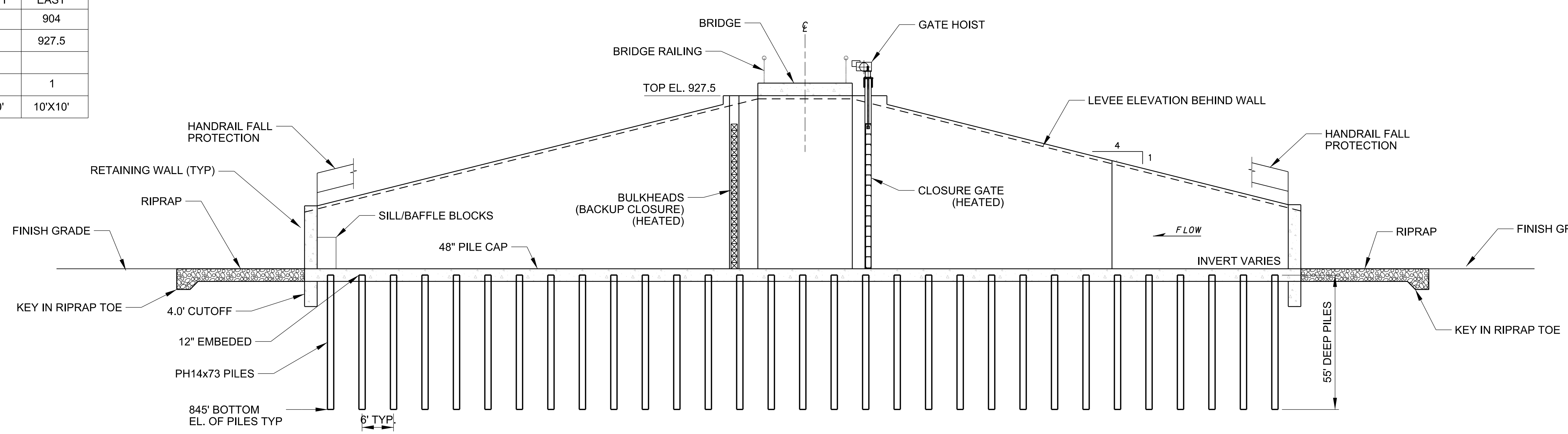
SHEET IDENTIFICATION
S-419



PLAN

STORAGE AREA 1 CLOSURE/DRAINAGE PLAN (TYP OF 2)
SCALE: N.T.S.

STORAGE AREA 1 CLOSURE/DRAINAGE STRUCTURES		
	NORTH	EAST
INV. EL	900	904
TOP EL.	927.5	927.5
L (FT)		
NO GATES	1	1
GATE SIZE	10'X10'	10'X10'



1
S-419

SECTION

STORAGE AREA 1 CLOSURE/DRAINAGE STRUCTURE (TYP. OF 2)
SCALE: N.T.S.

NOTES:

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DESIGNED BY: SDA/JJV/2/TSH
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-420HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

U.S. ARMY CORPS OF ENGINEERS
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ST. PAUL, MINNESOTA

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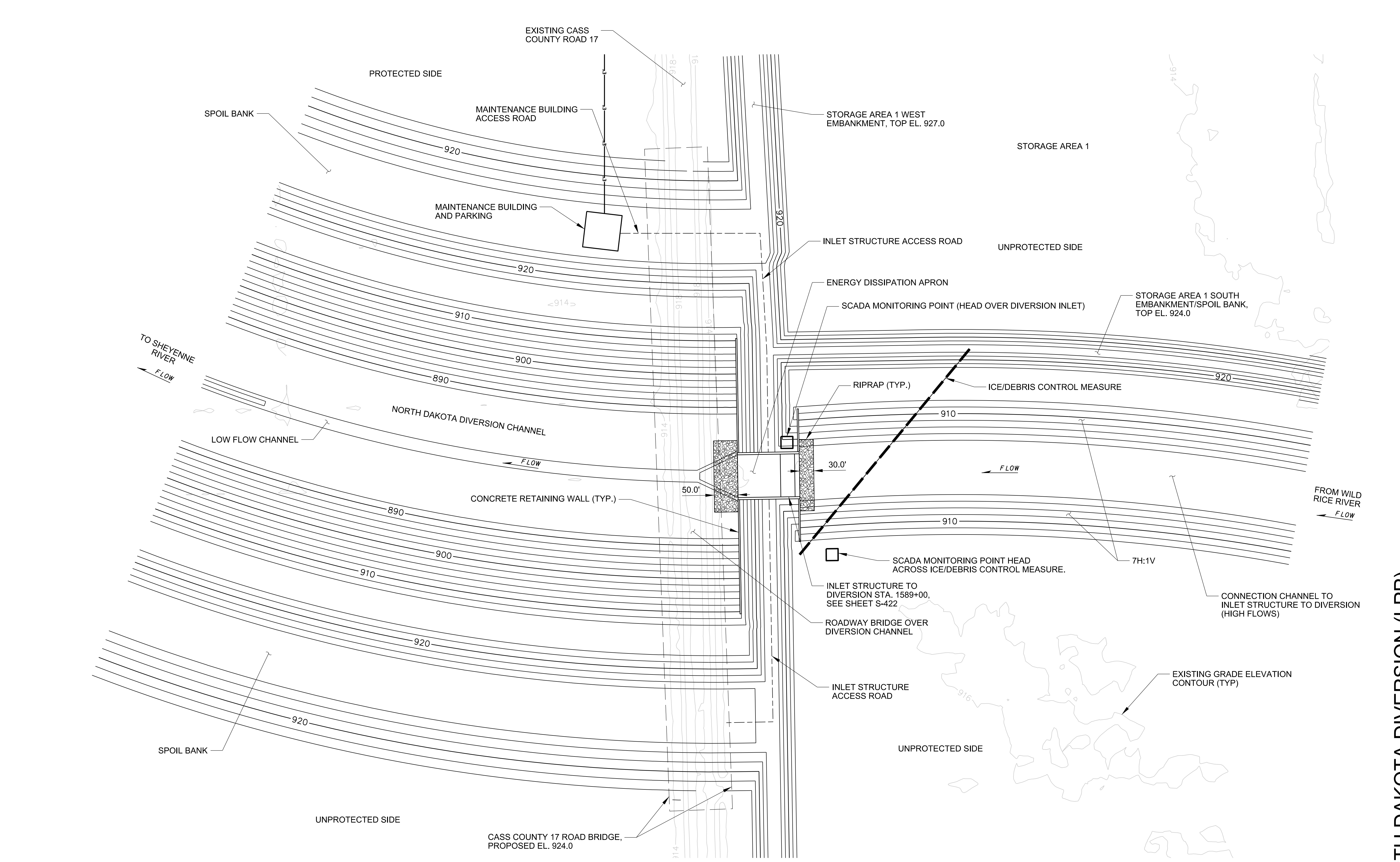
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - STORAGE AREA 1
TYPICAL CLOSURE DRAINAGE STRUCTURE
PLAN VIEW AND SECTION

NORTH DAKOTA DIVERSION (LLP)

SHEET IDENTIFICATION
S-420



NOTES:

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3. BRIDGE AND WEIR SHOWN AS INDEPENDENT STRUCTURES. CONSIDER A SINGLE INTEGRATED BRIDGE AND WEIR STRUCTRE DURING FINAL DESIGN.

PLAN VIEW

DIVERSION INLET WEIR STRUCTURE

100 0 100 200
SCALE IN FEET

N

HORIZONTAL COORDINATE SYSTEM:
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NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
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NORTH DAKOTA DIVERSION (LPP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

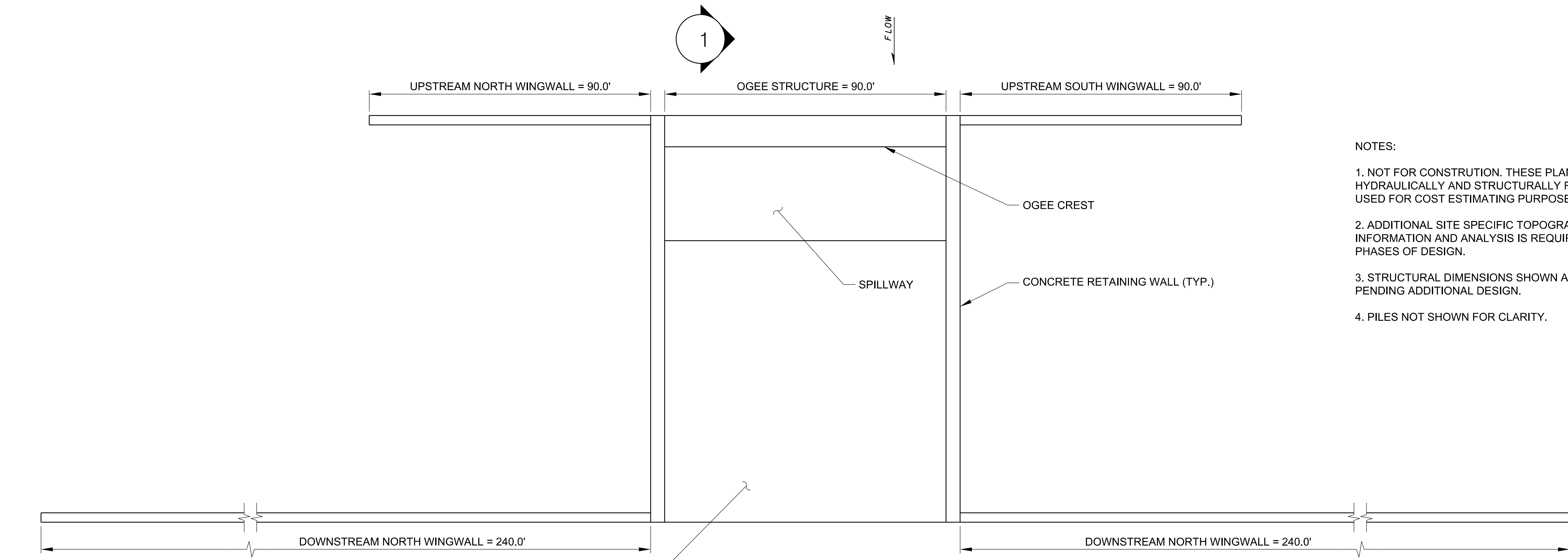
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
INLET STRUCTURE TO DIVERSION
PLAN VIEW

SHEET IDENTIFICATION
S-421



PLAN
INLET STRUCTURE
SCALE: N.T.S.

NOTES:

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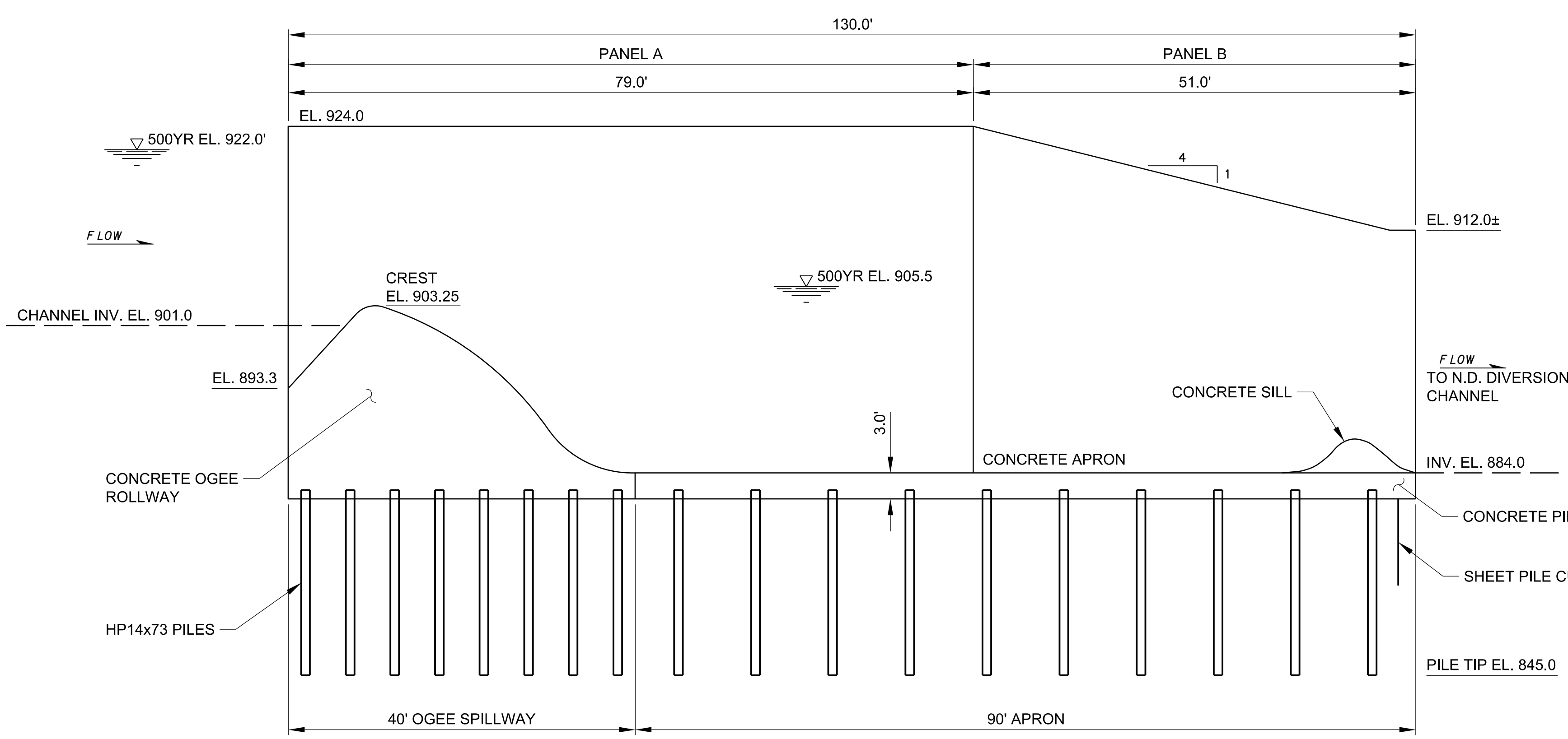
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. PILES NOT SHOWN FOR CLARITY.



1 SECTION
DIVERSION INLET STRUCTURE
SCALE: N.T.S.

QUANTITIES

PANEL	CONCRETE CY	STEEL REINF. LBS	PILES NO.	PILES NO. ROWS	PILES LENGTH FT	PILES TOTAL FT
OGEE SPILLWAY & APRON						
SPILLWAY	1,691	92,806	148	8	38	5,624
APRON	900	142,637	95	10	38	3,610
OGEE WINGWALLS (NORTH & SOUTH)						
A	1,054	88,288				
B	578	59,416				
TOTAL	4,223	383,147	243			9,234

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

HORIZONTAL COORDINATE SYSTEM:
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U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

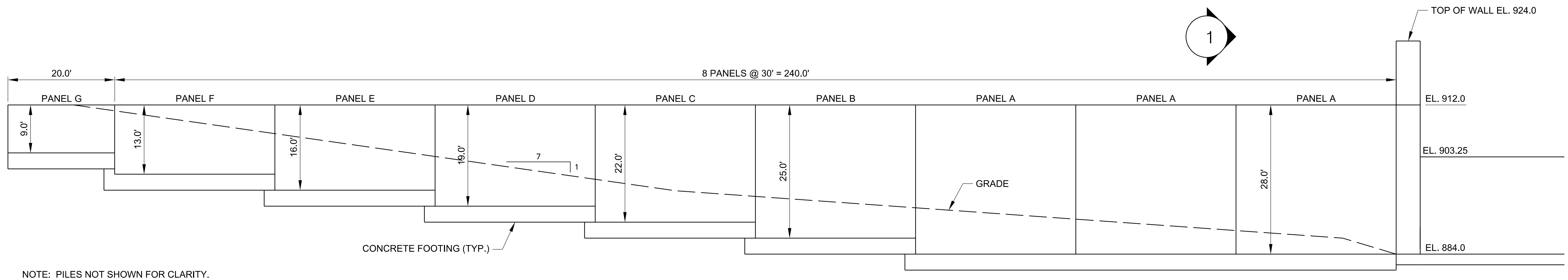
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MW1/PKN/JDA
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: S-422HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
LPP INLET STRUCTURE TO DIVERSION
PLAN VIEW AND ELEVATIONS

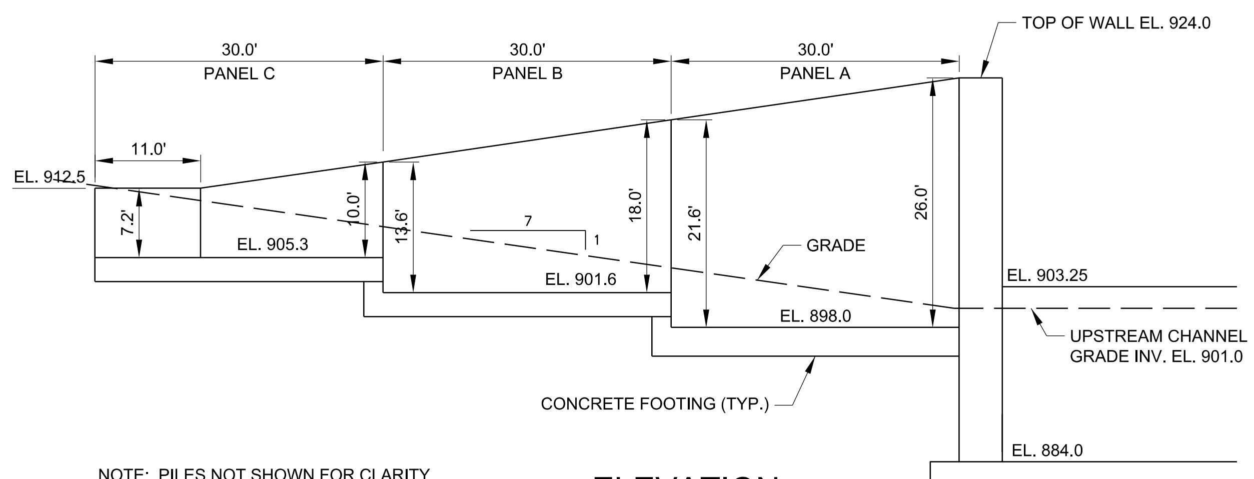
SHEET IDENTIFICATION
S-422



NOTE: PILES NOT SHOWN FOR CLARITY.

ELEVATION

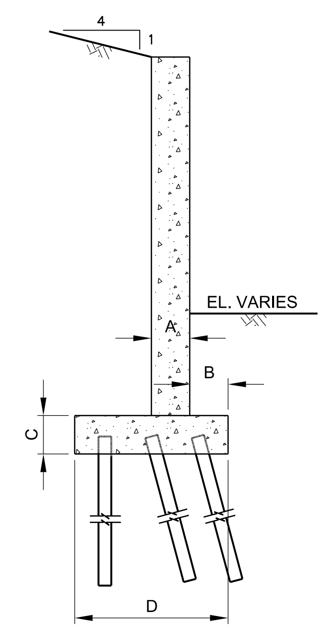
DOWNSTREAM NORTH WINGWALL
(SOUTH WINGWALL SYMMETRICAL)
N.T.S.



NOTE: PILES NOT SHOWN FOR CLARITY.

ELEVATION

UPSTREAM NORTH WINGWALL
(SOUTH WINGWALL SYMMETRICAL)
N.T.S.



1 SECTION
CONCRETE WINGWALL
SCALE: N.T.S.

QUANTITIES (WINGWALLS)

PANEL	STEM A (FT)	FTG. TOE B (FT)	FTG. THICKNESS C (FT)	FTG. WIDTH D (FT)	CONCRETE VOLUMES FTG. CY	CONCRETE VOLUMES STEM CY	STEEL REINF. LBS	PILES NO.	PILES NO. ROWS	PILES LENGTH FT	PILES TOTAL FT
DOWNSTREAM											
A	2.8	6.2	3.3	15.5	56	86	14,180	17	3	37	629
A	2.8	6.2	3.3	15.5	56	86	14,180	17	3	37	629
A	2.8	6.2	3.3	15.5	56	86	14,180	17	3	37	629
B	2.6	5.5	2.8	13.5	41	73	11,680	14	3	41	574
C	2.5	4.8	2.5	12.0	33	61	9,761	12	3	44	528
D	2.4	4.0	2.5	10.5	29	50	7,169	9	3	47	423
E	2.8	3.3	2.5	9.0	25	49	5,407	8	2	50	400
F	2.1	2.7	2.5	7.7	21	31	3,908	6	2	53	318
G	2.0	2.0	2.3	6.0	10	13	1,839	4	2	57	228
UPSTREAM											
A	2.7	5.8	3.0	14.0	47	71	11,443	16	3	52	832
B	2.3	3.8	2.5	10.0	28	41	6,339	9	3	56	504
C	2.0	2.3	2.5	6.7	19	18	3,047	6	2	59	354
				NORTH WINGWALL	421	663	103,132	135			6,048
				SOUTH WINGWALL	421	663	103,132	135			6,048
				TOTAL	842	1,326	206,264	270			12,096

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

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4. PILES NOT SHOWN FOR CLARITY.

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	56.3
UNDRAINED UNUSUAL	75.0
UNDRAINED EXTREME	97.9
DRAINED USUAL	30.2

HORIZONTAL COORDINATE SYSTEM:
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BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MWT/PKN/JDA
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: S-423HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

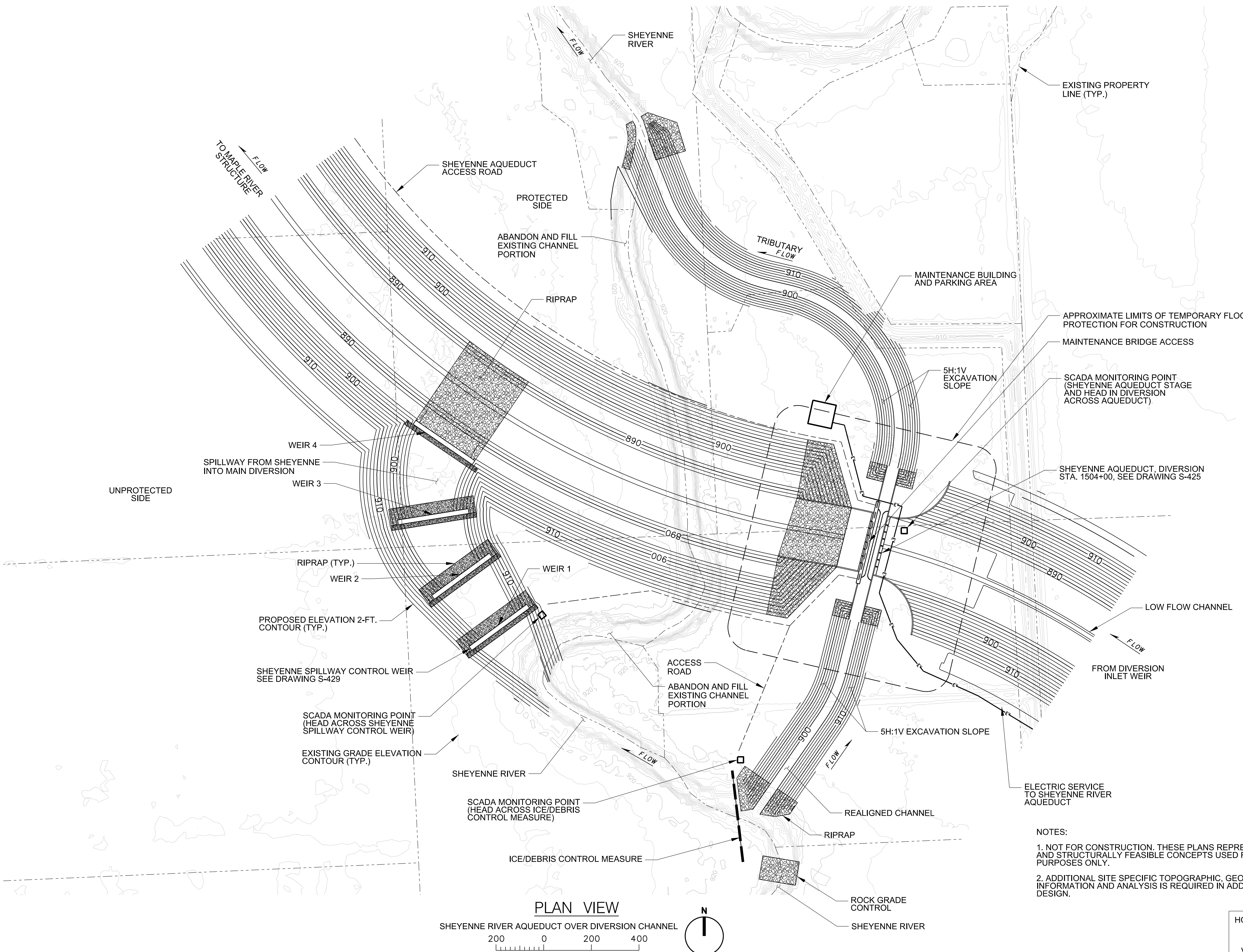
NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
LPP INLET STRUCTURE TO DIVERSION
PLAN VIEW AND ELEVATIONS

SHEET IDENTIFICATION
S-423

auto_time



PLAN VIEW

SHEYENNE RIVER AQUEDUCT OVER DIVERSION CHANNEL

NOTES:

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4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

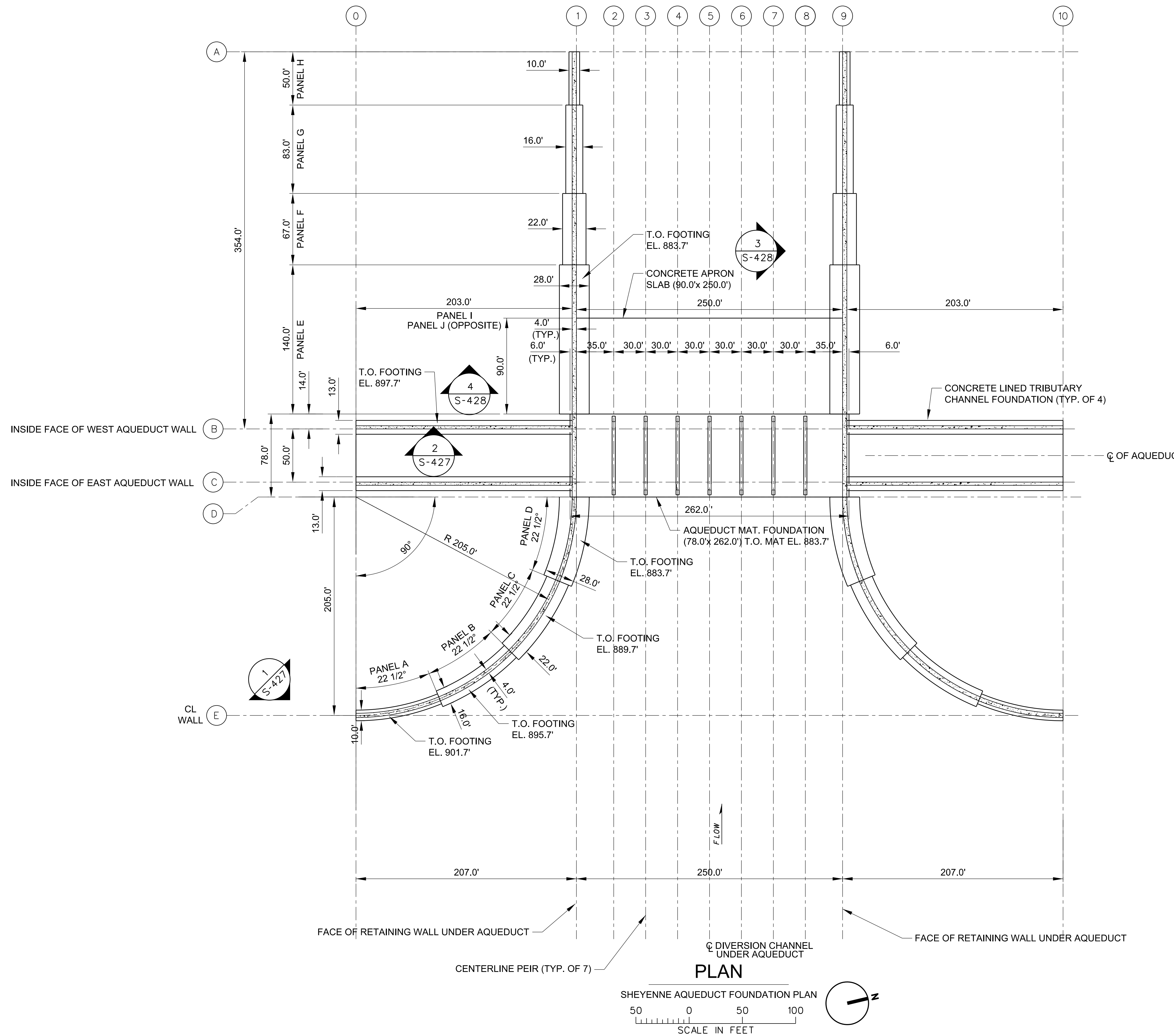
DESIGNED BY: MWT/MBI/JMA | DATE: 28FEB2011
DWN BY: MJJ | CKD BY: BKU/MBI | SOLICITATION NO.: N/A
SUBMITTED BY: N/A | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-424HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
SHEYENNE RIVER AQUEDUCT
PLAN VIEW

SHEET IDENTIFICATION
S-424



PLAN

SHEYENNE AQUEDUCT FOUNDATION PLAN

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3. H-PILES NOT SHOWN FOR CLARITY.

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DATE: 28FEB2011
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SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-425HYD.dgn

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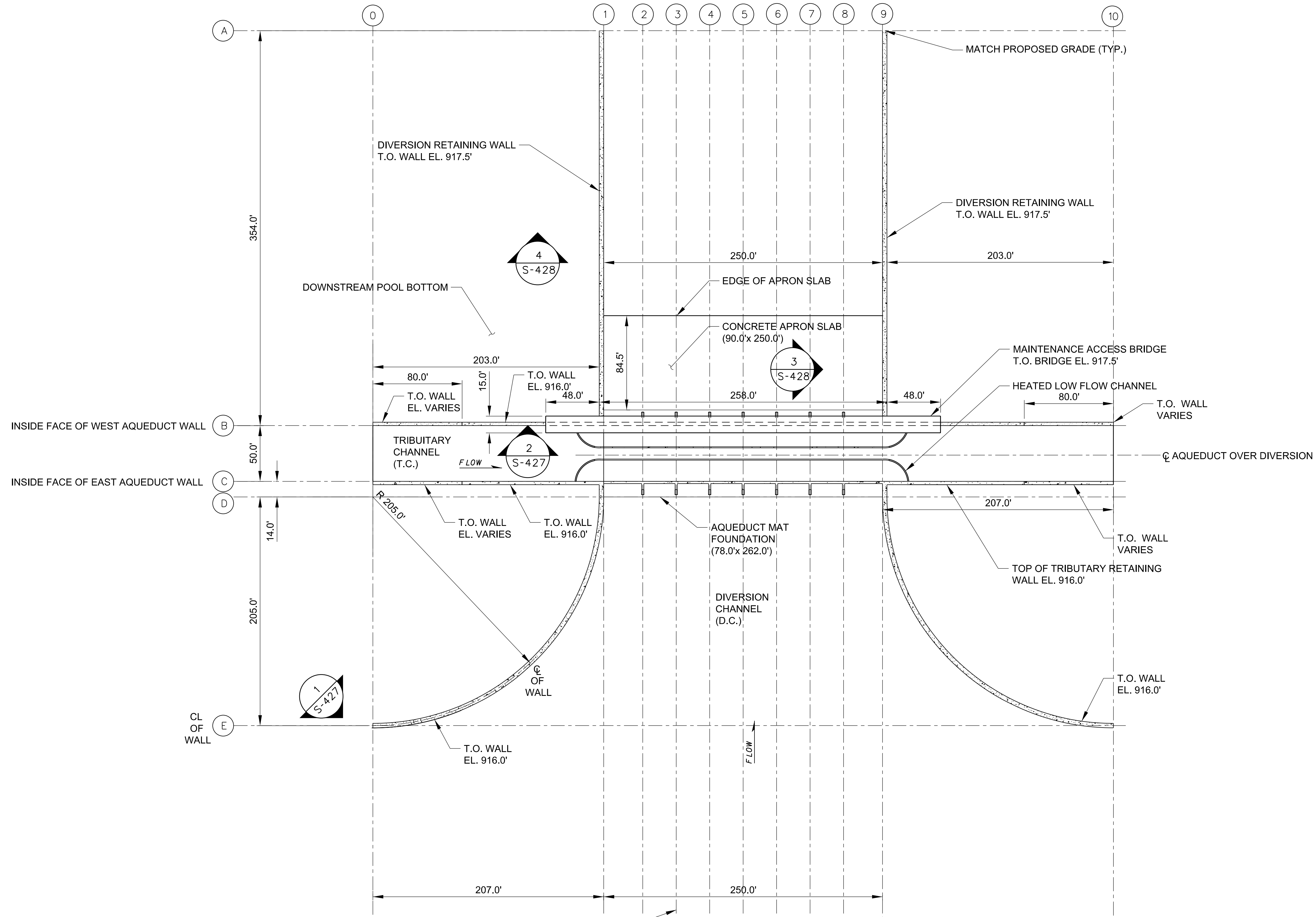
NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - SHEYENNE RIVER
AQUEDUCT FOUNDATION PLAN
PLAN VIEW

SHEET IDENTIFICATION
S-425



PLAN

SHEYENNE AQUEDUCT GENERAL ARRANGEMENT PLAN

50 0 50 100

SCALE IN FEET

N

NOTES:

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ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

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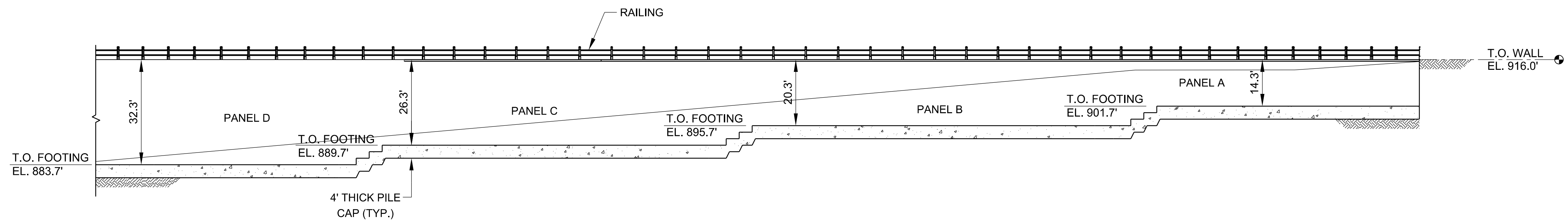
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MWT/MBI/JMA
DWN BY: MJJ
CKD BY: MBI/BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-426HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - SHEYENNE RIVER
AQUEDUCT GENERAL ARRANGEMENT PLAN
PLAN VIEW

SHEET
IDENTIFICATION
S-426

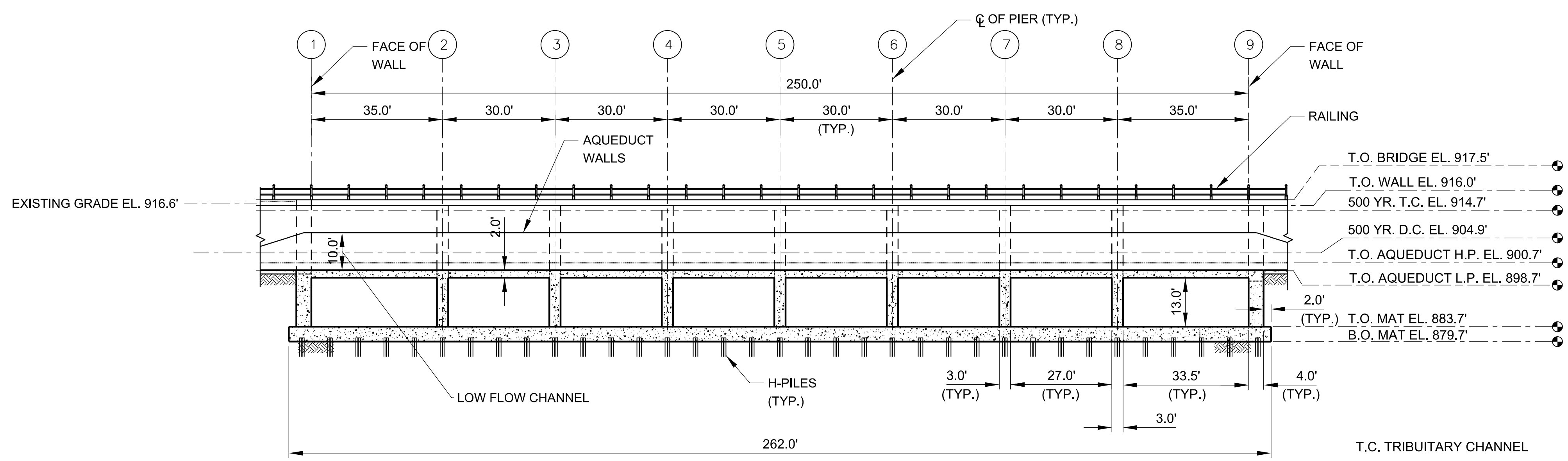


NOTES:
1. H-PILES NOT SHOWN FOR CLARITY.

1 SECTION
S-425, S-426

SHEYENNE RIVER AQUEDUCT
UPSTREAM RETAINING WALL

20 0 20 40
SCALE IN FEET



T.C. TRIBUITARY CHANNEL

D.C. DIVERSION CHANNEL

2 SECTION
S-425, S-426

SHEYENNE RIVER AQUEDUCT STRUCTURE

20 0 20 40
SCALE IN FEET

NOTES:

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ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

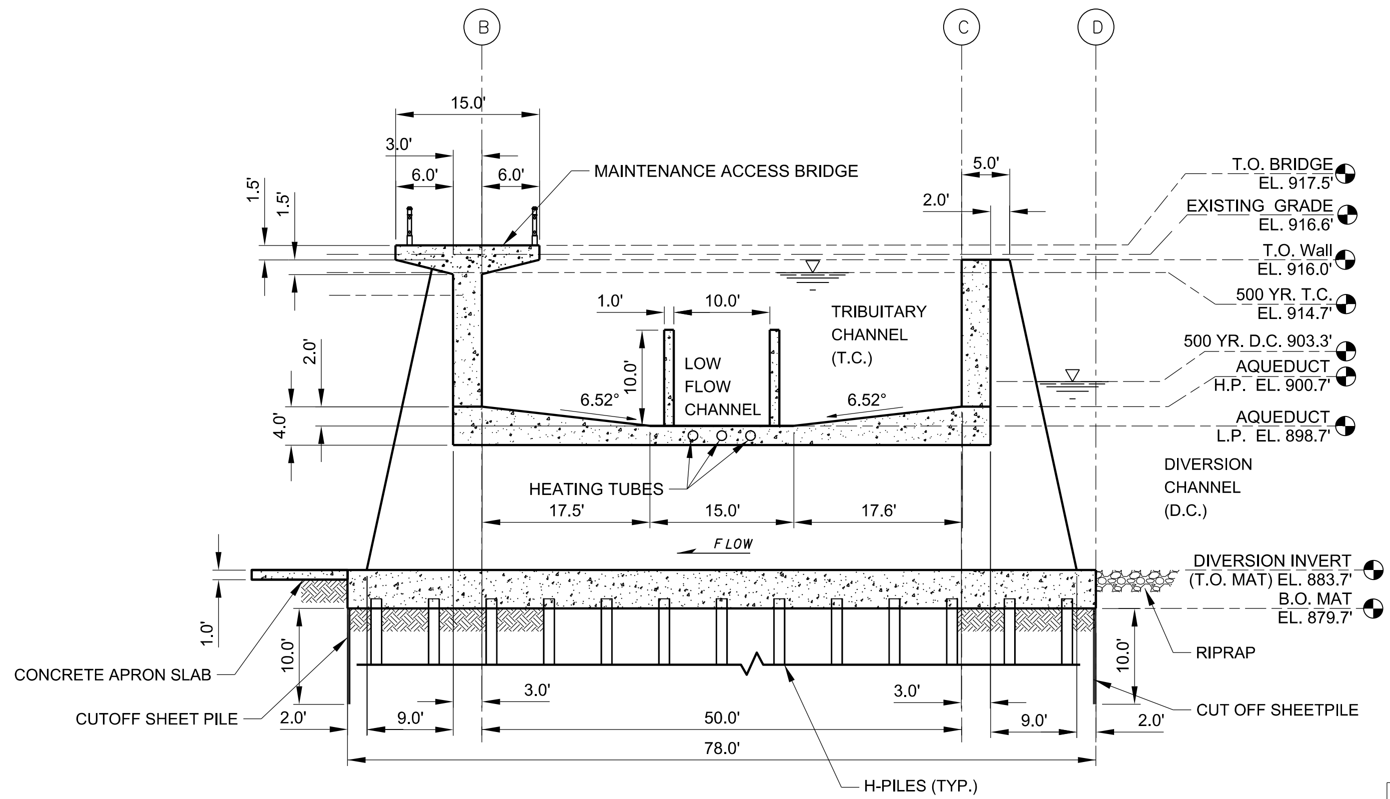
DESIGNED BY: MWT/MBI/JMA
DATE: 28FEB2011
SOLICITATION NO.: N/A
DWN BY: MJJ
CKD BY: BKL/MBI
CONTRACT NO.: N/A
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-427HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - SHEYENNE RIVER
AQUEDUCT
SECTIONS

SHEET
IDENTIFICATION
S-427

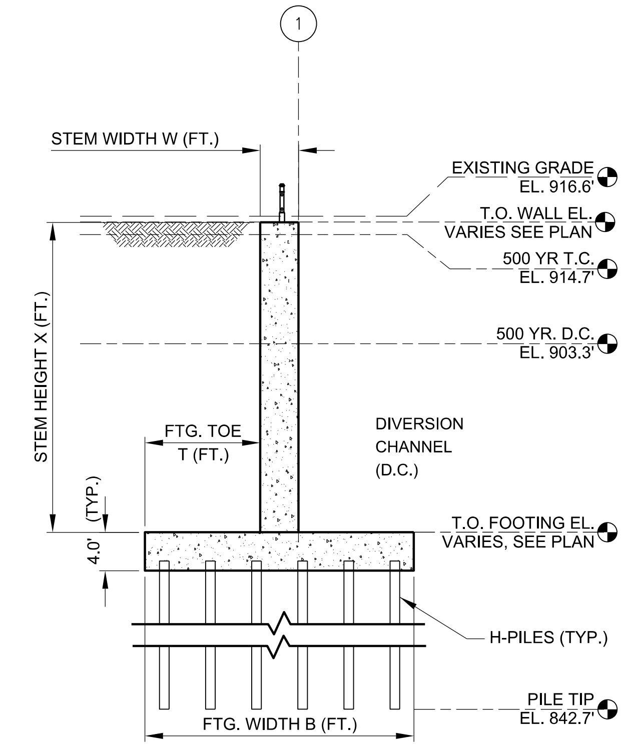


3 S-425, S-426

SECTION

SHEYENNE AQUEDUCT

10 0 10 20 SCALE IN FEET



4 S-425, S-426

SECTION

RETAINING WALL

10 0 10 20 SCALE IN FEET

ALLOWABLE PILE CAPACITY AXIAL COMPRESSION	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	62.0
UNDRAINED UNUSUAL	82.6
UNDRAINED EXTREME	107.7
DRAINED USUAL	36.5

NOTES:

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2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC AND GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

QUANTITIES (AQUEDUCT)

	CONCRETE VOLUMES (CY.)	STEEL (LBS.)
FOOTINGS	3,781	832,483
WALLS	1,310	333,965
DECK ELEVATED SLAB	1,932	539,022
PIERS	874	206,502
TOTAL	7,897	1,911,972

PILES				
NO.	NO. ROWS	NO. COLUMNS	LENGTH (FT.)	TOTAL (FT.)
420	35	12	38	15,960

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

QUANTITIES (RETAINING WALLS)

					CONCRETE VOLUMES		STEEL REINF.		PILES			
PANEL	STEM W (FT)	STEM HEIGHT X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	FTG. (CY)	STEM (CY)	STEM (LBS.)	FTG.	NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	4	14.3	10	3	120	171	31,662	21,897	24	12	56	1,344
B	4	20.3	16	6	191	243	44,929	35,035	36	12	50	1,800
C	4	26.3	22	9	277	314	58,195	48,173	48	12	44	2,112
D	4	32.3	28	12	334	450	83,197	61,311	70	14	38	2,660
E	4	33.8	28	12	581	768	140,972	106,624	115	23	38	4,370
F	4	27.8	22	9	219	277	50,699	40,093	40	10	44	1,760
G	4	21.8	16	6	197	269	49,261	36,122	36	12	50	1,800
H	4	15.8	10	3	74	118	21,515	13,600	14	7	56	784
I	3	18.3	13	5	391	413	101,156	71,781	72	24	52	3,744
J	3	18.3	13	5	391	413	101,156	71,781	72	24	52	3,744
				SOUTH WALL	2,775	3,436	682,742	506,417	527			24,118
				NORTH WALL	2,775	3,436	682,742	506,417	527			24,118
				TOTAL	5,550	6,872	1,365,484	1,012,834	1,054			48,236

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

HORIZONTAL COORDINATE SYSTEM: STATE PLANE - ND SOUTH NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET

US Army Corps of Engineers® St. Paul District

PH4 CONCEPT - NOT FOR CONSTRUCTION 28FEB2011

DESIGNED BY: MW1/PKN/MBI/JMA | DWN BY: MJJ | CKD BY: MBI/BKL | SUBMITTED BY: N/A | PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | SIZE: ANSI D | FILE NAME: FMM_S-428HYD.dgn | DATE: 28FEB2011 | SOLICITATION NO.: N/A | CONTRACT NO.: N/A | FILE NUMBER: N/A

U.S. ARMY CORPS OF ENGINEERS ST. PAUL DISTRICT ST. PAUL, MINNESOTA

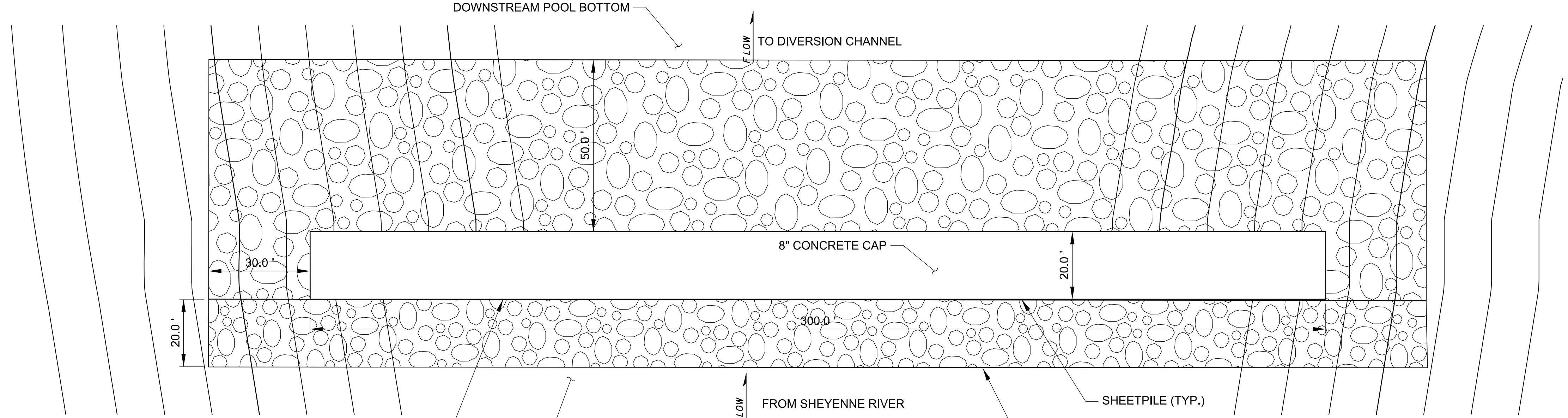
BARR ENGINEERING COMPANY 4700 WEST 77TH STREET MINNEAPOLIS, MN 55435

CONCEPT DOCUMENTATION DRAWINGS RED RIVER DIVERSION HYDRAULIC STRUCTURES FLOOD RISK MANAGEMENT FEASIBILITY STUDY FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - SHEYENNE RIVER AQUEDUCT SECTIONS

NORTH DAKOTA DIVERSION (LPP)

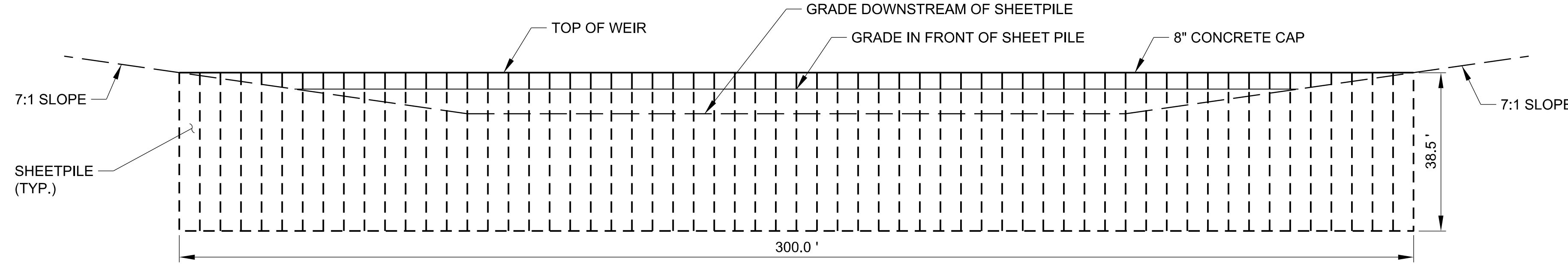
SHEET IDENTIFICATION S-428



PLAN

SHEYENNE AQUEDUCT SPILLWAY WEIRS
(TYP. OF 4)

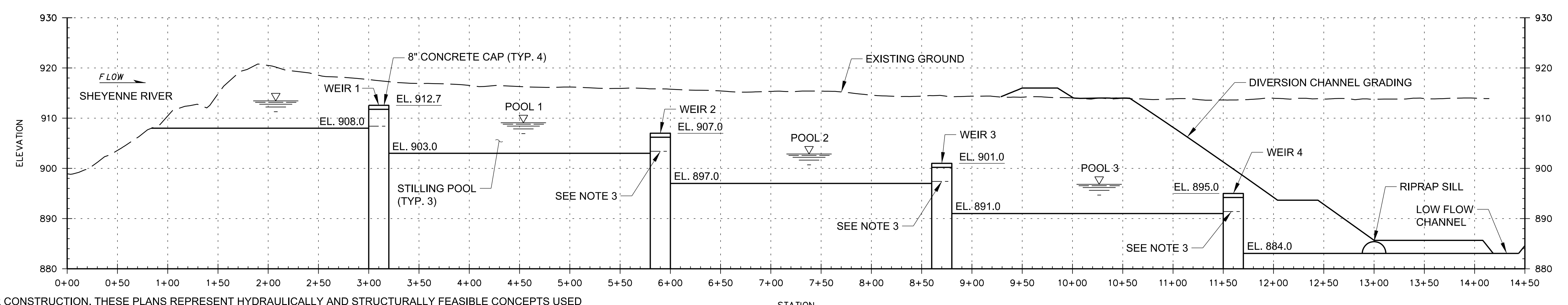
20 0 20 40
SCALE IN FEET



ELEVATION

SHEYENNE AQUEDUCT SPILLWAY WEIR

20 0 20 40
SCALE IN FEET



SECTION

SCHEMATIC SECTION THROUGH
SHEYENNE RIVER SPILLWAY
(N.T.S.)

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC AND GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. SMALL CAPACITY NOTCH IN WEIRS 2-4 AND RIPRAP BERM TO ALLOW STILLING POOLS TO DRAIN.

4. SHEETPILE WALLS TO BE STRUCTURALLY CONNECTED NEAR TOP OF WALL. (NOT SHOWN)

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MW1/PKN/BKL/JMA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-429HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - SHEYENNE RIVER
AQUEDUCT SPILLWAY WEIRS

SHEET IDENTIFICATION
S-429

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

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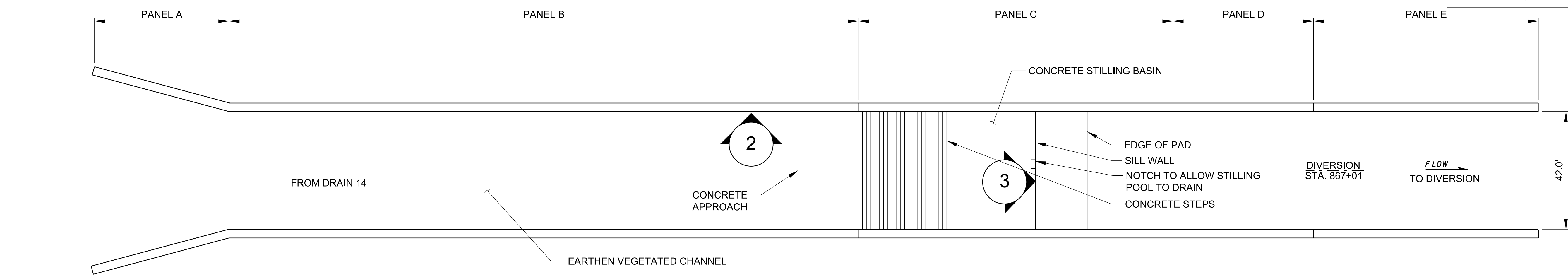
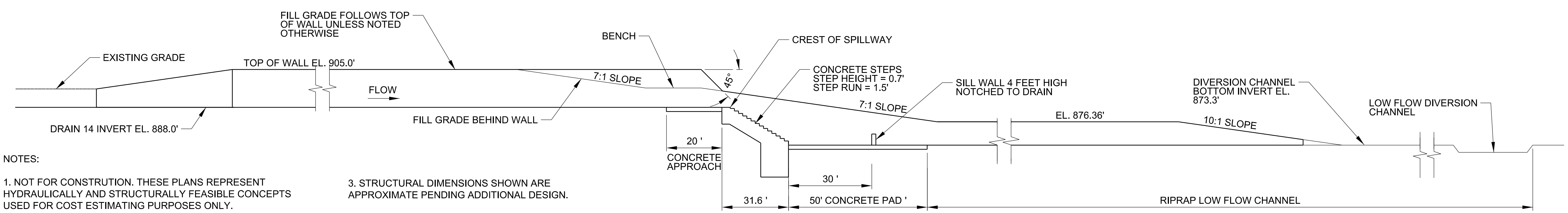
St. Paul District



PLAN

DRAIN 14 FOUNDATION PLAN

20 0 20 40
SCALE IN FEET



NOTES:

1. NOT FOR CONSTRUTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

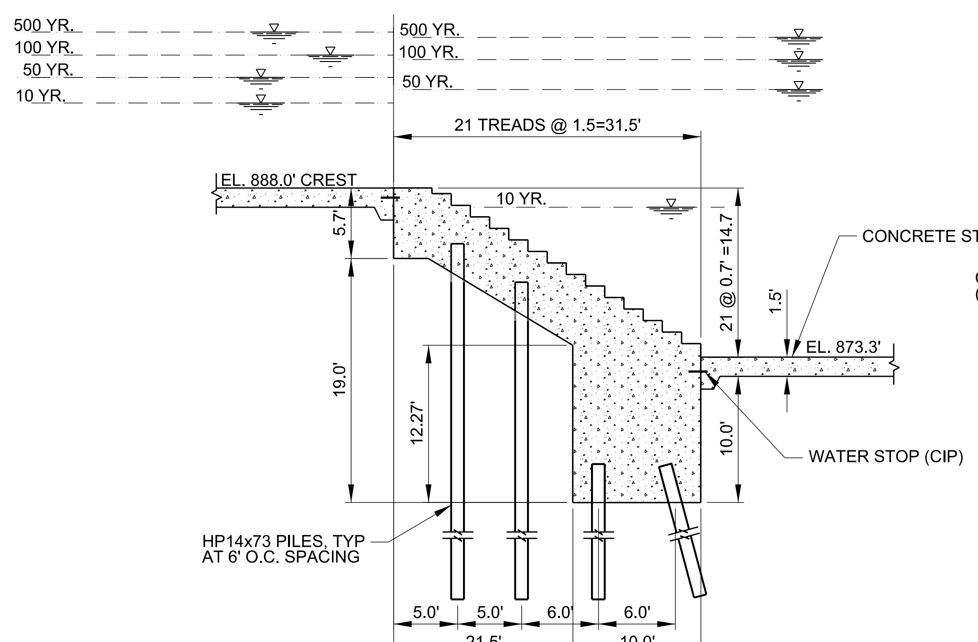
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. FILL BEHIND WALLS FOLLOWS SPOIL PILES FROM DIVERSION.

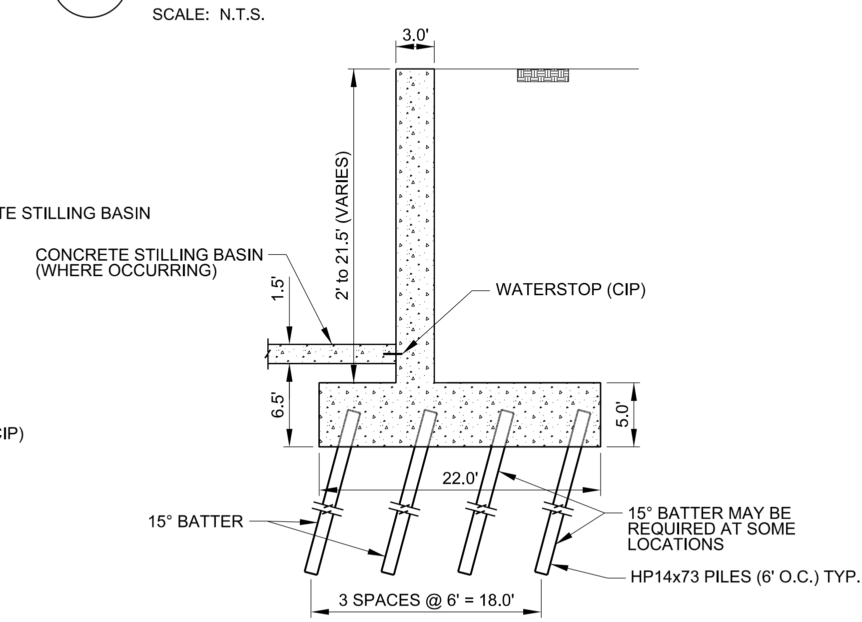
5. PILES NOT SHOWN FOR CLARITY.

1 ELEVATION
SOUTH RETAINING WALL
SCALE: N.T.S.

DIVERSION



2 SECTION
CONCRETE STEPS
SCALE: N.T.S.



3 SECTION
CONCRETE SIDE WALL
SCALE: N.T.S.

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

MARK	DESCRIPTION	DATE	APPR
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWT/JGT/AKH
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: XXX
PLOT SCALE: AS SHOWN
PLOT DATE: 31DEC2009
SIZE: ANSI D
FILE NAME: S-430HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

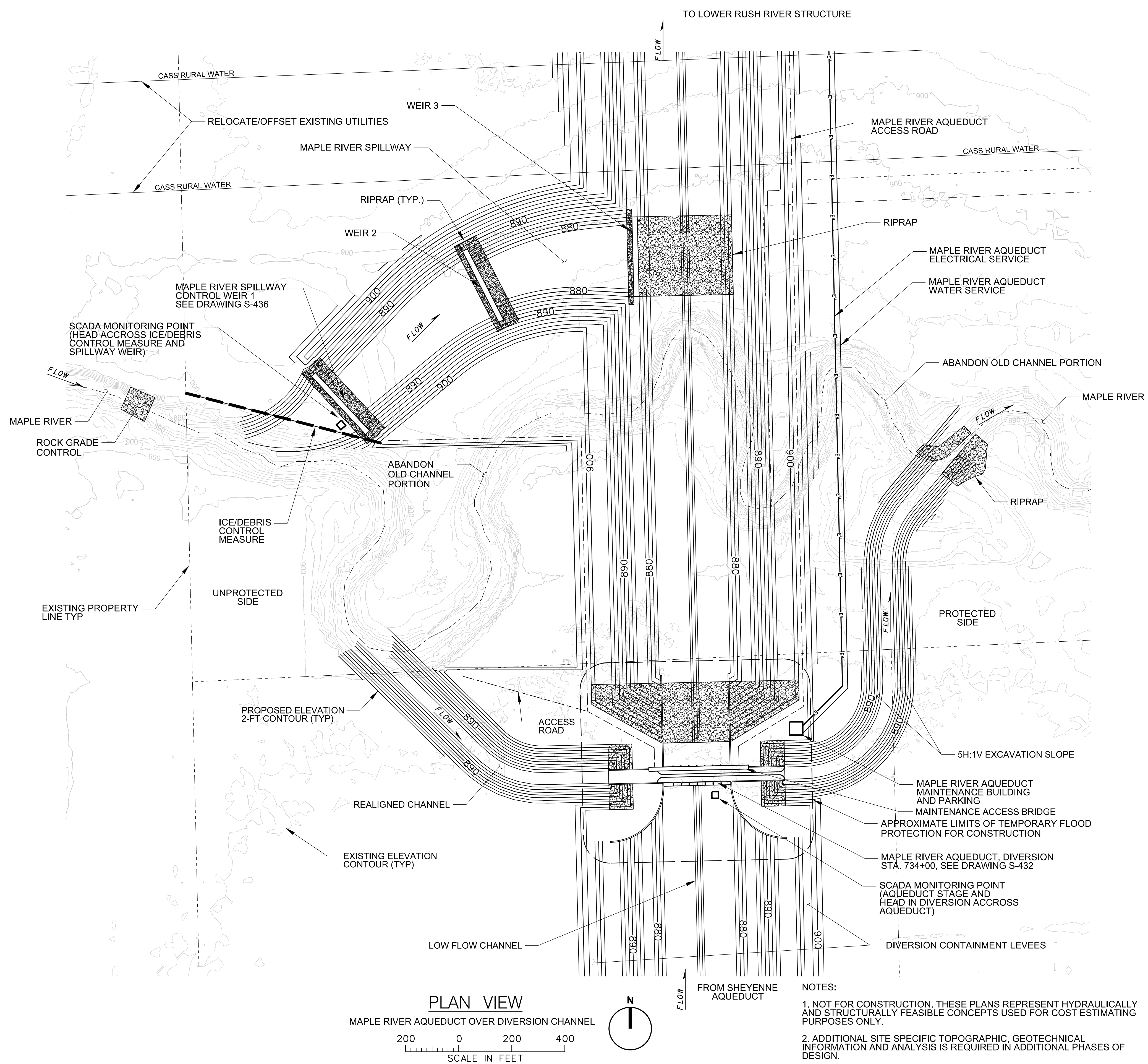
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LLP)

CONCEPT DRAWING
DIVERSION WORKS FEASIBILITY
RED RIVER
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
DRAIN 14 DROP STRUCTURE
PLAN VIEW AND SECTIONS

SHEET IDENTIFICATION
S-430



PLAN VIEW

MAPLE RIVER AQUEDUCT OVER DIVERSION CHANNEL

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

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St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

NORTH DAKOTA DIVERSION (LLP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

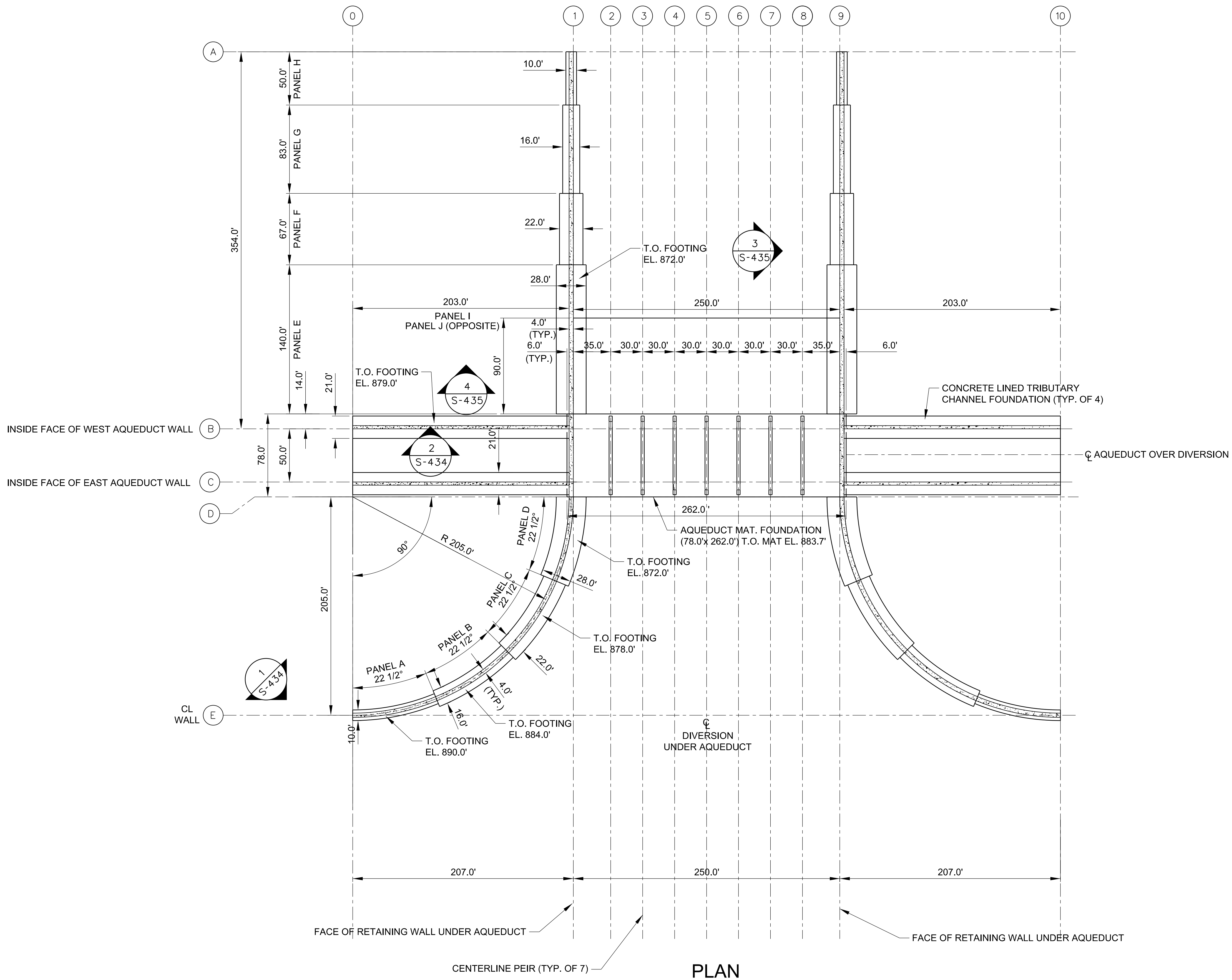
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MBI/MAM/YPKN
DWN BY: MJJ
CKD BY: MBI/BKL
SUBMITTED BY: N/A
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-431HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
MAPLE RIVER AQUEDUCT
PLAN VIEW

SHEET IDENTIFICATION
S-431



PLAN

MAPLE AQUEDUCT FOUNDATION PLAN

50 0 50 100
SCALE IN FEET

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. H-PILES NOT SHOWN FOR CLARITY.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

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St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MW1/MBI/JMA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: MBI/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S432HYD.dgn

U.S. ARMY CORPS OF ENGINEERS
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ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
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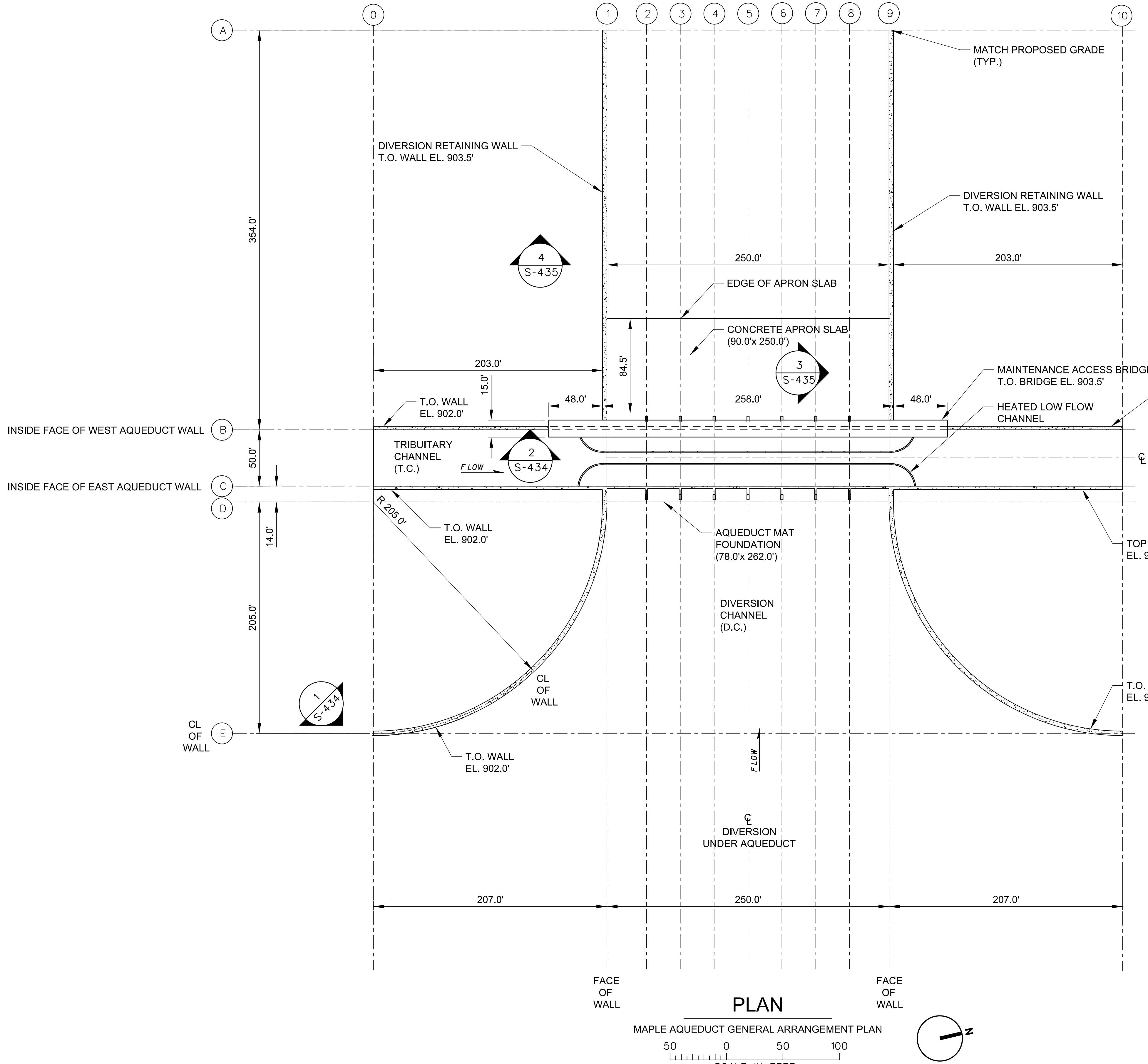
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - MAPLE RIVER
AQUEDUCT FOUNDATION PLAN
PLAN VIEW

SHEET IDENTIFICATION
S-432



PLAN

MAPLE AQUEDUCT GENERAL ARRANGEMENT PLAN

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

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MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

MARK	DESCRIPTION	DATE	APPR.

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ST. PAUL, MINNESOTA

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MINNEAPOLIS, MN 55435

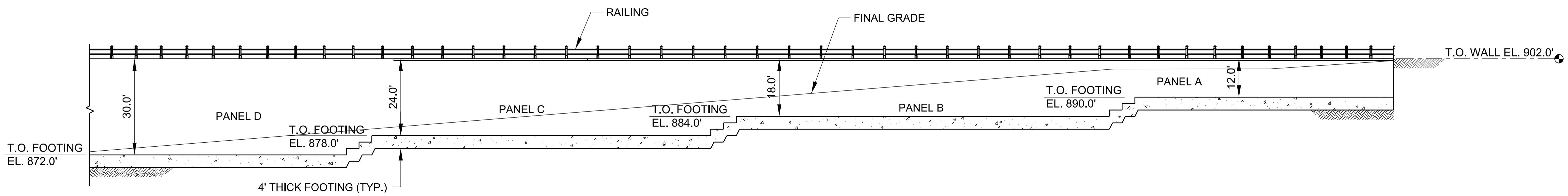
DESIGNED BY: MW1/MBI/JMA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: MBI/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-433HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - MAPLE RIVER
AQUEDUCT GENERAL ARRANGEMENT PLAN
PLAN VIEW

SHEET IDENTIFICATION
S-433

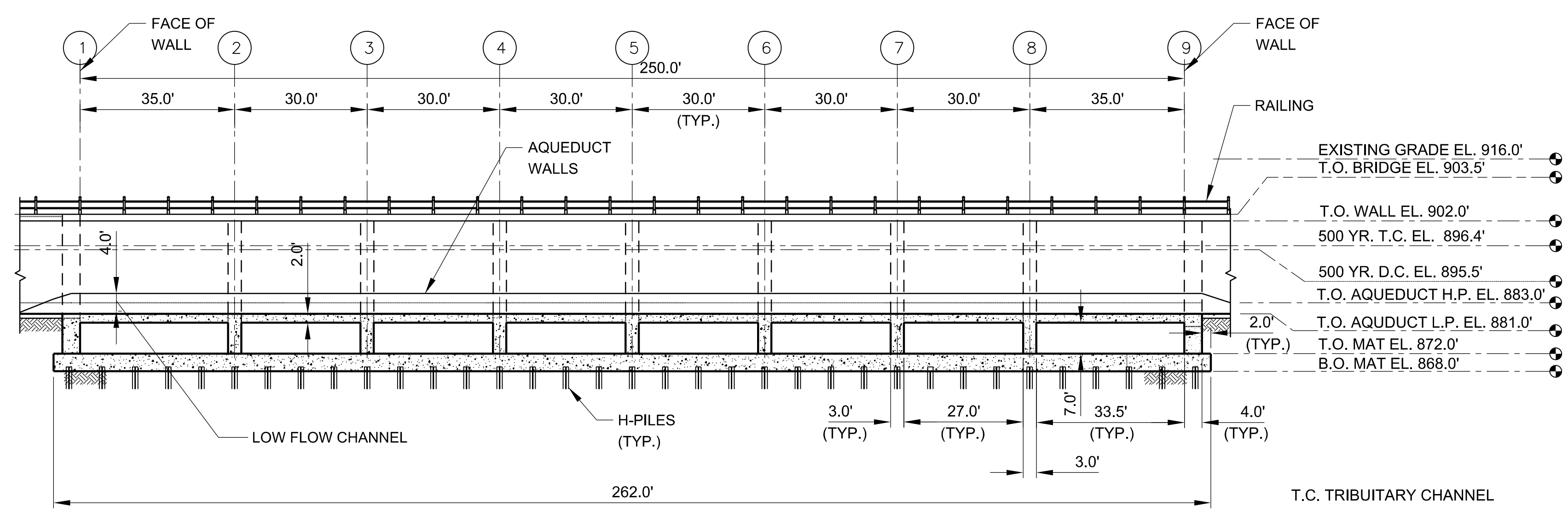


NOTES:
1. H-PILES NOT SHOWN FOR CLARITY.

1 SECTION
S-432, S-433

MAPLE RIVER AQUEDUCT UPSTREAM RETAINING WALL

20 0 20 40
SCALE IN FEET



T.C. TRIBUITARY CHANNEL

D.C. DIVERSION CHANNEL

2 SECTION
S-432, S-433

MAPLE RIVER AQUEDUCT STRUCTURE

20 0 20 40
SCALE IN FEET

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

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HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWT/MBI/JMA | DWN BY: MJJ | CKD BY: BKL/MBI | SUBMITTED BY: N/A | PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | SIZE: ANSI D | FILE NAME: FMM_S-434HYD.dgn | DATE: 28FEB2011 | SOLICITATION NO.: N/A | CONTRACT NO.: N/A | FILE NUMBER: N/A

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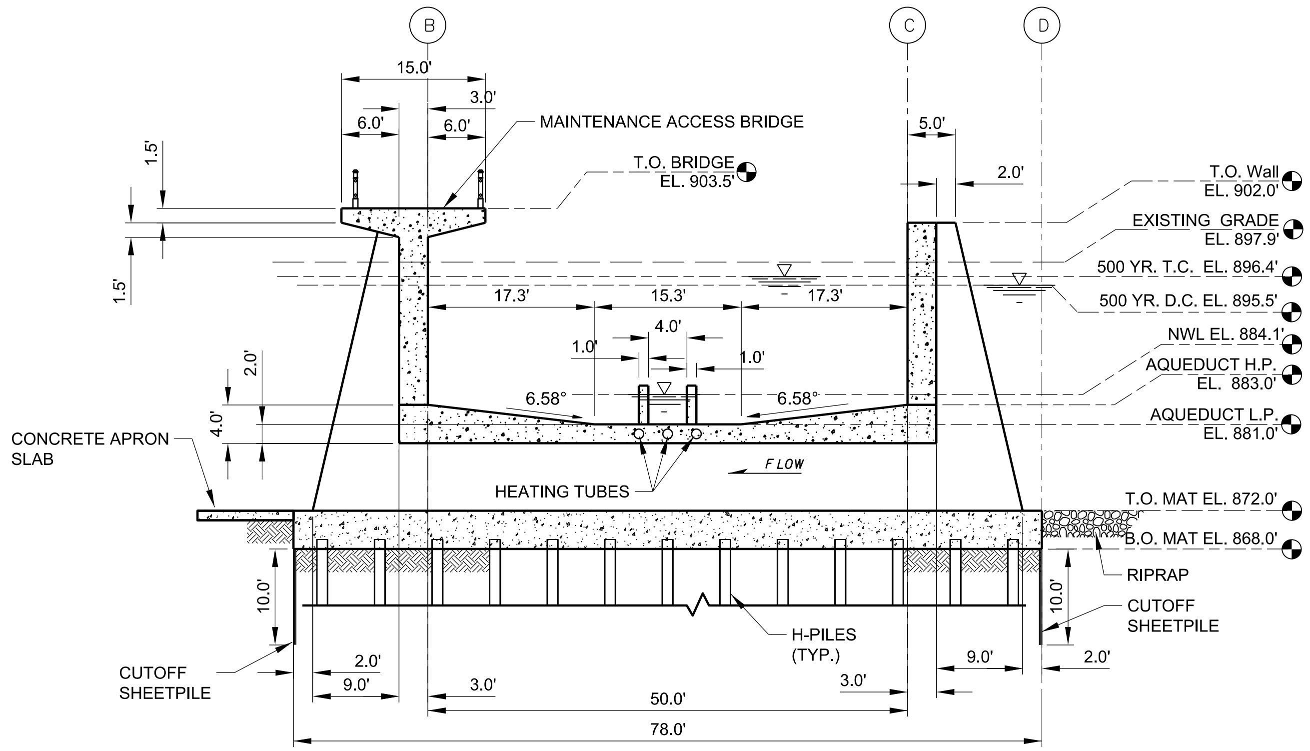
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

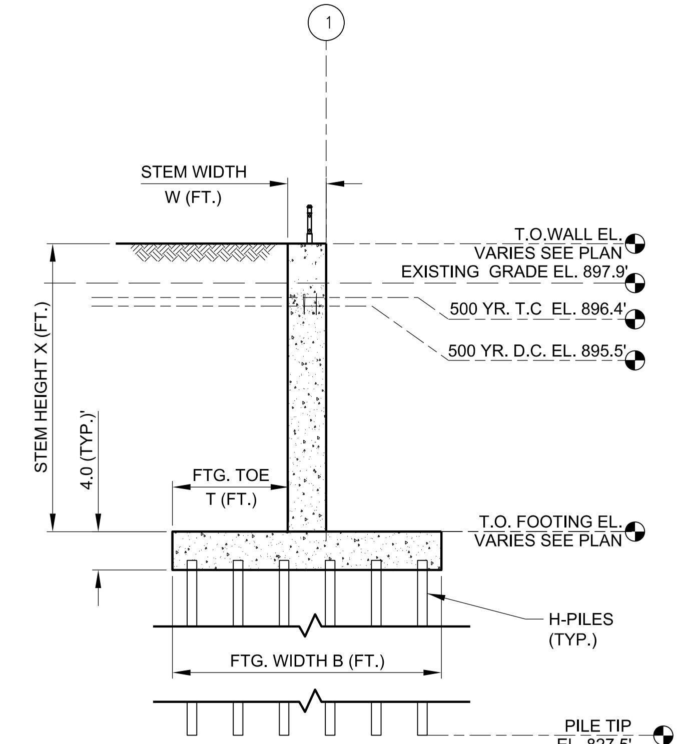
FLOOD CONTROL - MAPLE RIVER AQUEDUCT SECTIONS

SHEET IDENTIFICATION
S-434



3
S-432,
S-433
SECTION
MAPLE RIVER AQUEDUCT

10 0 10 20
SCALE IN FEET



4
S-432,
S-433
SECTION
RETAINING WALL

10 0 10 20
SCALE IN FEET

ALLOWABLE PILE CAPACITY AXIAL COMPRESSION	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	57.2
UNDRAINED UNUSUAL	76.2
UNDRAINED EXTREME	99.4
DRAINED USUAL	31.4

QUANTITIES (AQUEDUCT)

	CONCRETE VOLUMES (CY.)	STEEL (LBS.)
FOOTINGS	3,781	832,483
WALLS	1,268	310,733
DECK ELEVATED SLAB	1,932	539,022
PIERS	561	137,333
TOTAL	7,542	1,819,571

PILES				
NO.	NO. ROWS	NO. COLUMNS	LENGTH (FT.)	TOTAL (FT.)
455	35	13	42	19,110

QUANTITIES (RETAINING WALLS)

					CONCRETE VOLUMES		STEEL (LBS.)		PILES			
PANEL	STEM W (FT)	STEM HEIGHT X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	FTG. CY	STEM CY	FTG.	STEM	NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	4	11.94	10	3	120	143	21,897	26,400	24	12	60	1,440
B	4	17.94	16	6	191	214	35,035	39,666	36	12	54	1,944
C	4	23.94	22	9	263	286	48,173	52,933	52	13	48	2,496
D	4	29.94	28	12	334	418	61,311	77,315	70	14	42	2,940
E	4	31.44	28	12	581	716	106,624	131,411	125	25	42	5,250
F	4	25.44	22	9	219	253	40,093	46,362	44	11	48	2,112
G	4	19.44	16	6	197	239	36,122	43,888	36	12	54	1,944
H	4	13.44	10	3	74	100	13,600	18,278	16	8	60	960
I	3	22.94	21	9	632	518	115,954	126,666	120	30	49	5,880
J	3	22.94	21	9	632	518	115,954	126,666	120	30	49	5,880
			SOUTH RETAINING WALLS		3,243	3,405	594,763	689,585	643			30,846
			NORTH RETAINING WALLS		3,243	3,405	594,763	689,585	643			30,846
			TOTAL		6,486	6,810	1,189,526	1,379,170	1,286			61,692

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

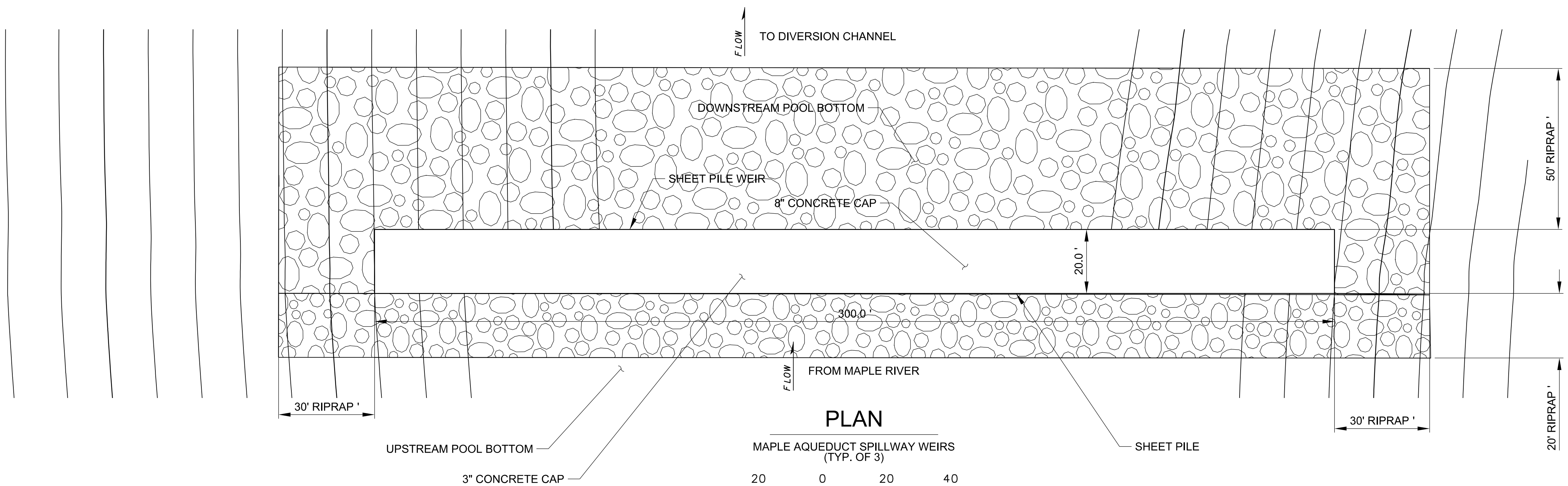
DESIGNED BY: MWT/MBI/JMA
DWN BY: MJJ
CKD BY: BKL/MBI
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-435HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - MAPLE RIVER
AQUEDUCT SECTIONS

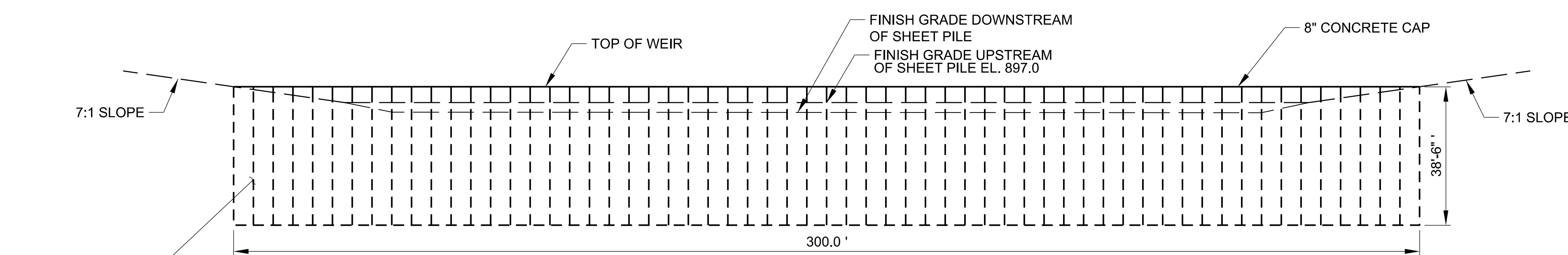
SHEET IDENTIFICATION
S-435



PLAN

MAPLE AQUEDUCT SPILLWAY WEIRS
(TYP. OF 3)

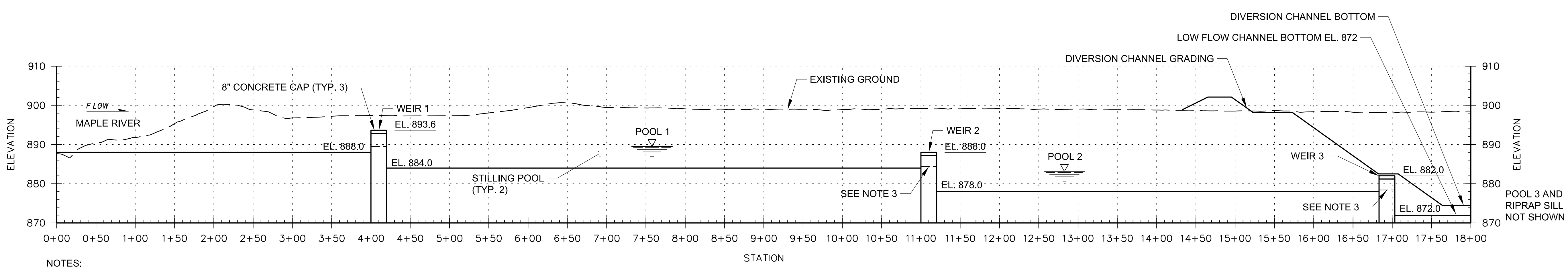
SCALE IN FEET (20 0 20 40)



ELEVATION

MAPLE AQUEDUCT SPILLWAY WEIR

SCALE IN FEET (20 0 20 40)



SECTION

SECTION THROUGH
MAPLE RIVER SPILLWAY
(N.T.S.)

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC AND GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. SMALL CAPACITY NOTCH IN WEIRS 2 AND 3 AND RIPRAP BERM TO ALLOW STILLING POOLS TO DRAIN.
4. SHEET PILE WALLS TO BE STRUCTURALLY CONNECTED NEAR TOP OF WALL (NOT SHOWN)

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

NORTH DAKOTA DIVERSION (LLP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MW1/PKN/BKL/JMA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-436HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - MAPLE RIVER
AQUEDUCT SPILLWAY WEIRS

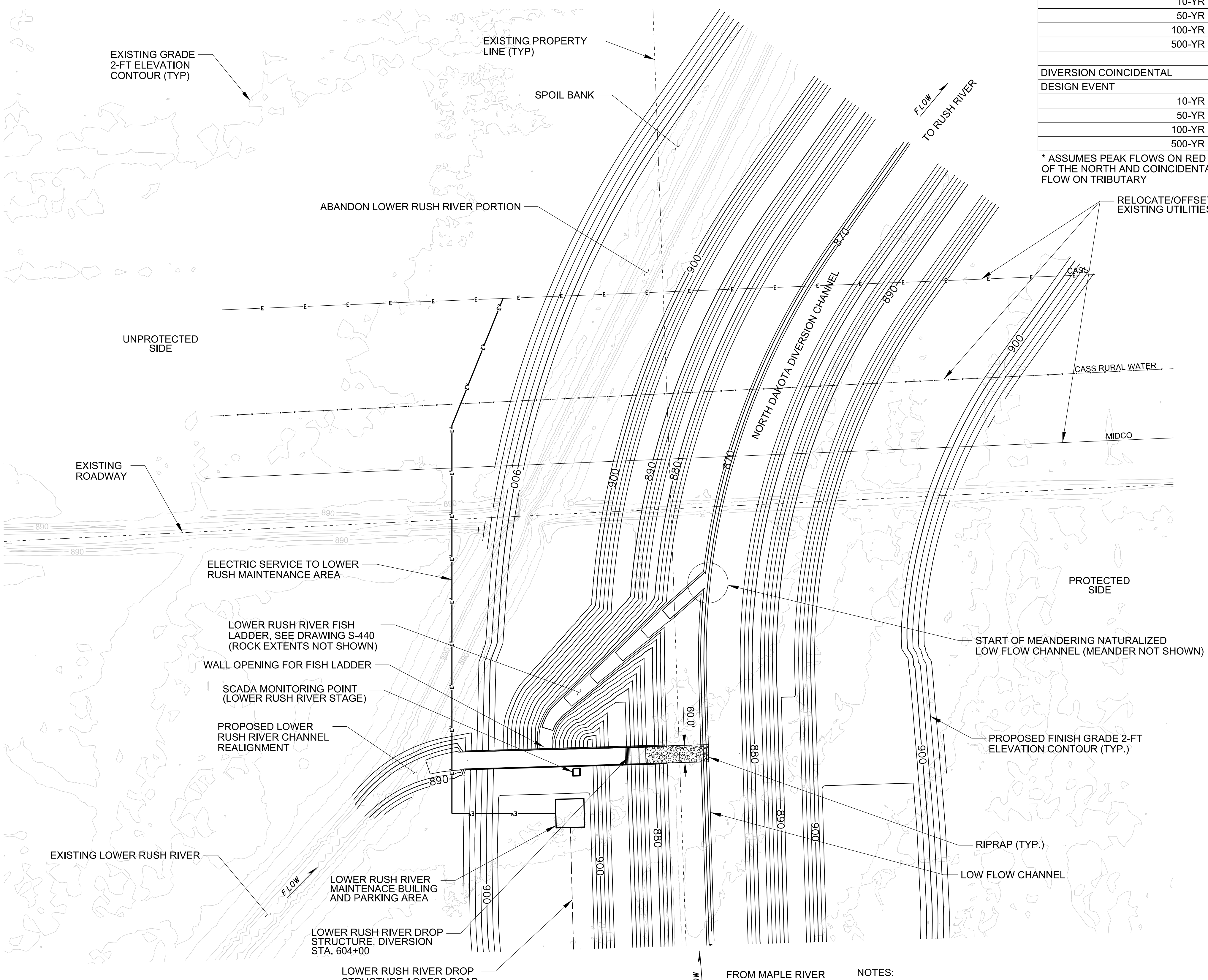
SHEET IDENTIFICATION
S-436

TABLE: DESIGN WATER SURFACE ELEVATIONS AT LOWER RUSH RIVER STRUCTURE

LOWER RUSH COINCIDENTAL		LOWER RUSH LOCAL	
DESIGN EVENT	WSEL	DESIGN EVENT	WSEL
10-YR	892.87	10-YR	892.84
50-YR	894.00	50-YR	894.32
100-YR	894.38	100-YR	894.60
500-YR	894.69	500-YR	894.88
DIVERSION COINCIDENTAL		DIVERSION LOCAL	
DESIGN EVENT		DESIGN EVENT	
10-YR	884.93	10-YR	881.80
50-YR	891.41	50-YR	888.70
100-YR	892.84	100-YR	890.62
500-YR	893.24	500-YR	892.49

* ASSUMES PEAK FLOWS ON RED RIVER OF THE NORTH AND COINCIDENTAL FLOW ON TRIBUTARY

* ASSUMES LOCAL PEAK FLOW ON TRIBUTARY AND COINCIDENTAL FLOW ON RED RIVER OF THE NORTH (STRUCTURE WAS SIZED BASED ON THIS CRITERIA)



PLAN VIEW

LOWER RUSH RIVER DROP STRUCTURE

200 0 200 400
SCALE IN FEET

N

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. FISH PASSAGE ROCK EXTENTS NOT SHOWN (FOR CLARITY).

HORIZONTAL COORDINATE SYSTEM: STATE PLANE - ND SOUTH NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET

US Army Corps of Engineers® St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWT/JGT/JMA | DWN BY: MJJ | CKD BY: PKN/BKL | SUBMITTED BY: N/A | PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | SIZE: ANSI D | FILE NAME: FMM_S-437HY0.dgn | DATE: 28FEB2011 | SOLICITATION NO.: N/A | CONTRACT NO.: N/A | FILE NUMBER: N/A

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

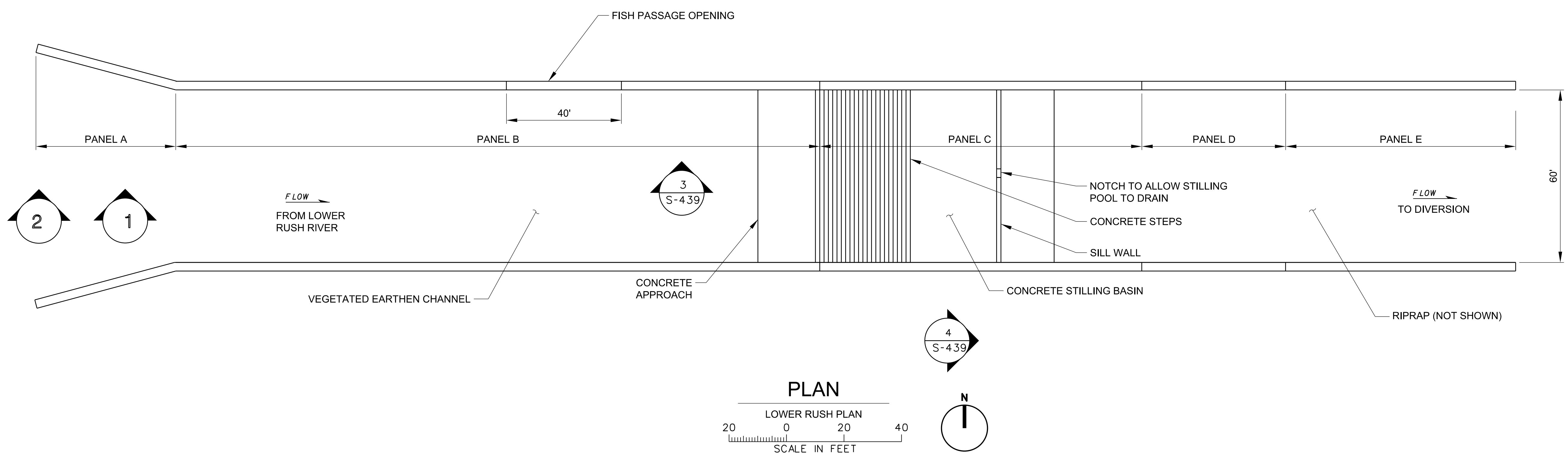
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

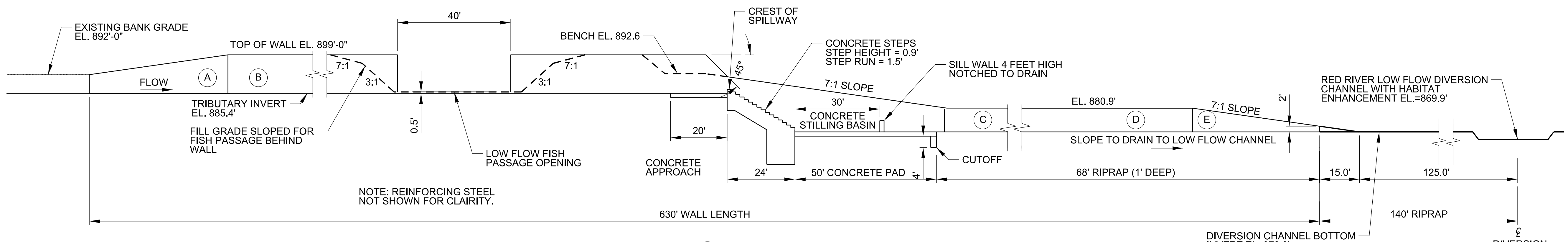
FLOOD CONTROL
LOWER RUSH RIVER DROP STRUCTURE
PLAN VIEW

SHEET IDENTIFICATION
S-437



PLAN

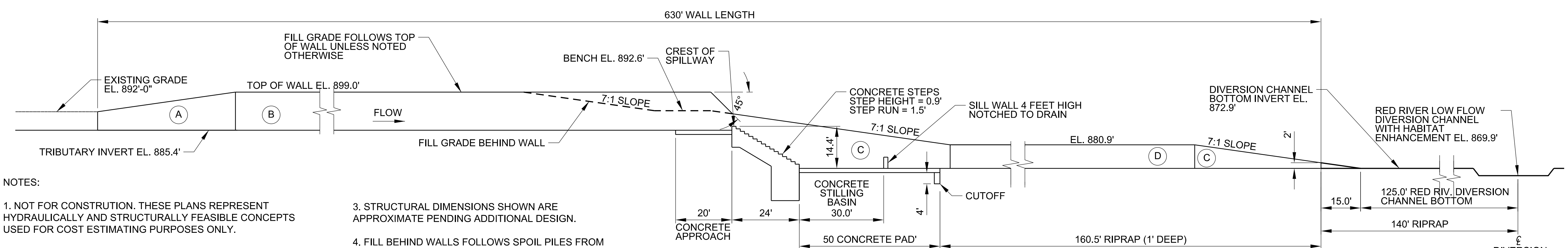
LOWER RUSH PLAN



1 ELEVATION

NORTH RETAINING WALL

SCALE: N.T.S.



2 ELEVATION

SOUTH RETAINING WALL

SCALE: N.T.S.

NOTES:

1. NOT FOR CONSTRUTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

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3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. FILL BEHIND WALLS FOLLOWS SPOIL PILES FROM DIVERSION AND FISH PASSAGE EXCAVATION.

5. PILES NOT SHOWN FOR CLARITY.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: JGT/MW1/JMA | DATE: 28FEB2011
DWN BY: MJJ | CKD BY: PKN/BKL | SOLICITATION NO.: N/A
SUBMITTED BY: N/A | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-438HYD.dgn

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

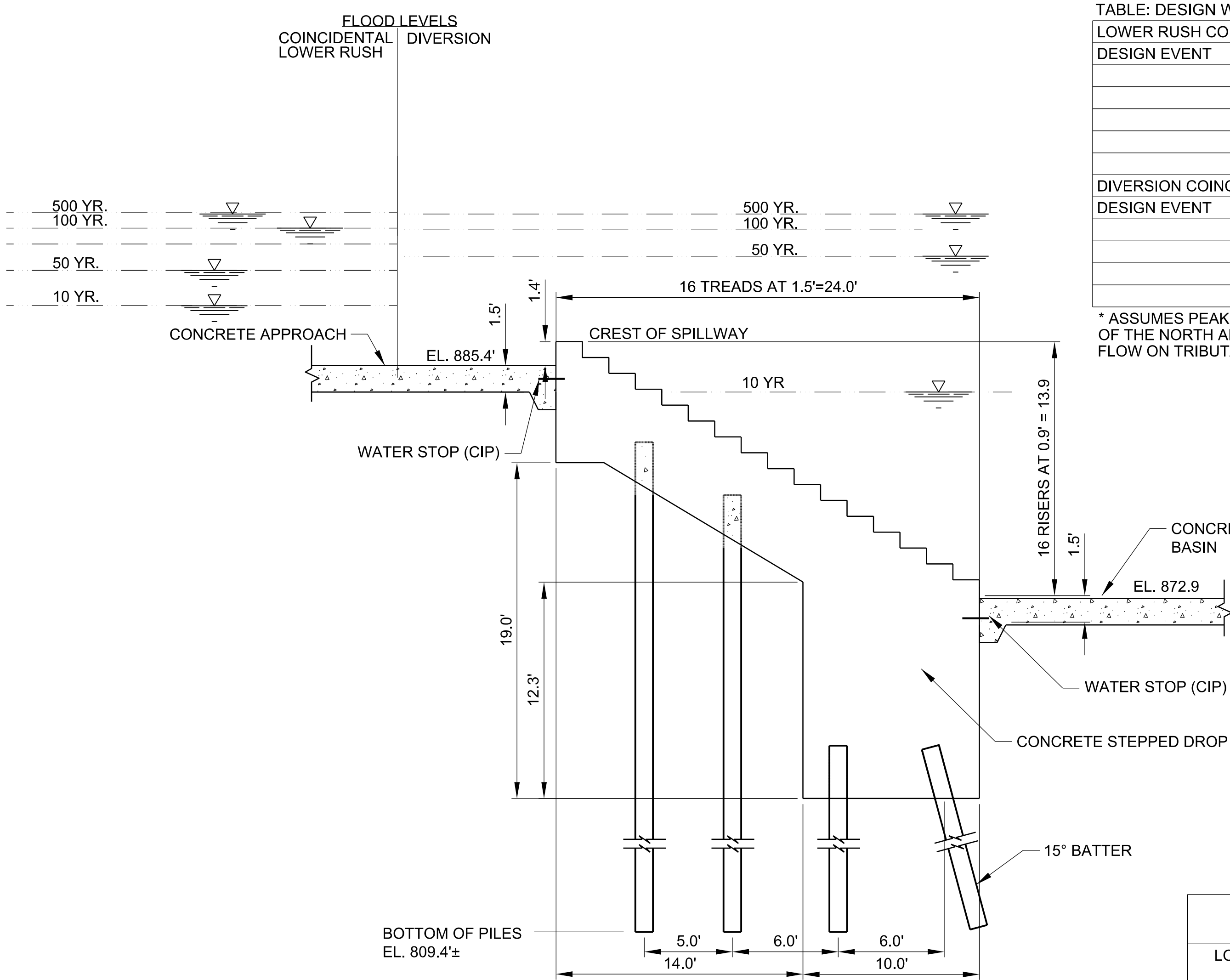
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LLP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LOWER RUSH RIVER
DROP STRUCTURE PLAN
AND ELEVATIONS

SHEET IDENTIFICATION
S-438



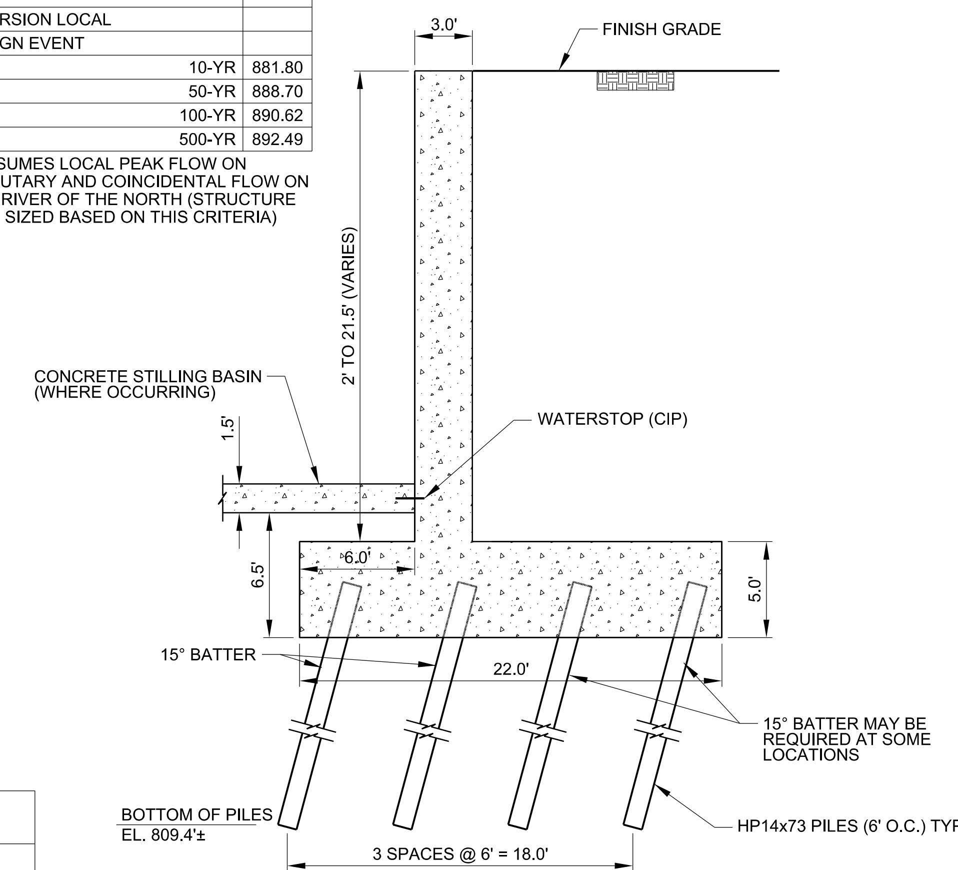
3 S-438 SECTION
CONCRETE STEPPED DROP
SCALE: N.T.S.

TABLE: DESIGN WATER SURFACE ELEVATIONS AT LOWER RUSH RIVER STRUCTURE

LOWER RUSH COINCIDENTAL		LOWER RUSH LOCAL	
DESIGN EVENT	WSEL	DESIGN EVENT	WSEL
10-YR	892.87	10-YR	892.84
50-YR	894.00	50-YR	894.32
100-YR	894.38	100-YR	894.60
500-YR	894.69	500-YR	894.88
DIVERSION COINCIDENTAL		DIVERSION LOCAL	
DESIGN EVENT		DESIGN EVENT	
10-YR	884.93	10-YR	881.80
50-YR	891.41	50-YR	888.70
100-YR	892.84	100-YR	890.62
500-YR	893.24	500-YR	892.49

* ASSUMES PEAK FLOWS ON RED RIVER OF THE NORTH AND COINCIDENTAL FLOW ON TRIBUTARY

* ASSUMES LOCAL PEAK FLOW ON TRIBUTARY AND COINCIDENTAL FLOW ON RED RIVER OF THE NORTH (STRUCTURE WAS SIZED BASED ON THIS CRITERIA)



4 S-438 SECTION
CONCRETE SIDE WALL
SCALE: N.T.S.

ALLOWABLE PILE CAPACITY AXIAL COMPRESSION	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	78.5
UNDRAINED UNUSUAL	104.7
UNDRAINED EXTREME	136.5
DRAINED USUAL	47.0

QUANTITIES (DROP STRUCTURE)

CONCRETE VOLUMES			STEEL	PILES			
APPROACH	STILLING	STEPS	REINF.		NO.	LENGTH	TOTAL
CY	CY	CY	LBS	NO.	ROWS	FT	FT
70	170	517	118,720	20	10	52	1,040
				20	10	60	1,200
			TOTAL	40			2,240

QUANTITIES (WALLS)

PANEL	STEM W (FT)	WALL LENGTH (FT)	STEM HEIGHT X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. CY	STEM CY	STEEL REINF. LBS	PILES NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	3	72'	13.1	22	6	294	105	52,452	48	24	74	3,552
B	3	416'	16.5	22	6	1695	726	319,806	280	140	74	20,720
C	3	85'	16	22	6	347	151	66,261	56	28	55	3,080
D	3	15'	11	22	6	61	16	10,373	12	6	55	660
E	3	42'	8	22	6	171	31	26,827	14	7	55	770
					SOUTH WALL	2,568	1,029	475,719	410			28,782
					NORTH WALL	2,568	1,029	475,719	410			28,782
					TOTAL	5,136	2,058	951,438	820			57,564

NOTES:

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3. THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER SLIGHTLY FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

NORTH DAKOTA DIVERSION (LLP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

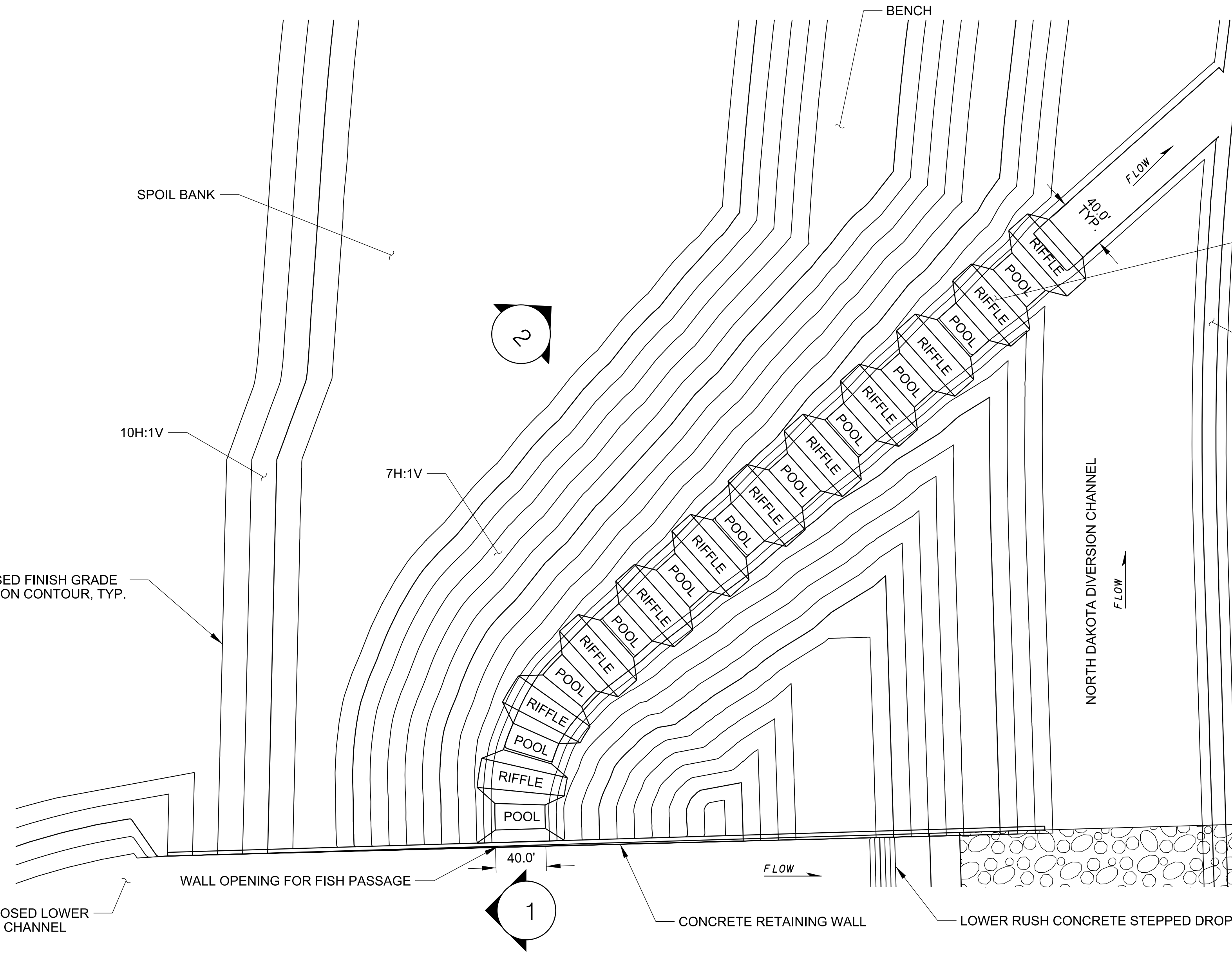
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LOWER RUSH RIVER
DROP STRUCTURE
SECTIONS

PH4 CONCEPT - NOT FOR CONSTRUCTION 28FEB2011

SHEET IDENTIFICATION
S-439

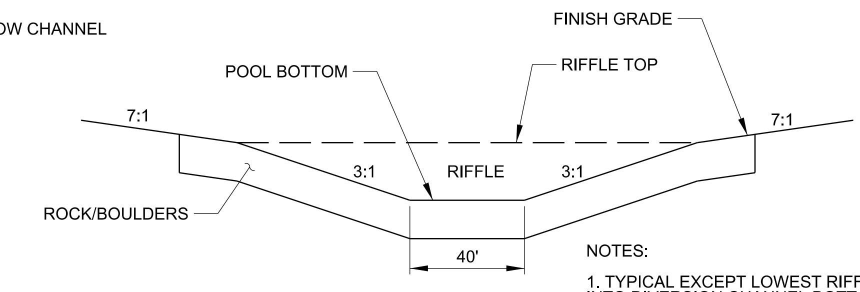


PLAN VIEW

LOWER RUSH RIVER FISH PASSAGE

200 0 200 400

SCALE IN FEET



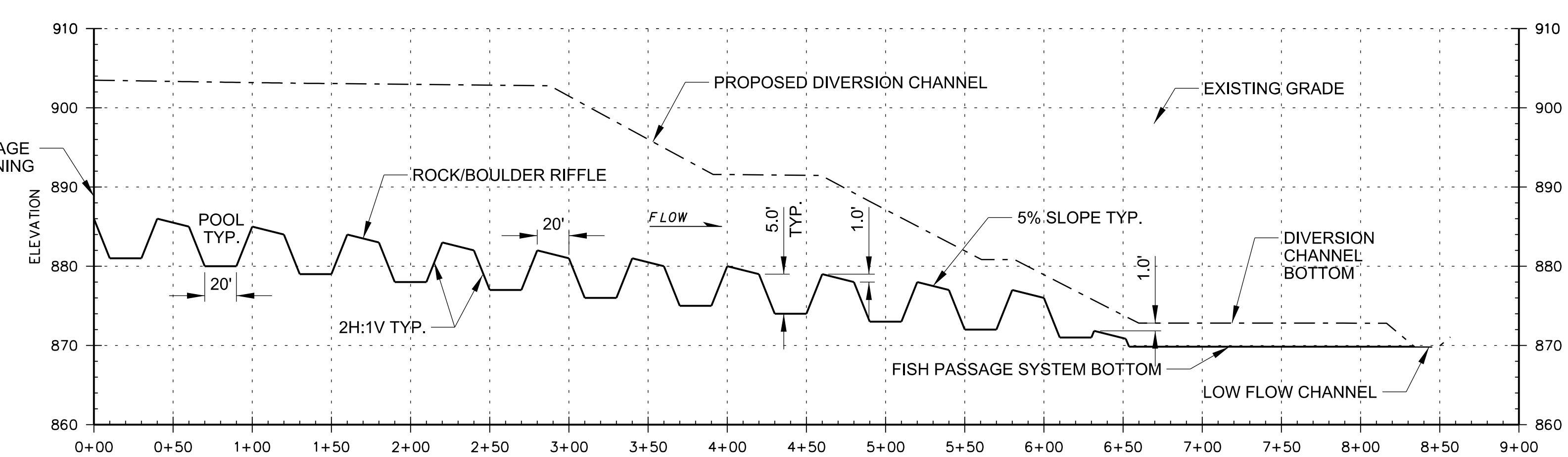
NOTES:

1. TYPICAL EXCEPT LOWEST RIFFLE CUT INTO DIVERSION CHANNEL BOTTOM

2 SECTION

TYPICAL FISH PASSAGE

SCALE: N.T.S.



1 PROFILE

LOWER RUSH FISH PASSAGE SYSTEM–CENTERLINE

SCALE: N.T.S.

NOTES:

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3. FISH PASSAGE ROCK EXTENTS NOT SHOWN (FOR CLARITY).

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWT/SGW/JMA	DATE: 28FEB2011	
DWN BY: MJJ	CKD BY: BKL	SOLICITATION NO.: N/A
SUBMITTED BY: N/A		CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011	FILE NUMBER: N/A
SIZE: ANSI D	FILE NAME: FMM_S-440HYD.dgn	

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LOWER RUSH
RIVER FISH PASSAGE SYSTEM
PLAN VIEW, PROFILE AND SECTION

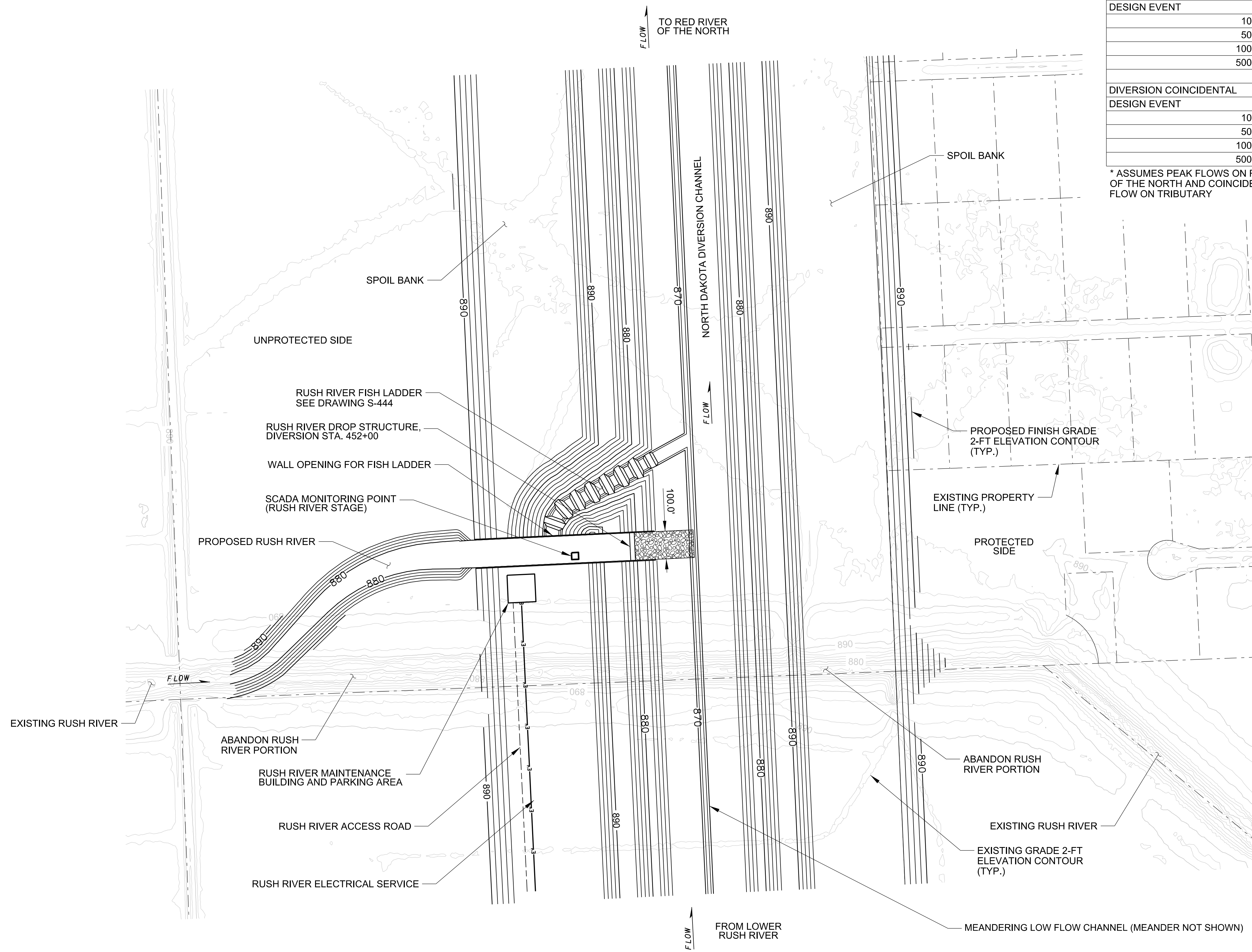
SHEET IDENTIFICATION
S-440

TABLE: DESIGN WATER SURFACE ELEVATIONS AT RUSH RIVER STRUCTURE

RUSH COINCIDENTAL		RUSH LOCAL	
DESIGN EVENT	WSEL	DESIGN EVENT	WSEL
10-YR	888.97	10-YR	886.68
50-YR	950.52	50-YR	887.98
100-YR	891.36	100-YR	888.77
500-YR	891.82	500-YR	890.63
DIVERSION COINCIDENTAL		DIVERSION LOCAL	
DESIGN EVENT		DESIGN EVENT	
10-YR	883.28	10-YR	880.05
50-YR	889.08	50-YR	886.77
100-YR	890.47	100-YR	888.47
500-YR	890.90	500-YR	890.32

* ASSUMES PEAK FLOWS ON RED RIVER OF THE NORTH AND COINCIDENTAL FLOW ON TRIBUTARY

* ASSUMES LOCAL PEAK FLOW ON TRIBUTARY AND COINCIDENTAL FLOW ON RED RIVER OF THE NORTH (STRUCTURE WAS SIZED BASED ON THIS CRITERIA)



PLAN VIEW

RUSH RIVER DROP STRUCTURE

200 0 200 400
SCALE IN FEET

N

NOTES:

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3. FISH PASSAGE ROCK EXTENTS NOT SHOWN (FOR CLARITY).

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

NORTH DAKOTA DIVERSION (LPP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

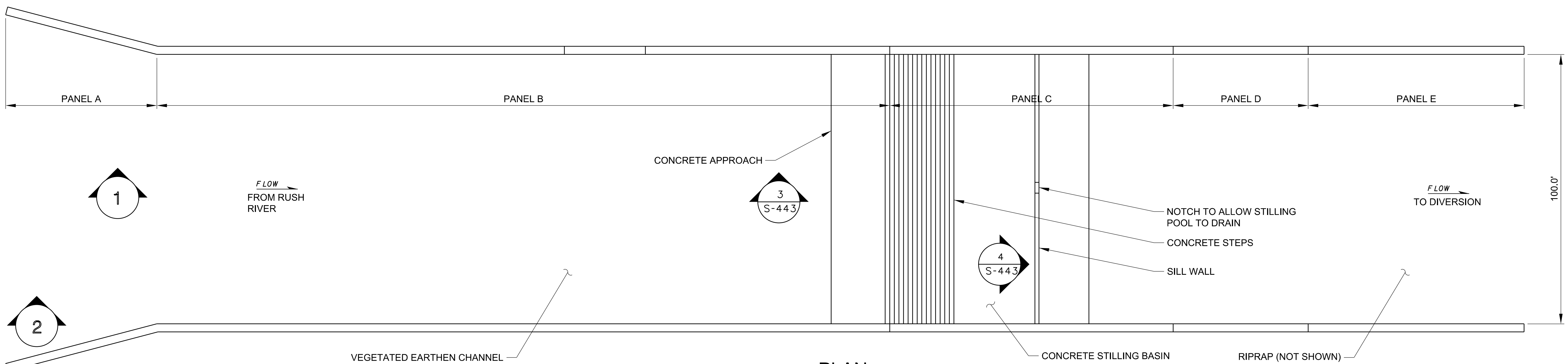
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MWT/SGW/JMA | DATE: 28FEB2011
DWN BY: MJJ | CKD BY: PKN/BKL | SOLICITATION NO.: N/A
SUBMITTED BY: N/A | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-441HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

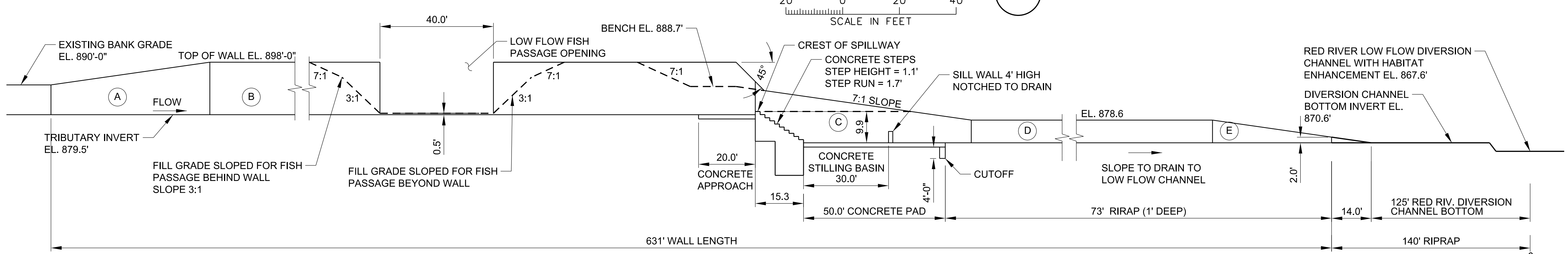
FLOOD CONTROL
RUSH RIVER DROP STRUCTURE
PLAN VIEW

SHEET IDENTIFICATION
S-441

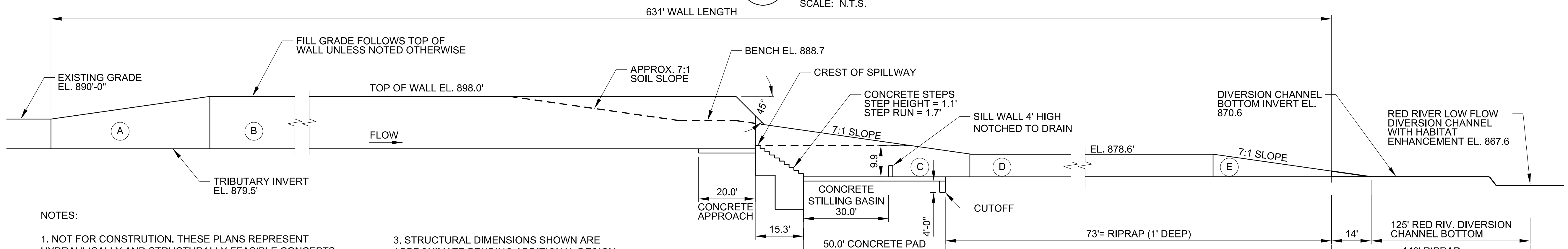


PLAN

RUSH PLAN
20 0 20 40
SCALE IN FEET



1 ELEVATION
NORTH RETAINING WALL
SCALE: N.T.S.



NOTES:

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3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. FILL BEHIND WALLS FOLLOWS SPOIL PILES FROM DIVERSION AND FISH PASSAGE EXCAVATION.

5. PILES NOT SHOWN FOR CLARITY.

2 ELEVATION
SOUTH RETAINING WALL
SCALE: N.T.S.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

SOUTH RETAINING WALL

US Army Corps of Engineers ®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWT/SGW/JMA | DATE: 28FEB2011
DWN BY: MJJ | CKD BY: BKL | SOLICITATION NO.: N/A
SUBMITTED BY: XXX | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-442HYD.dgn

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

NORTH DAKOTA DIVERSION (LLP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - RUSH RIVER
DROP STRUCTURE PLAN
AND ELEVATIONS

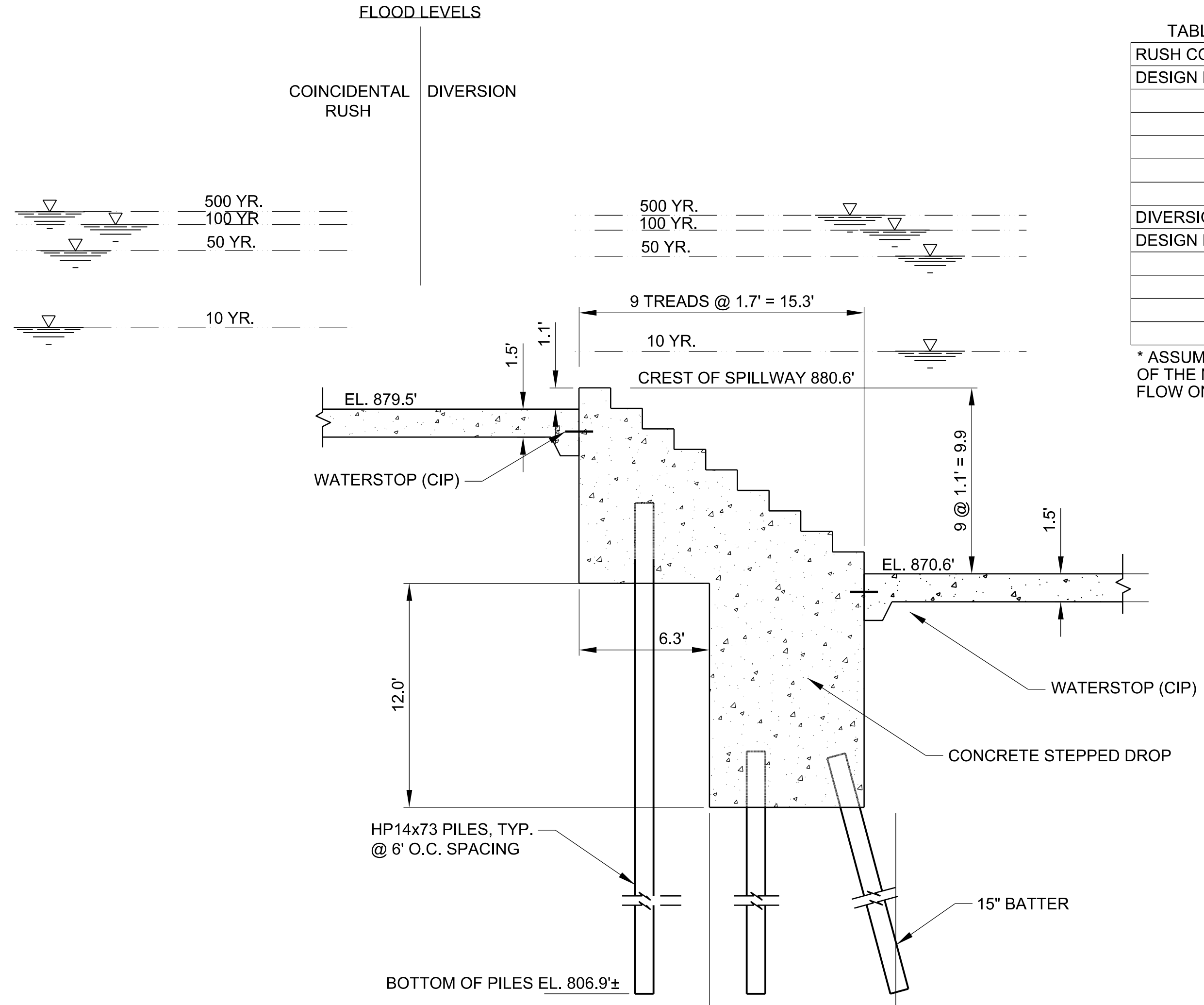
SHEET IDENTIFICATION
S-442

TABLE: DESIGN WATER SURFACE ELEVATIONS AT RUSH RIVER STRUCTURE

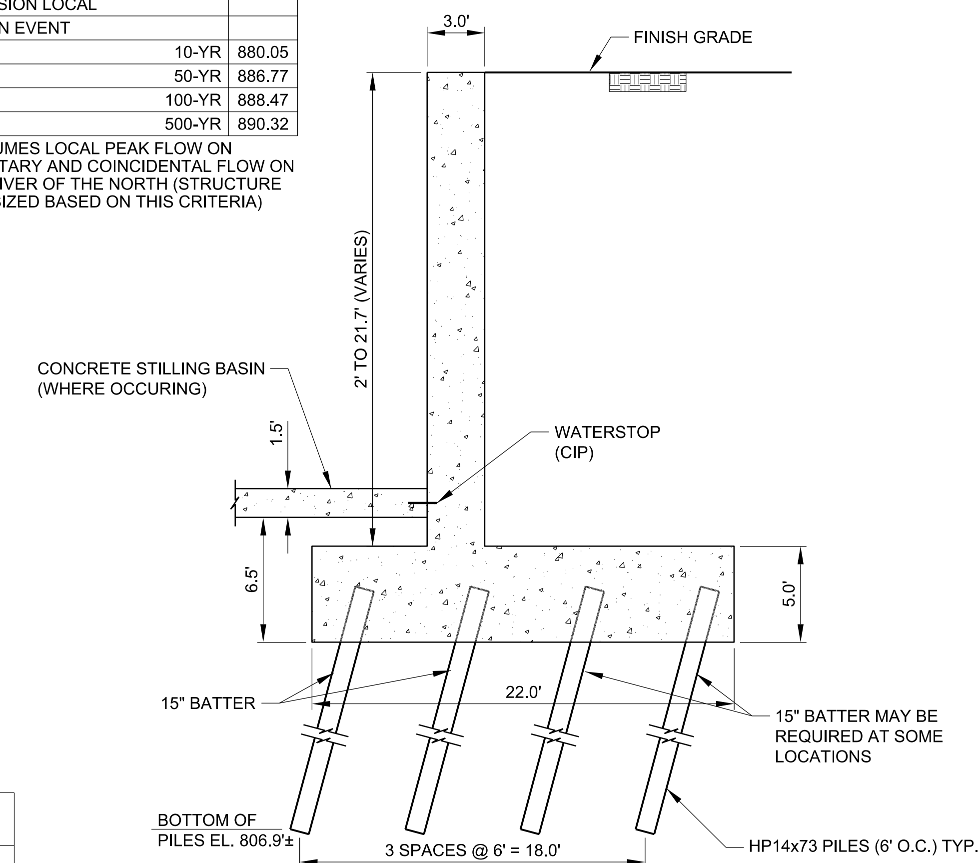
RUSH COINCIDENTAL		RUSH LOCAL	
DESIGN EVENT	WSEL	DESIGN EVENT	WSEL
10-YR	888.97	10-YR	886.68
50-YR	950.52	50-YR	887.98
100-YR	891.36	100-YR	888.77
500-YR	891.82	500-YR	890.63
DIVERSION COINCIDENTAL		DIVERSION LOCAL	
DESIGN EVENT		DESIGN EVENT	
10-YR	883.28	10-YR	880.05
50-YR	889.08	50-YR	886.77
100-YR	890.47	100-YR	888.47
500-YR	890.90	500-YR	890.32

* ASSUMES PEAK FLOWS ON RED RIVER OF THE NORTH AND COINCIDENTAL FLOW ON TRIBUTARY

* ASSUMES LOCAL PEAK FLOW ON TRIBUTARY AND COINCIDENTAL FLOW ON RED RIVER OF THE NORTH (STRUCTURE WAS SIZED BASED ON THIS CRITERIA)



3 S-442 SECTION
CONCRETE STEPPED DROP
SCALE: N.T.S.



4 S-442 SECTION
CONCRETE SIDE WALL
SCALE: N.T.S.

ALLOWABLE PILE CAPACITY AXIAL COMPRESSION	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	59.5
UNDRAINED UNUSUAL	79.3
UNDRAINED EXTREME	103.4
DRAINED USUAL	47.6

QUANTITIES (DROP STRUCTURE)

CONCRETE VOLUMES			STEEL	PILES			
APPROACH CY	STILLING CY	STEPS CY	REINF. LBS	NO.	NO. ROWS	LENGTH FT	TOTAL FT
70	170	517	118,720	34	17	45.5	1,547
				17	17	56	952
			TOTAL	51			2,499

QUANTITIES (WALLS)

PANEL	STEM 21.5 W (FT)	WALL LENGTH (FT)	STEM HEIGHT X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. CY	STEM CY	STEEL REINF. LBS	PILES NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	3	82'	17.5	22	6	334	160	66,087	56	28	71	3,976
B	3	409'	21.5	22	6	1,667	930	348,520	272	136	71	19,312
C	3	83'	16.7	22	6	338	154	65,724	56	28	55.5	3,108
D	3	15'	11	22	6	61	16	10,373	12	6	55.5	666
E	3	42'	8.0	22	6	171	31	26,827	14	7	55.5	777
					SOUTH WALL	2,571	889	517,531	410			27,839
					NORTH WALL	2,571	889	517,531	410			27,839
					TOTAL	5,142	2,582	1,035,062	820			55,678

NOTES:

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HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MWT/JGT/AMA
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-443HYD.dgn

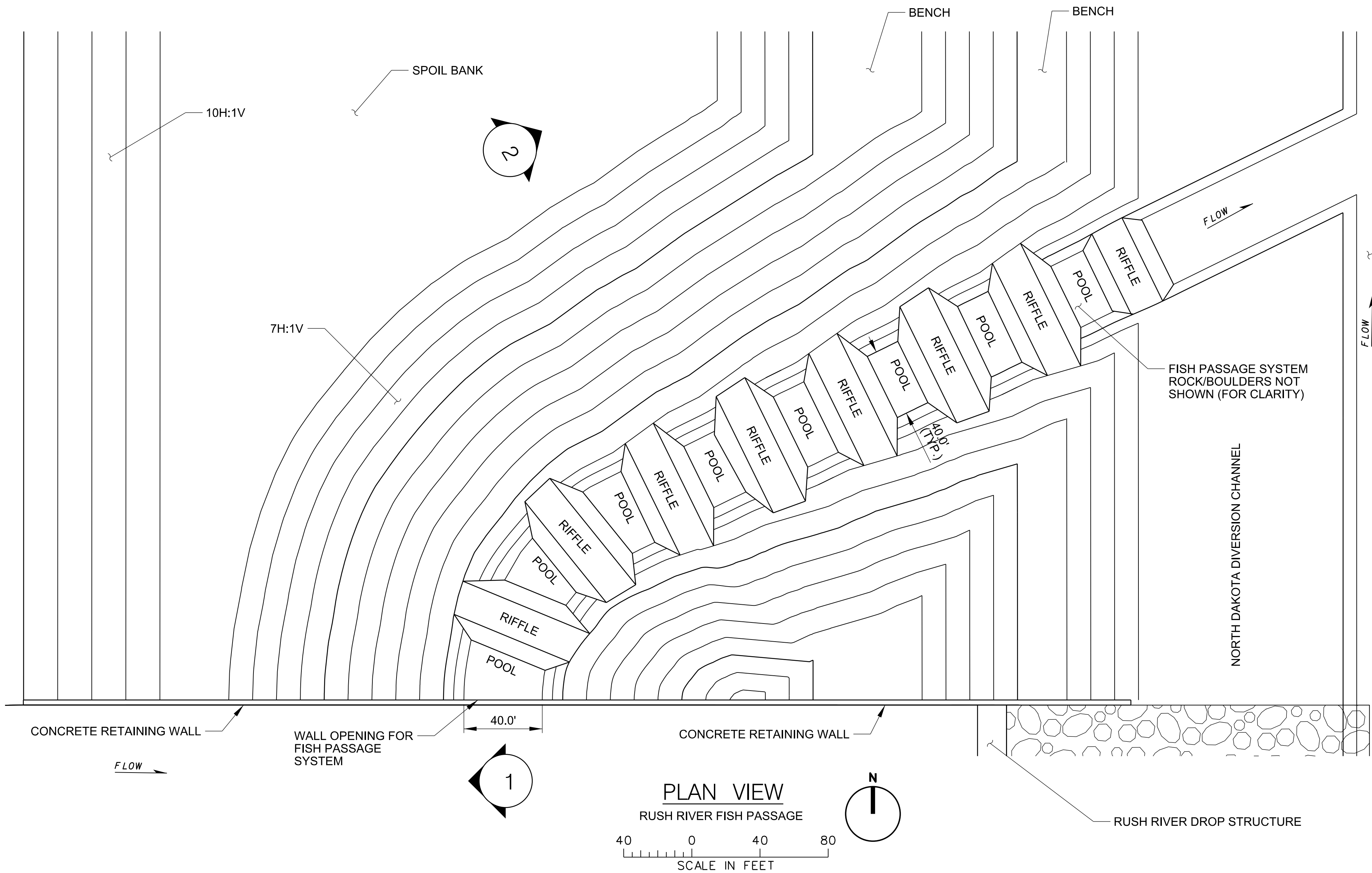
NORTH DAKOTA DIVERSION (LLP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

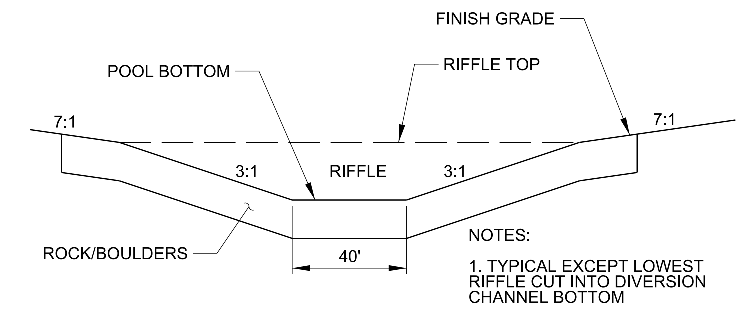
FLOOD CONTROL - RUSH RIVER
DROP STRUCTURE SECTIONS

SHEET IDENTIFICATION
S-443

auto_time



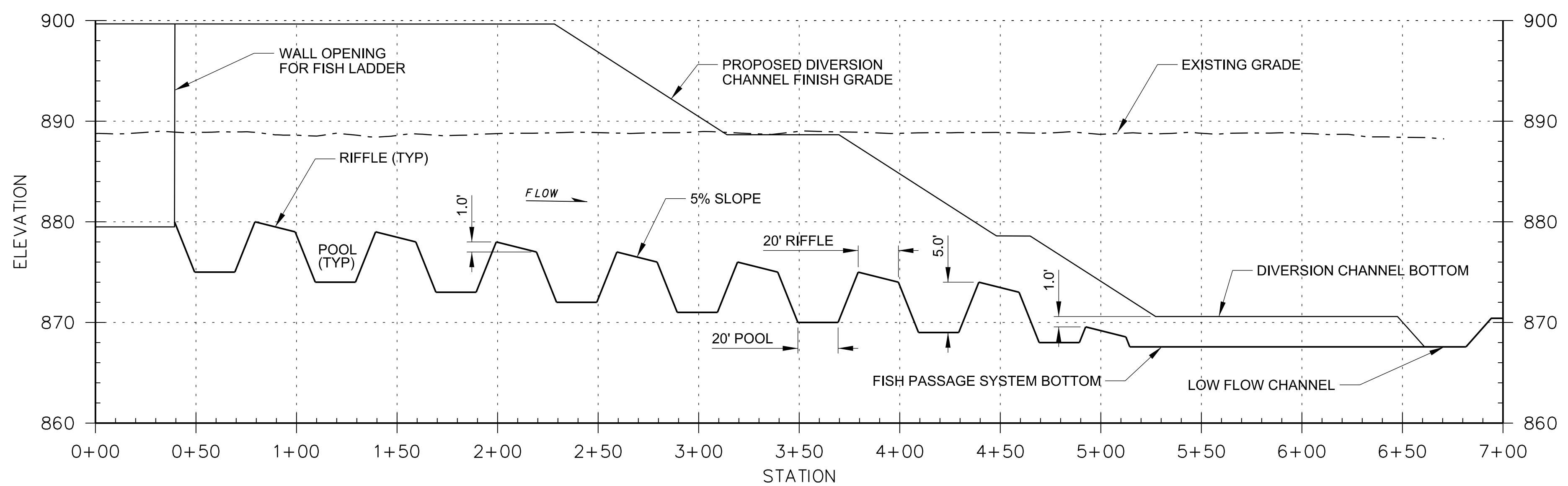
PLAN VIEW
RUSH RIVER FISH PASSAGE



NOTES:

1. TYPICAL EXCEPT LOWEST RIFFLE CUT INTO DIVERSION CHANNEL BOTTOM

2 SECTION
TYPICAL FISH PASSAGE
SCALE: N.T.S.



1 PROFILE
RUSH FISH PASSAGE SYSTEM-CENTERLINE
SCALE: N.T.S.

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. FISH PASSAGE SYSTEM ROCK EXTENTS NOT SHOWN (FOR CLARITY).

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

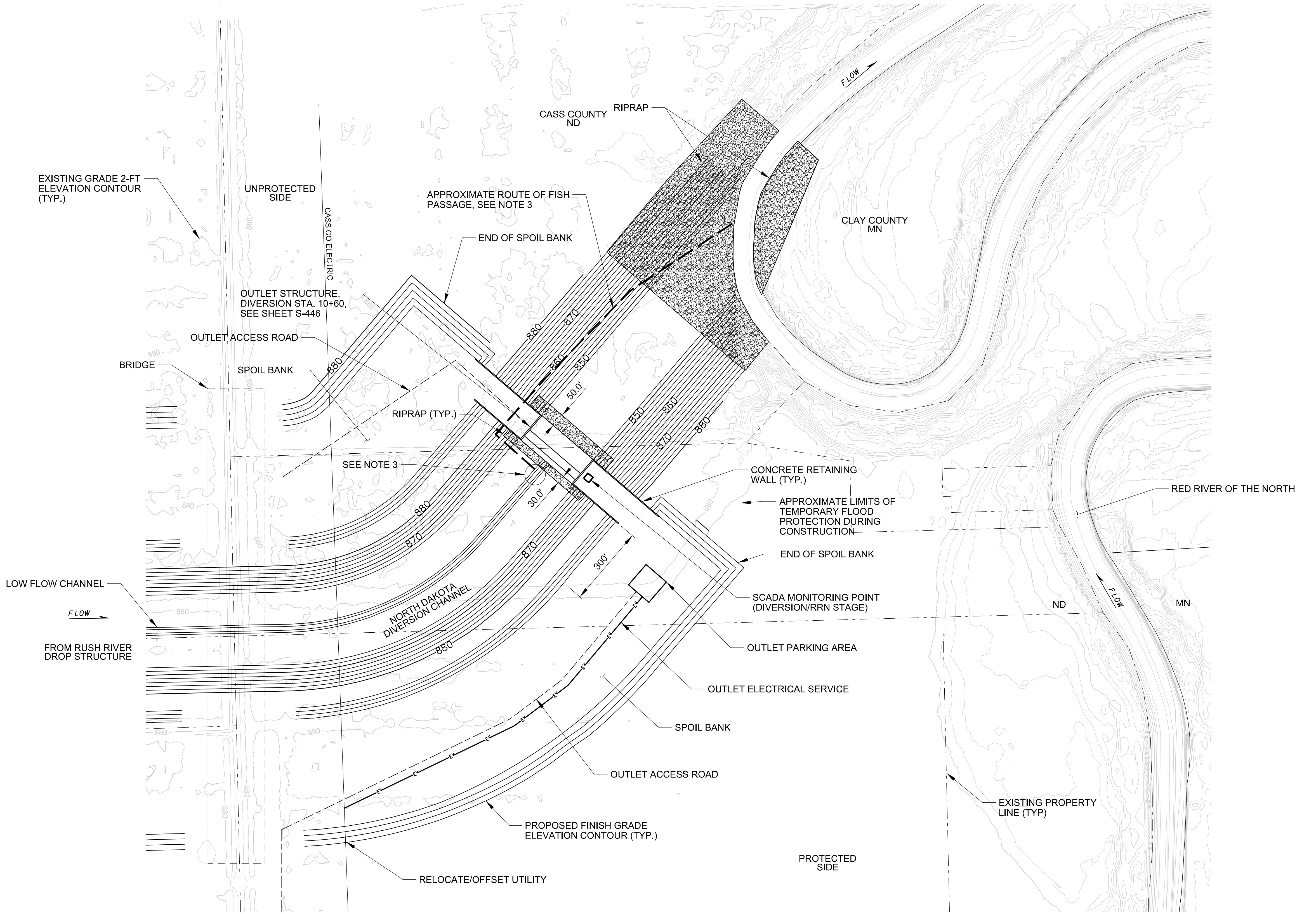
DESIGNED BY: MWT/SGW/JMA
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-444HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
RUSH RIVER FISH PASSAGE SYSTEM
PLAN VIEW & PROFILE

SHEET IDENTIFICATION
S-444



PLAN VIEW

LLP OUTLET STRUCTURE

200 0 200 400
SCALE IN FEET

N

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. FISH PASSAGE TO THE RED RIVER OF THE NORTH IS INCLUDED IN THE PROJECT COSTS BUT NOT SHOWN IN DETAIL IN THE PLANS.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

NORTH DAKOTA DIVERSION (LPP)

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

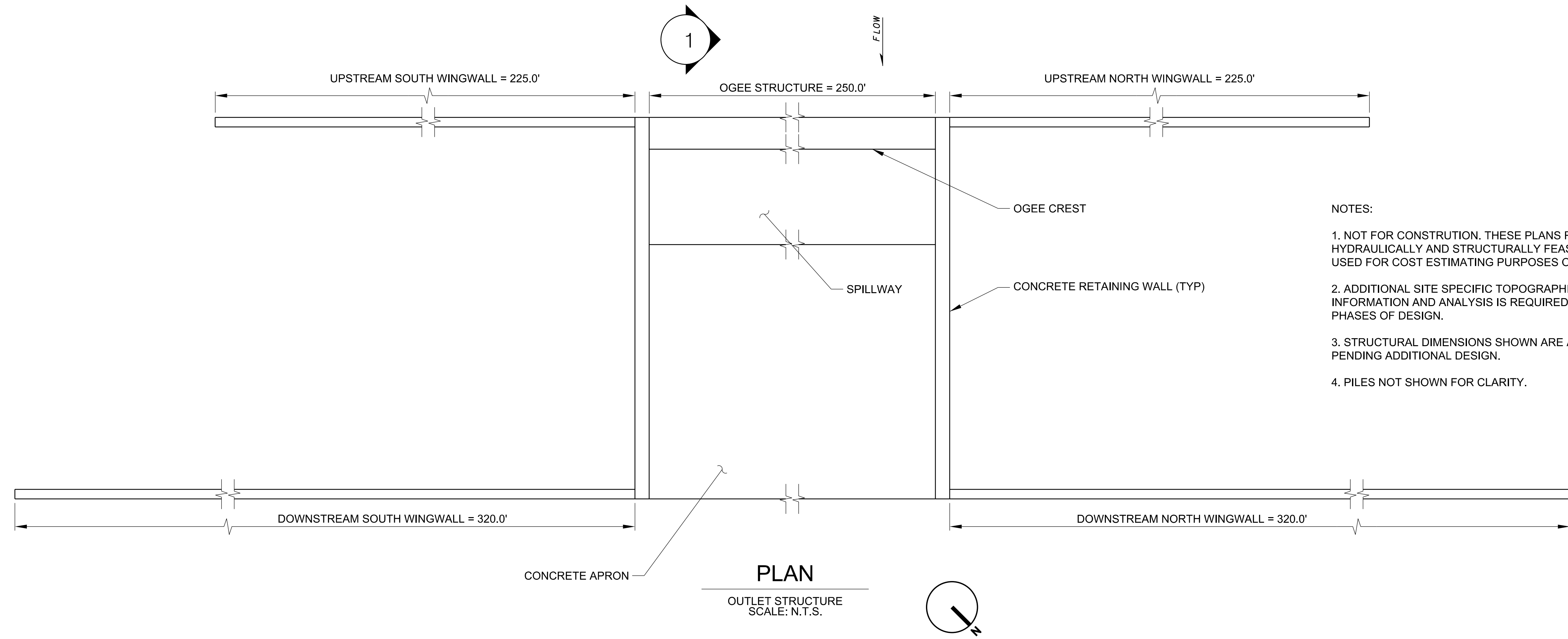
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MVI/PKN/JDA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: FMM_S-444HYD.dgn

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LPP OUTLET STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW

SHEET IDENTIFICATION
S-445



PLAN

OUTLET STRUCTURE
SCALE: N.T.S.

NOTES:

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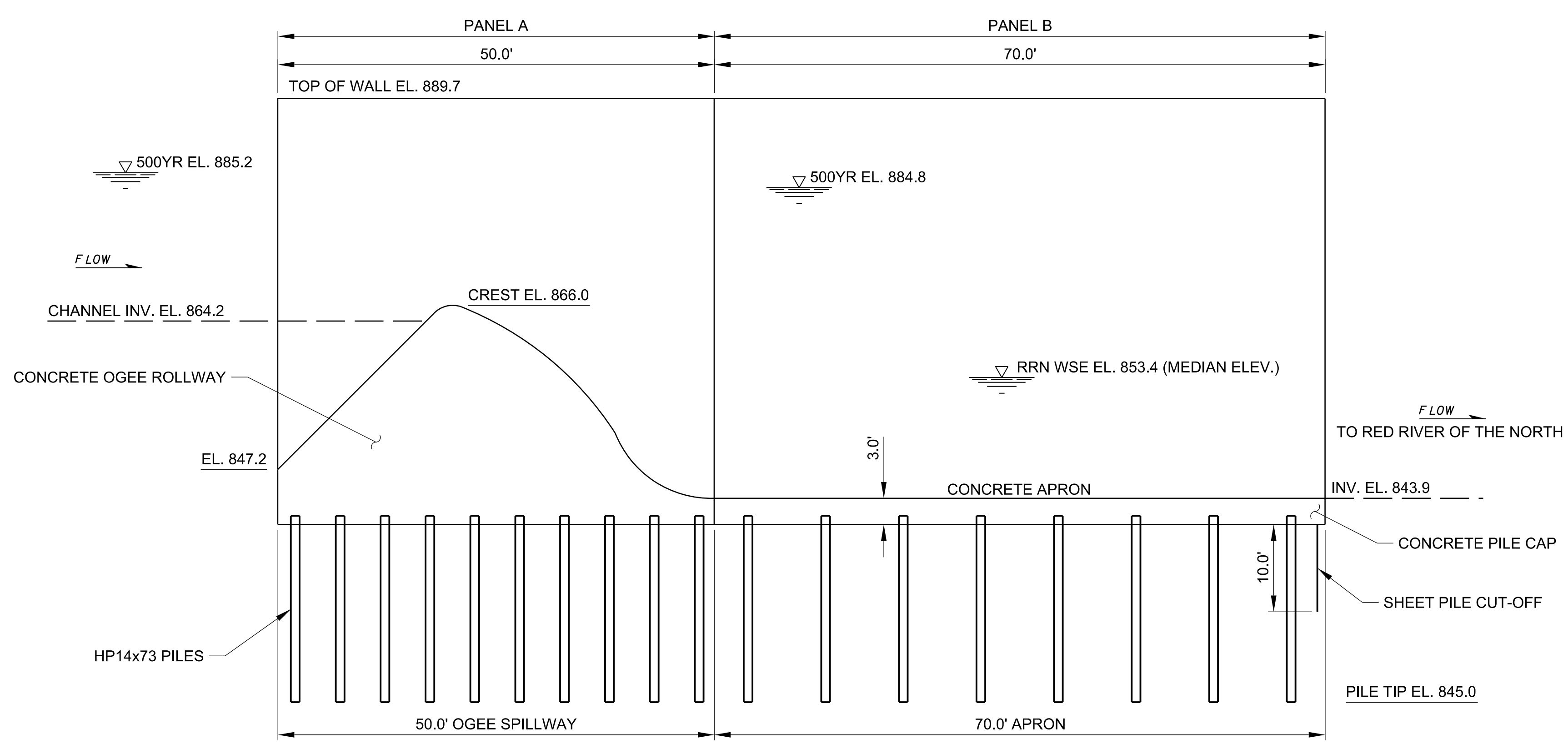
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. PILES NOT SHOWN FOR CLARITY.



QUANTITIES

PANEL	CONCRETE CY	STEEL REINF. LBS	PILES NO.	PILES NO. ROWS	PILES LENGTH FT	PILES TOTAL FT
OGEE SPILLWAY & APRON						
SPILLWAY	6,731	300,821	330	11	64	21,120
APRON	1,944	285,030	224	8	64	14,336
OGEE WINGWALLS (NORTH & SOUTH)						
A	458	33,480				
B	1,374	100,440				
TOTAL	10,507	719,771	554			35,456

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G

1 SECTION

DIVERSION OUTLET STRUCTURE
SCALE: N.T.S.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

DESIGNED BY: MWI/PKN/JDA
DWN BY: MJJ
CKD BY: PKN/BKL
SUBMITTED BY: N/A
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: S-446HYD.dgn

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

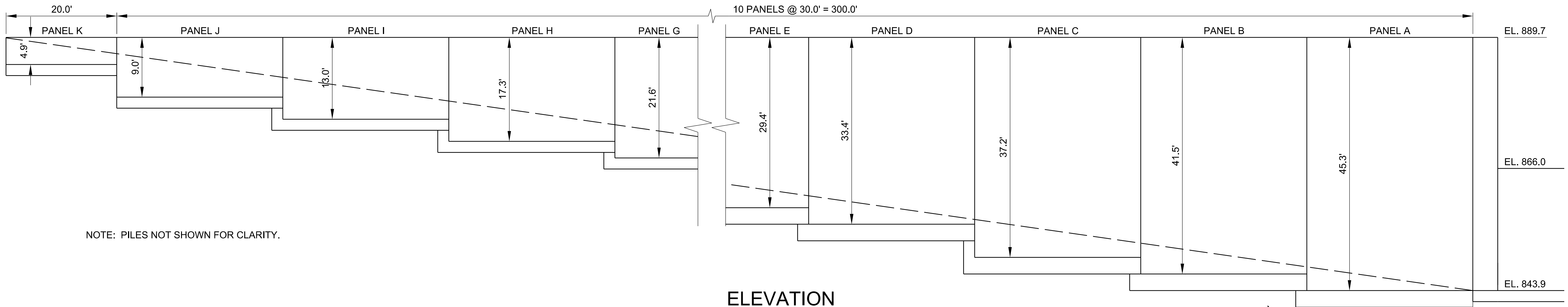
NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LPP OUTLET STRUCTURE TO RED RIVER OF THE NORTH
PLAN VIEW AND SECTION

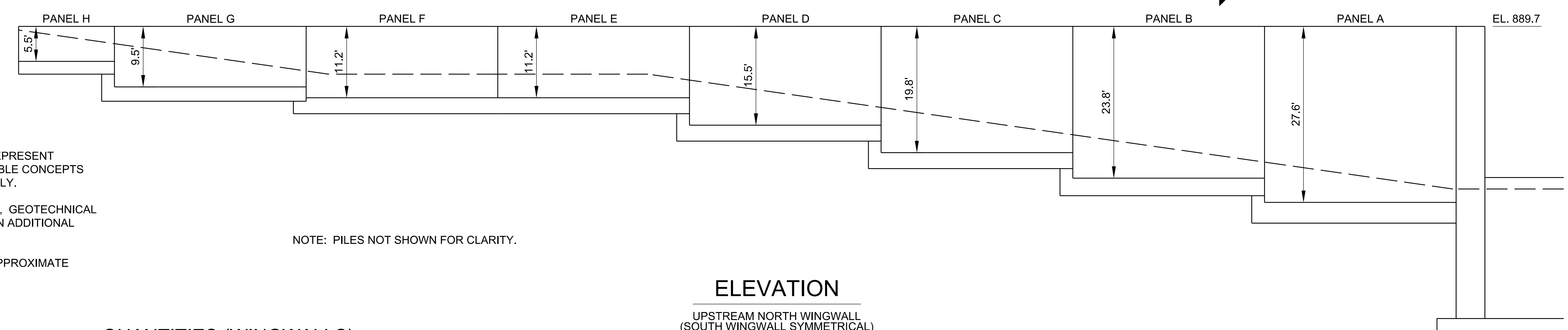
SHEET IDENTIFICATION
S-446

auto_time



ELEVATION

DOWNSTREAM SOUTH WINGWALL
(NORTH WINGWALL SYMMETRICAL)
SCALE: N.T.S.



NOTES:

1. NOT FOR CONSTRUTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
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4. PILES NOT SHOWN FOR CLARITY.

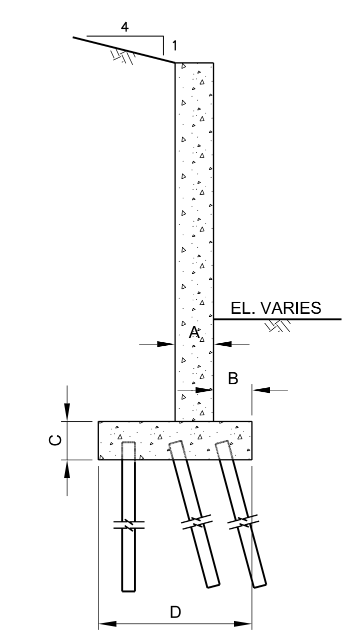
ELEVATION

UPSTREAM NORTH WINGWALL
(SOUTH WINGWALL SYMMETRICAL)
SCALE: N.T.S.

QUANTITIES (WINGWALLS)

PANEL	STEM A (FT)	FTG. TOE B (FT)	FTG. THICKNESS C (FT)	FTG. WIDTH D (FT)	CONCRETE VOLUMES FTG. CY	CONCRETE VOLUMES STEM CY	STEEL REINF. LBS	PILES NO.	PILES NO. ROWS	PILES LENGTH FT	PILES TOTAL FT
DOWNSTREAM											
A	4.5	8.0	4.5	23.0	115	227	20,940	20	4	63	1,260
B	4.5	7.5	4.5	21.0	105	208	20,940	20	4	68	1,360
C	4.0	7.0	4.0	21.0	93	165	16,440	20	4	72	1,440
D	3.5	6.5	3.5	20.0	78	130	13,467	18	4	76	1,368
E	2.8	6.2	3.3	15.5	56	90	14,190	17	3	81	1,377
F	2.7	5.8	3.0	14.0	47	75	11,420	16	3	85	1,360
G	2.5	4.8	2.5	12.0	33	60	9,761	12	3	89	1,068
H	2.3	3.5	2.5	9.5	26	44	5,830	9	3	93	837
I	2.1	2.7	2.5	7.7	21	31	3,908	6	2	98	588
J	2.0	2.0	2.3	6.0	15	20	2,759	6	2	102	612
K	1.8	1.8	2.0	5.8	9	7	1,527	4	2	106	424
UPSTREAM											
A	2.8	6.2	3.3	15.5	56	84	14,180	17	3	82	1,394
B	2.6	5.3	2.8	13.0	40	68	10,951	13	3	87	1,131
C	2.4	4.3	2.5	11.0	31	53	8,098	10	3	91	910
D	2.3	3.3	2.5	9.0	25	39	5,407	8	3	95	760
E	2.0	2.3	2.5	6.7	19	25	3,329	6	2	99	594
F	2.0	2.3	2.5	6.7	19	25	3,329	6	2	99	594
G	2.0	2.2	2.3	6.3	16	21	3,020	6	2	101	606
H	1.8	1.8	2.0	5.8	8	6	1,251	4	2	106	424
				NORTH WINGWALL	808	1,378	170,749	218			18,107
				SOUTH WINGWALL	808	1,378	170,749	218			18,107
				TOTAL	1,616	2,756	341,498	436			36,214

NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G



1 SECTION
CONCRETE WINGWALL
SCALE: N.T.S.

ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	56.3
UNDRAINED UNUSUAL	75.0
UNDRAINED EXTREME	97.9
DRAINED USUAL	30.2

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

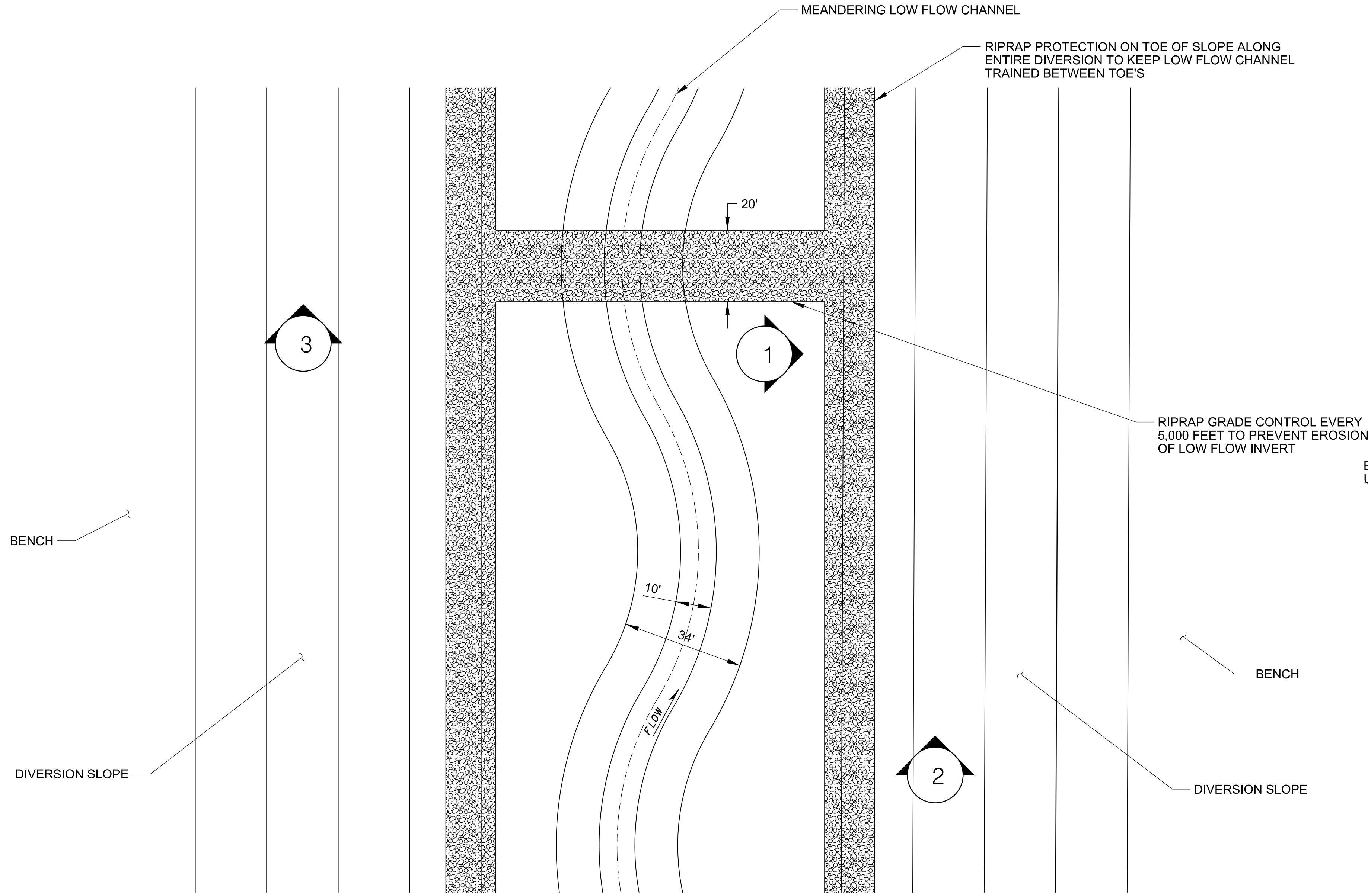
DESIGNED BY: MW1/PKN/JDA
DATE: 28FEB2011
DWN BY: MJJ
CKD BY: PKN/BKL
SOLICITATION NO.: N/A
SUBMITTED BY: N/A
CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
FILE NUMBER: N/A
SIZE: ANSI D
FILE NAME: S-447HYD.dgn

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - LPP OUTLET STRUCTURE
TO RED RIVER OF THE NORTH
ELEVATIONS AND SECTION

SHEET IDENTIFICATION
S-447

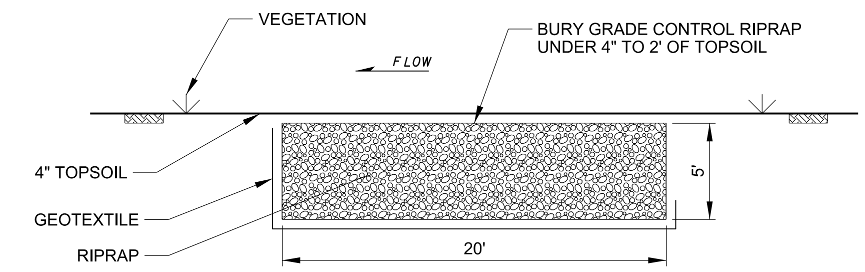


PLAN VIEW

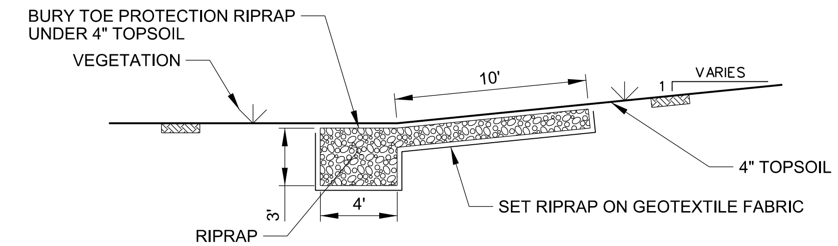
RRD - LOW FLOW CHANNEL

20 0 20 40
SCALE IN FEET

N



1 SECTION
RIPRAP GRADE CONTROL
SCALE: N.T.S.



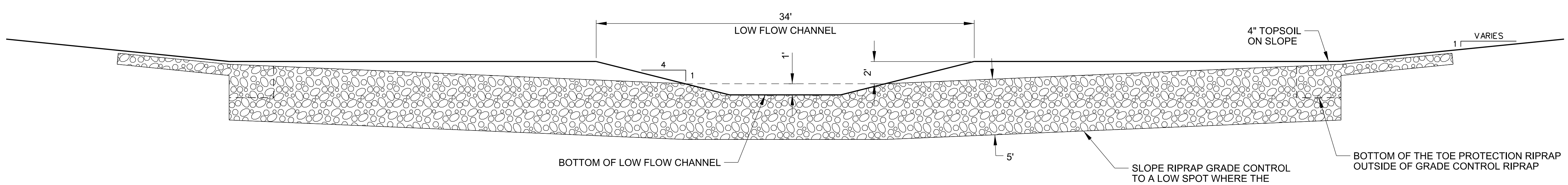
2 SECTION
TOE PROTECTION RIPRAP
SCALE: N.T.S.

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC AND GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. LOW FLOW CHANNEL DIMENSIONS MUST BE REFINED ONCE LOW FLOW HYDROLOGY IS CONFIRMED.



3 SECTION
GRADE CONTROL
SCALE: N.T.S.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

US Army Corps of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

NORTH DAKOTA DIVERSION (LLP)

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

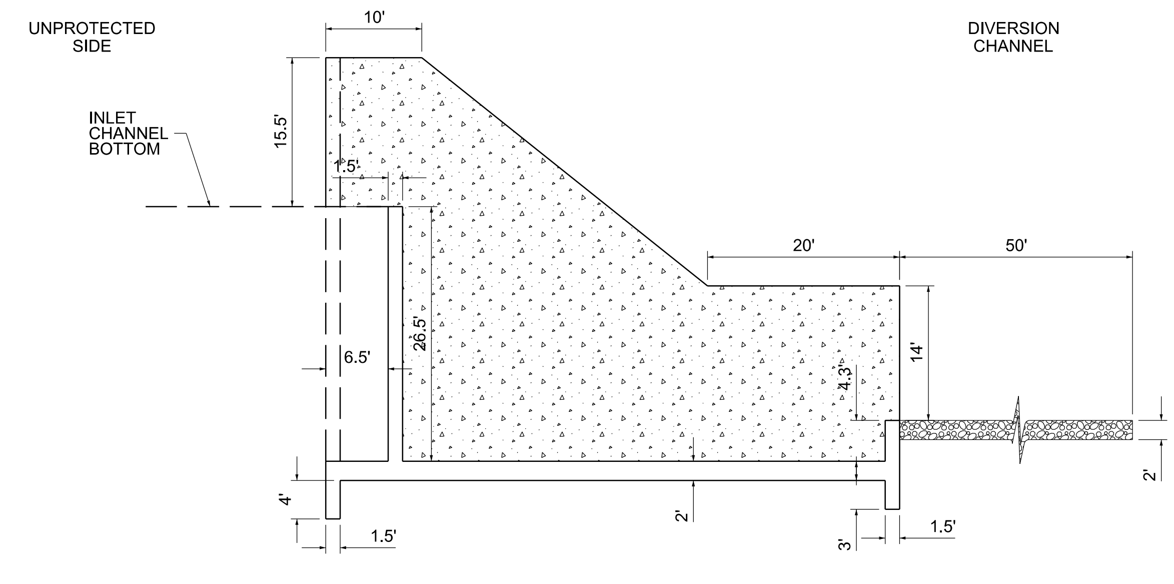
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MRM/MW1/TEM		DATE: 28FEB2011
DWN BY: MJJ	CKD BY: BKL	SOLICITATION NO.: N/A
SUBMITTED BY: N/A		CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN	PLOT DATE: 28FEB2011	FILE NUMBER: N/A
SIZE: ANSI D	FILE NAME: FMM_S-448HYD.dgn	

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

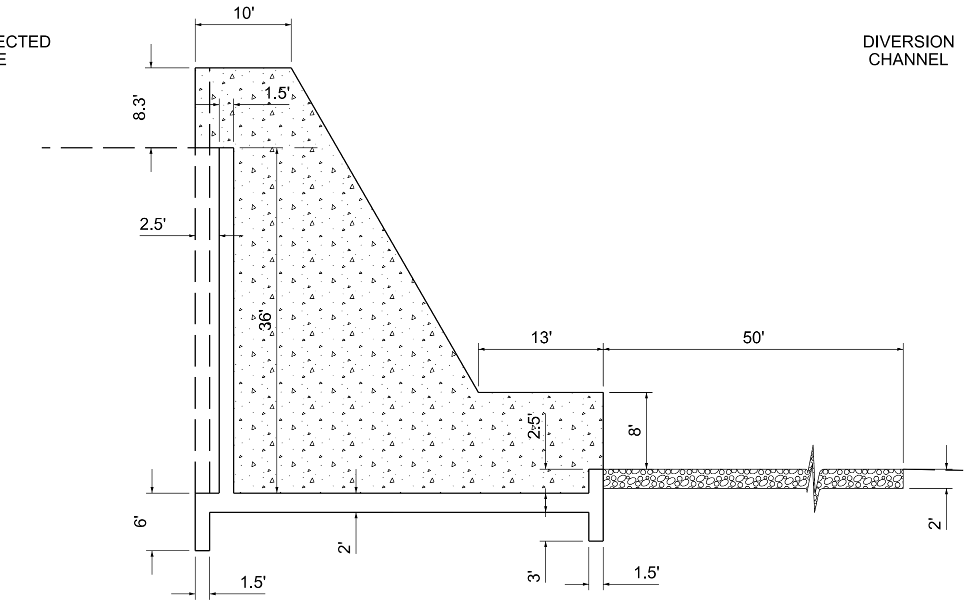
FLOOD CONTROL - LOW FLOW CHANNEL
DOWNSTREAM OF LOWER RUSH RIVER
PLAN VIEW & SECTIONS

SHEET IDENTIFICATION
S-448



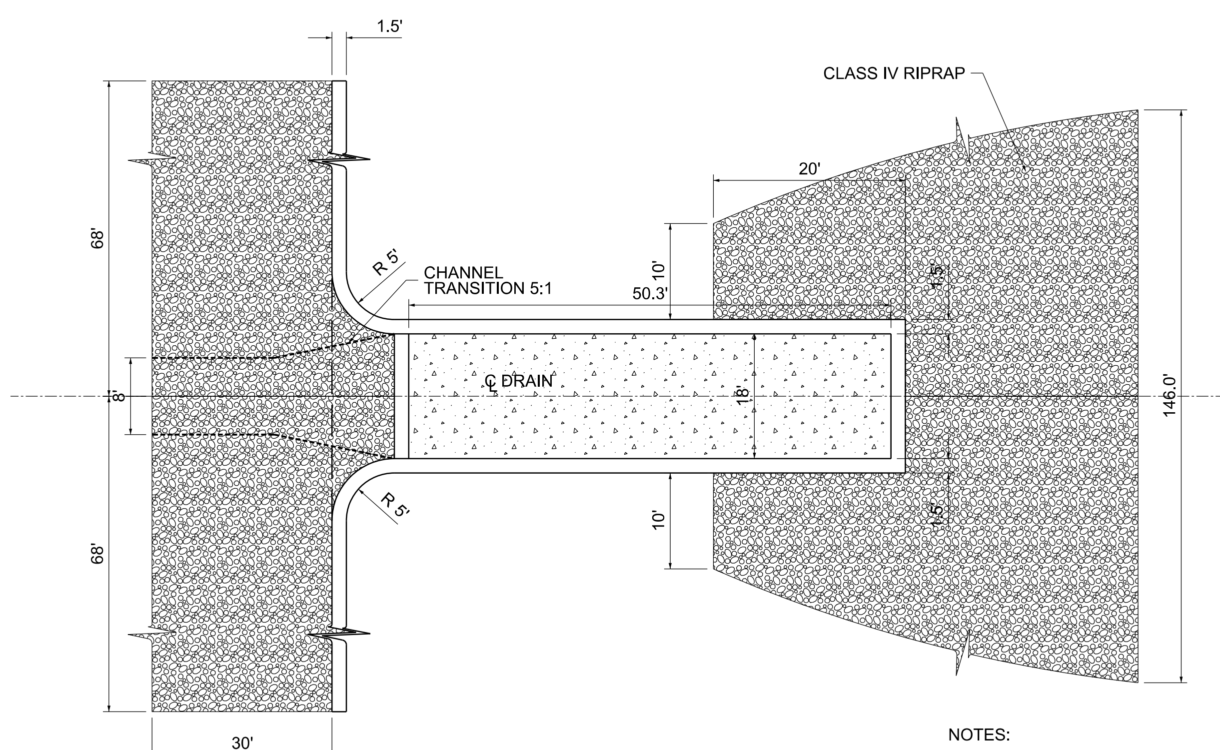
1 CENTERLINE SECTION

DESIGN 1
SCALE: N.T.S.



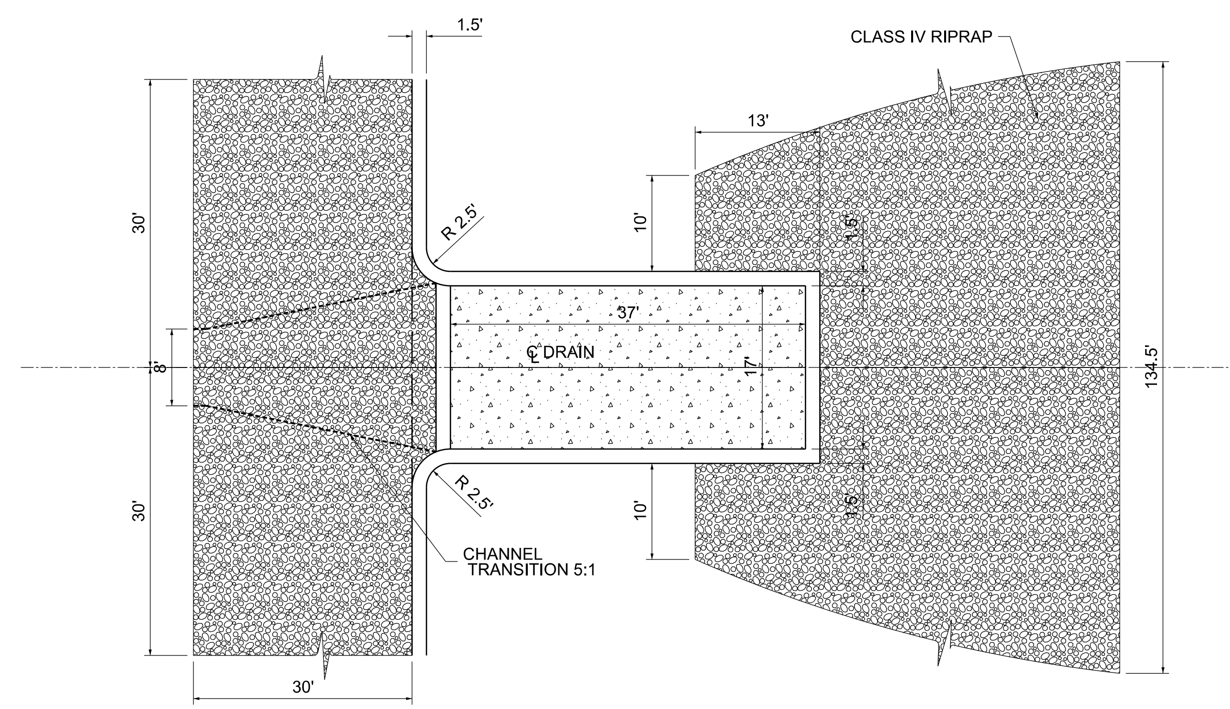
3 CENTERLINE SECTION

DESIGN 2
SCALE: N.T.S.



2 PLAN VIEW

DESIGN 1
SCALE: N.T.S.



4 PLAN VIEW

DESIGN 2
SCALE: N.T.S.

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

REFERENCE: USACE EM1110-2-1601 PLATE B-48

4.DETERMINE PILE OR OTHER FOUNDATION REQUIREMENTS DURING FINAL DESIGN.

5. ELEVATION AND DIMENSIONS SHOWN ARE FOR REFERENCE ONLY AND WILL VARY BY LOCATION. DETERMINE ACTUAL ELEVATIONS AND DIMENSIONS DURING FINAL DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

NORTH DAKOTA DIVERSION (LLP) AND MINNESOTA DIVERSION (FLP)

US Army Corps
of Engineers®
St. Paul District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-				

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

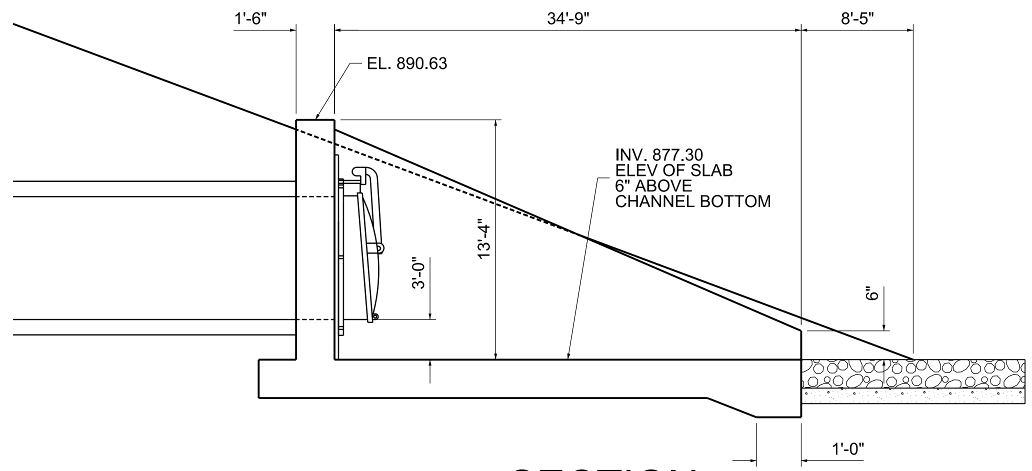
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MEI | DATE: 28FEB2011
DWN BY: MJJ | CKD BY: BKL | SOLICITATION NO.: N/A
SUBMITTED BY: N/A | CONTRACT NO.: N/A
PLOT SCALE: AS SHOWN | PLOT DATE: 28FEB2011 | FILE NUMBER: N/A
SIZE: ANSI D | FILE NAME: FMM_S-449HYD.dgn

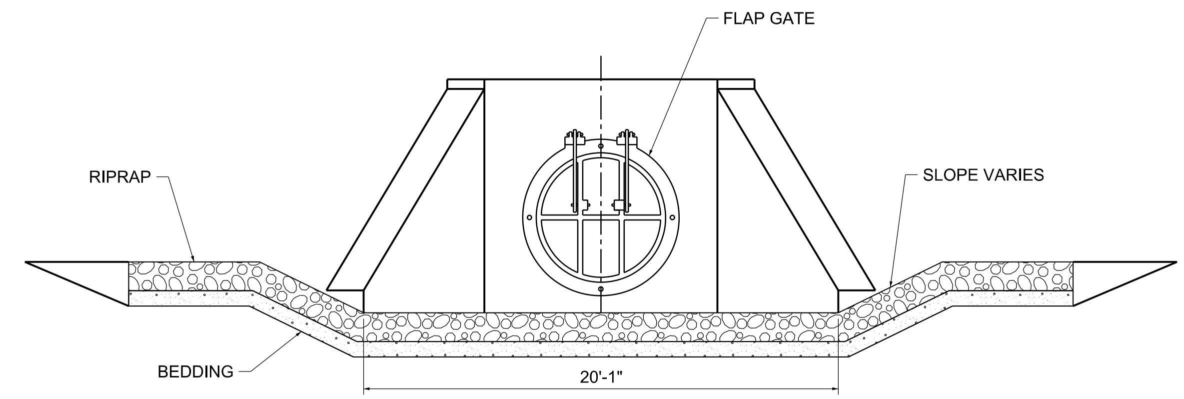
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL - SIDE CHANNEL
CONTROL STRUCTURES (TYPICAL)
PLAN VIEW AND SECTIONS

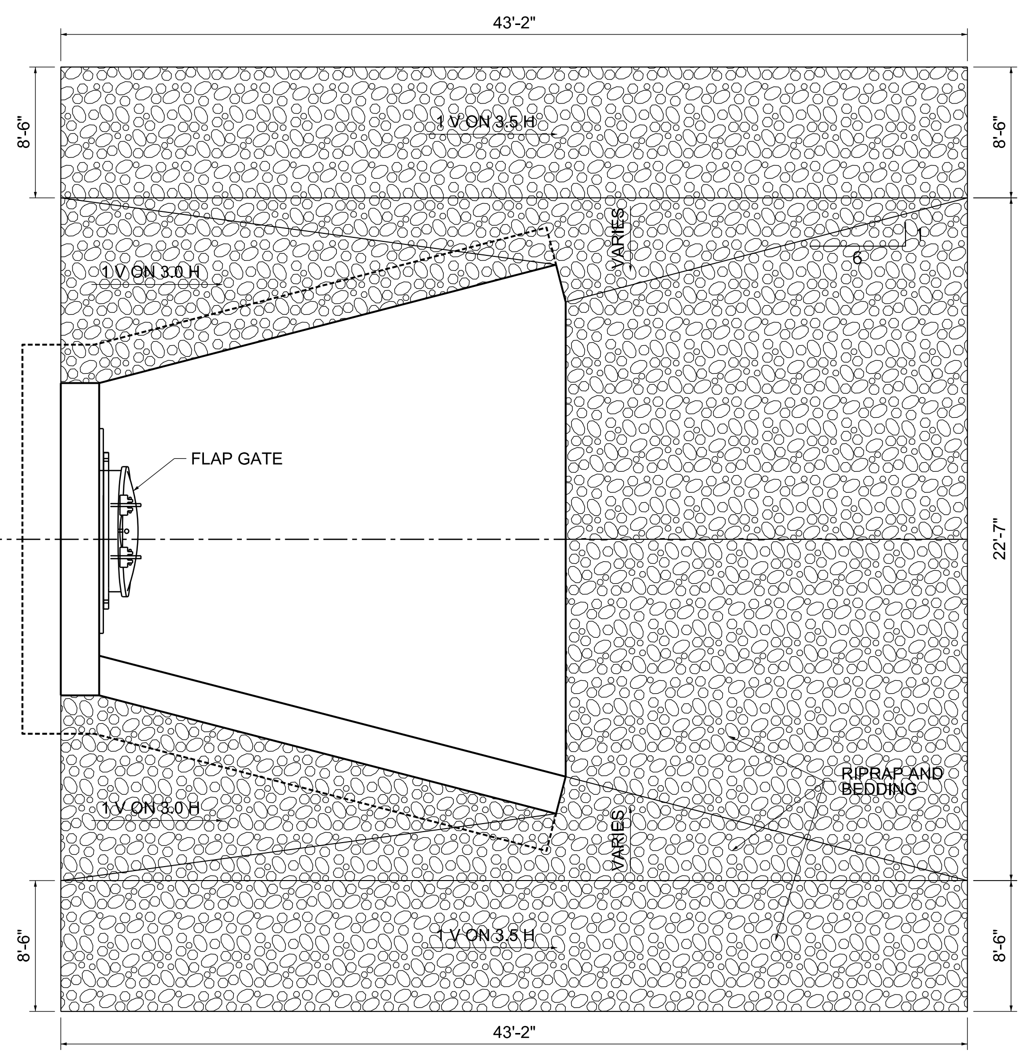
SHEET
IDENTIFICATION
S-449



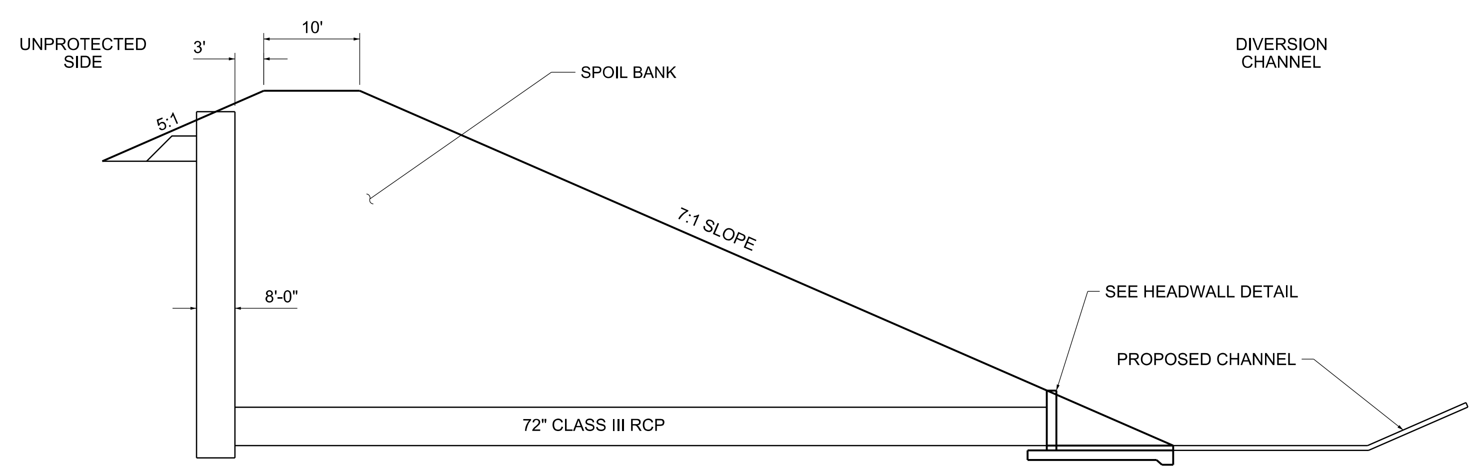
1 SECTION
SIDE DITCH INLET
SCALE: N.T.S.



3 SECTION
SIDE DITCH INLET
SCALE: N.T.S.



2 PLAN
SIDE DITCH INLET
SCALE: N.T.S.



4 PROFILE
SIDE DITCH INLET
SCALE: N.T.S.

NOTES:

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3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. ELEVATIONS AND DIMENSIONS SHOWN ARE FOR REFERENCE ONLY AND WILL VARY BY LOCATION. DETEMINE ACTUAL LOCATIONS ELEVATIONS AND DIMENSIONS DURING FINAL DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

NORTH DAKOTA DIVERSION (LLP) AND MINNESOTA DIVERSION (FLP)

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH 4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

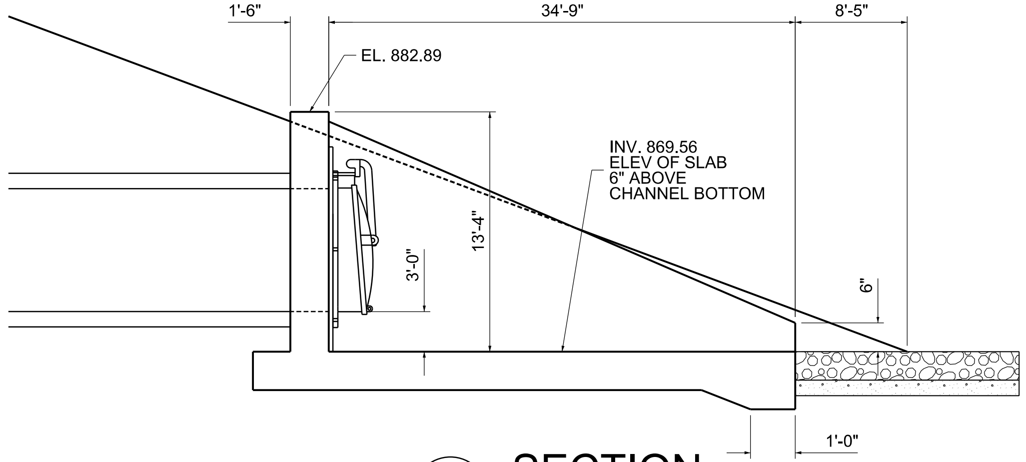
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MEI
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-450HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

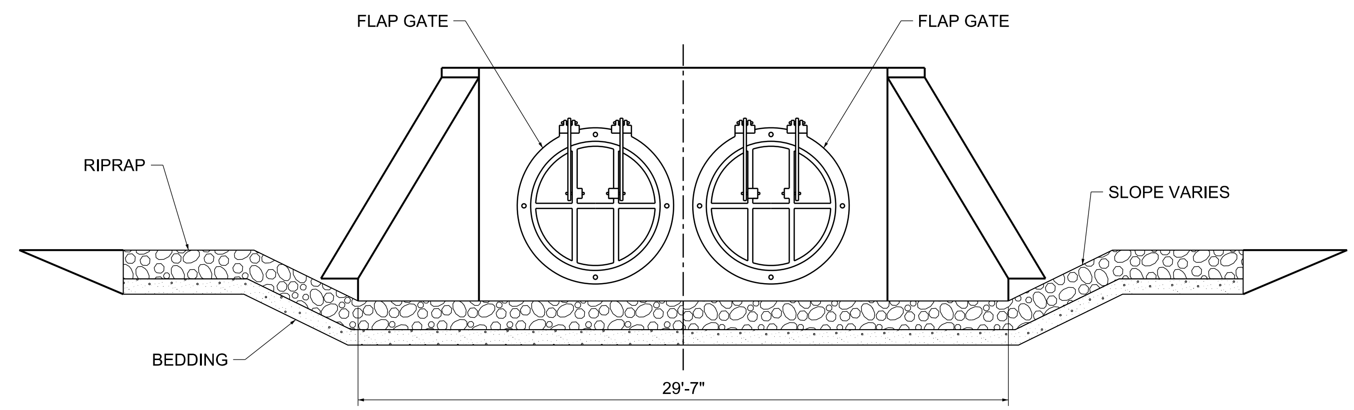
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
SIDE INLET 72" RCP (TYPICAL)
PLAN VIEW AND SECTIONS

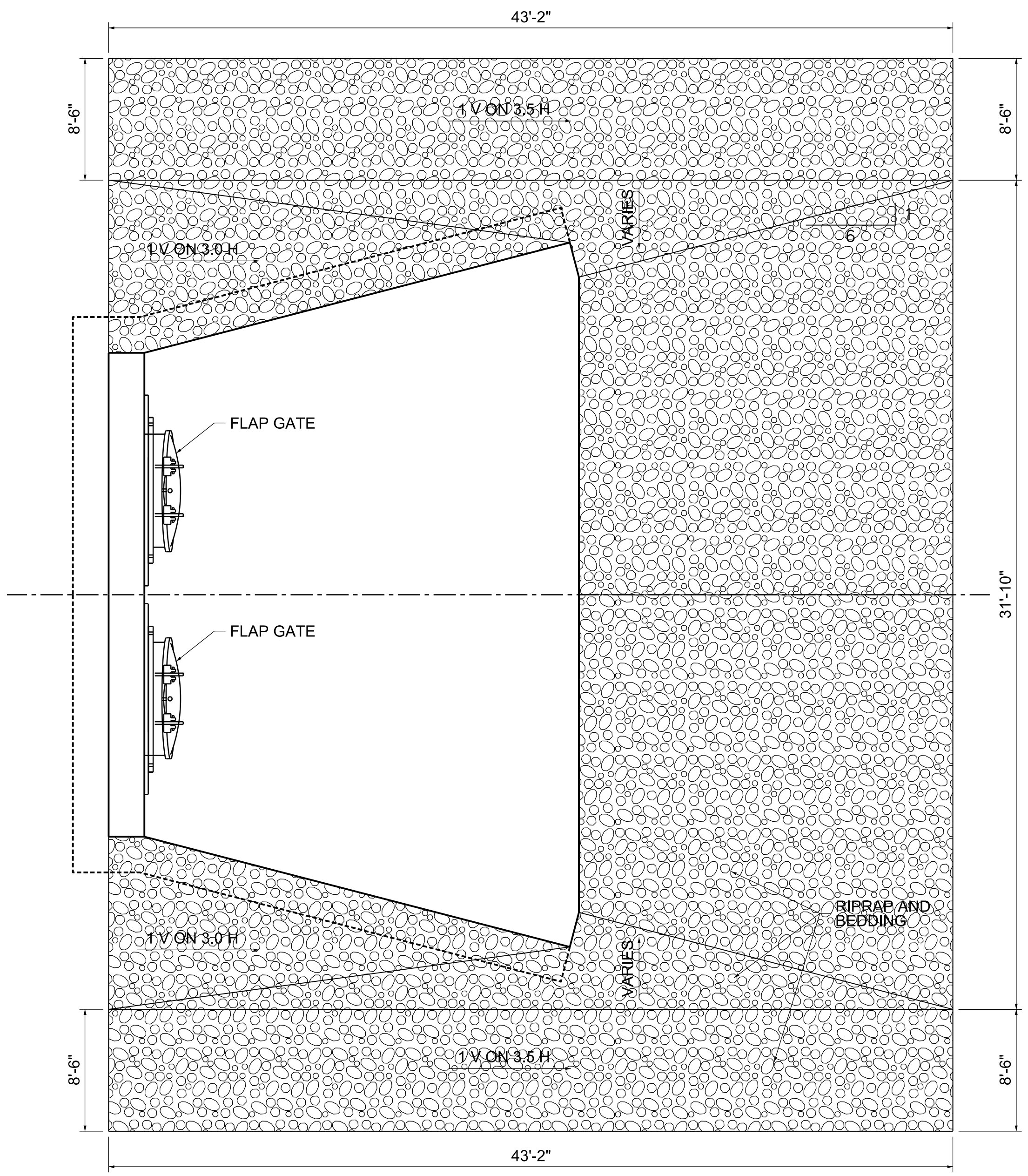
SHEET
IDENTIFICATION
S-450



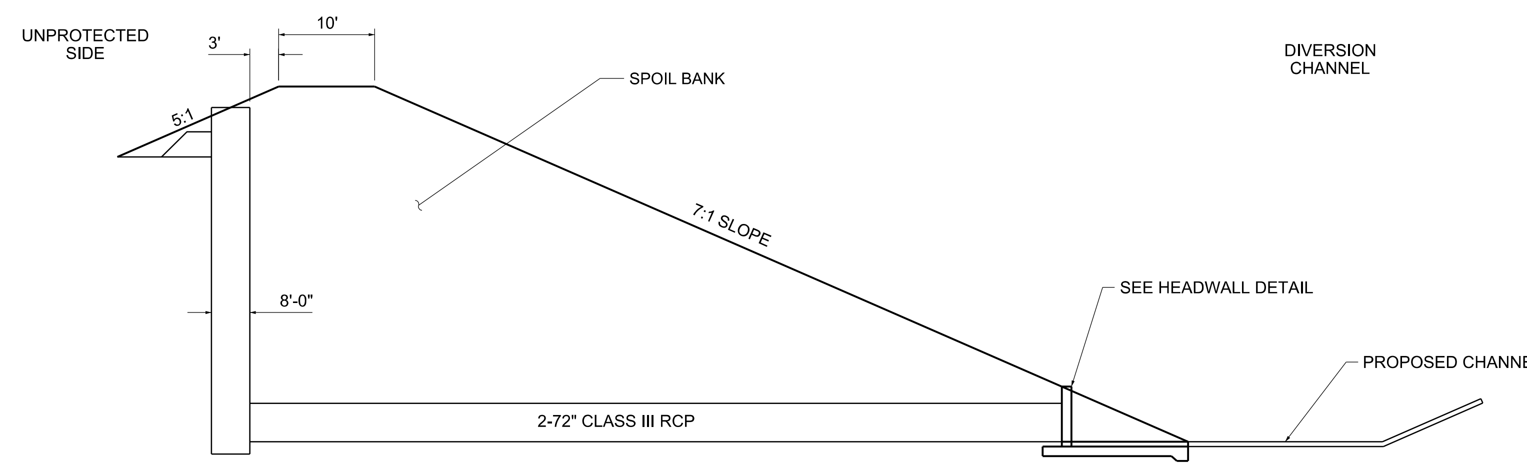
1 SECTION
SIDE DITCH INLET
SCALE: N.T.S.



3 SECTION
SIDE DITCH INLET
SCALE: N.T.S.



2 PLAN
SIDE DITCH INLET
SCALE: N.T.S.



4 PROFILE
SIDE DITCH INLET
SCALE: N.T.S.

NOTES:

1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.

2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.

4. ELEVATIONS AND DIMENSIONS AHOWN ARE FOR REFERENCE ONLY AND WILL VARY BY LOCATION. DETERMINE ACTUAL ELEVATIONS AND DIMENSIONS DURING FINAL DESIGN.

HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET

VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

NORTH DAKOTA DIVERSION (LLP) AND MINNESOTA DIVERSION (FLP)

US Army Corps of Engineers
St. Paul District

MARK	DESCRIPTION	DATE	APPR
-	PH4 CONCEPT - NOT FOR CONSTRUCTION	28FEB2011	-

U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435

DESIGNED BY: MEI
DWN BY: MJJ
CKD BY: BKL
SUBMITTED BY: N/A
PLOT SCALE: AS SHOWN
PLOT DATE: 28FEB2011
SIZE: ANSI D
FILE NAME: FMM_S-451HYD.dgn
DATE: 28FEB2011
SOLICITATION NO.: N/A
CONTRACT NO.: N/A
FILE NUMBER: N/A

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL
SIDE INLET 2-72" RCP (TYPICAL)
PLAN VIEW AND SECTIONS

SHEET IDENTIFICATION
S-451